

# DRAFT



## DRAFT FOUNDATION INVESTIGATION AND DESIGN REPORT

### Pipe Culvert Extensions Highway 17 and Municipal Road 55 West Junction Intersection Improvements Denison Township, District of Sudbury Ministry of Transportation, Ontario GWP 5032-19-00

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## **PART A**

FOUNDATION INVESTIGATION REPORT  
PIPE CULVERT EXTENSIONS  
MUNICIPAL ROAD 55 – STATION 10+147 DENISON TOWNSHIP  
HIGHWAY 17 EBL – STATION 14+563 DENISON TOWNSHIP  
DISTRICT OF SUDBURY  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5032-19-00

## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail design foundation engineering services for the proposed Highway 17 and Municipal Road 55 (MR55) west junction intersection improvements in the Denison Township in the District of Sudbury, Ontario. As part of the intersection improvements, widening is being proposed along Highway 17 between about STA 14+300 and 14+725. In addition, a new south-east (S-E) ramp is to be constructed at the Hwy 17 / MR55 intersection. To accommodate the proposed widening and ramp, two existing corrugated plastic pipe culverts and one existing structural box culvert (i.e., the Fairbanks Creek culvert) near the Highway 17 and MR55 intersection will need to be extended.

This report addresses the investigation for the above noted pipe culvert extensions. The investigations for the proposed Highway 17 widening and the structural box culvert extension are addressed in separate reports.

One of the corrugated plastic pipe culverts is located on MR55 at about STA 10+147 (i.e., about 123 m south of the Highway 17 intersection; latitude 46.376178, longitude -81.345475) and the other culvert is located within the eastbound lane of Highway 17 at about STA 14+563 (i.e., about 265 m east of the MR 55 intersection; latitude 46.378611, longitude -81.344722). The general locations of the culverts are shown on the Key Plans on Drawings 1 and 2, respectively.

## 2.0 SITE DESCRIPTION

It should be noted that the orientation (i.e., north, south, east, west) stated in the text of this report is typically referenced to project north with Highway 17 running in a west/east direction and, therefore, may differ from magnetic north shown on the drawing.

In general, the topography of this area consists of rolling terrain and numerous bedrock outcrops separated by low-lying swamps with areas of standing water and various vegetation types and organic soils. The land use in the general area includes residential developments with scattered rural farm use.

### Existing Culvert – MR 55 STA 10+147

For the purpose this report, MR55 is oriented in a north-south direction with the culvert oriented in a generally west-east orientation on a skew from perpendicular to the roadway. The culvert is located near a horizontal curve and within a superelevated section of the roadway. The centerline of the road at the culvert location is at approximately Elevation 244.3 m and the existing embankments are about 2.9 m and 2.1 m high on the west and east sides of the road, respectively. The existing west and east embankment side slopes adjacent to the culvert are inclined at about 2.9 horizontal to 1 vertical (2.9H:1V) and 2.7H:1V, respectively. The roadway and surrounding ground surface conditions in the area of the culvert are shown on Photographs 1 to 5.

Based on the survey profile provided by AECOM, we understand the existing culvert consists of a 900 mm diameter corrugated plastic pipe culvert with inverts at about Elevation 241.5 m and 241.0 m at the west (inlet) and east (outlet) ends, respectively.

Based on our site observations, the existing roadway embankments in the culvert area appears to be performing satisfactorily with no visual evidence of instability (i.e., soil movement) on the embankment side slopes and no tension cracks near the embankment crest that would be indicative of instability. There are signs of pavement related distresses (i.e., longitudinal and map cracking) within the road immediately south of the culvert alignment; however, it is anticipated that these distresses are related to pavement deficiencies rather than slope instability or lack of foundation support for the existing culvert. In addition, granular sheeting / rip-rap is present along the west and east side slopes at the culvert inlet and outlet; however, there were no signs of erosion.

## Existing Culvert – Hwy 17 EBL STA 14+563

For the purpose this report, the Hwy 17 EBL is oriented in a west-east direction with the culvert oriented in a generally north-south orientation on a slight skew from perpendicular to the highway. The highway grade at the culvert location is at approximately Elevation 243.5 m and the existing highway embankments are about 2.4 m and 2.9 m high on the north and south sides of the road, respectively. The existing north and south embankments are inclined at about 3.0H:1V and 2.0H:1V along the north and south sides of the road, respectively. The roadway and surrounding ground surface conditions in the area of the culvert are shown on Photographs 6 to 10.

Based on the survey profile provided by AECOM, we understand the existing culvert consists of a 900 mm diameter plastic pipe (corrugated or profile wall) culvert with inverts at about Elevation 240.0 m and 239.9 m at the north (inlet) and south (outlet) ends, respectively.

Based on our site observations and a review of the available satellite images, the existing highway embankment in the culvert area appears to be performing satisfactorily with no visual evidence of instability (i.e., soil movement) on the embankment side slopes and no tension cracks near the embankment crest that would be indicative of instability. In addition, there were no observed signs of active erosion.

## 3.0 INVESTIGATION PROCEDURES

The field work for the subsurface explorations associated with the culvert extensions were carried out from February 1 to 8, 2021, during which time four boreholes (Boreholes 21-01, 21-02, 21-07 and 21-08) were advanced at the approximate locations shown in Drawings 1 and 2. Boreholes 21-01, 21-02, and 21-07 were advanced using a track-mounted CME-55 drilling rig and Borehole 21-08 was advanced using portable tripod drilling equipment. The CME-55 drilling rig and portable tripod drilling equipment were supplied and operated by Landcore Drilling (Landcore) of Sudbury, Ontario. Traffic control was performed in accordance with the Ontario Traffic Control Manual Book 7 – Temporary Conditions by Beacon Lite Ltd. of Sudbury, Ontario.

The boreholes were advanced using 108 mm inside diameter hollow stem augers or NW casing with wash boring techniques. Water from the local creek/stream was used to facilitate wash boring operations. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic hammer (for Boreholes advanced using the CME-55 drilling rig) or a rope-and-cathead hammer (for Borehole 21-08 advanced using the portable tripod) in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586)<sup>1</sup>. Select samples of the cohesive soils were obtained using 76 mm O.D. thin-walled Shelby Tubes (ASTM D1587)<sup>2</sup>. In situ vane shear tests were carried out in cohesive soils for measurement of undrained shear strengths (ASTM 2573)<sup>3</sup> using an MTO standard 'N'-size vane. The groundwater levels were measured upon completion of drilling and a temporary standpipe piezometer was installed in Borehole 21-01 to obtain a more stabilized ground level, as described on the borehole records provided in Appendix A. The boreholes were backfilled, and the temporary piezometer was decommissioned in general accordance with Ontario Regulation 903 Wells (as amended).

<sup>1</sup> ASTM D1586/D1586M-18 Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils

<sup>2</sup> ASTM D1587/D1587M-15 Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes

<sup>3</sup> ASTM D2573/D2573M-18 Standard Test Method of Field Vane Shear Test in Saturated Fine-Grained Soils

The field work was supervised on a full-time basis by a member of Golder's technical staff who: located the boreholes in the field; arranged for the clearance of underground services; supervised the drilling and sampling operations; logged the boreholes; and examined the soil samples. The soil samples were identified in the field, placed in labelled containers, and transported to Golder's geotechnical laboratory in Sudbury for further examination and laboratory testing. Index and classification testing consisting of water content determinations, grain size distributions, Atterberg limits, and organic contents were carried out on selected soil samples. An incrementally loaded consolidation test (ASTM D2435)<sup>4</sup> was carried out on one sample of the clay deposit in Borehole 21-08. The geotechnical laboratory testing was completed according to ASTM and MTO LS standards, as applicable. In addition, one soil sample was submitted to Bureau Veritas Laboratories in Sudbury, Ontario, an accredited analytical laboratory, for testing of a suite of corrosivity indicator parameters.

The as-drilled borehole locations were measured by a member of our technical staff referenced to the roadway centrelines or the existing culverts using a measuring tape and the locations were subsequently converted into northing and easting coordinates, as shown in the plan drawings (see Drawings 1 and 2). Given the relatively short distances between the boreholes and the existing roadway centerlines or existing culverts, the measurements are considered accurate to within 0.5 m horizontally. The ground surface at the borehole locations was surveyed by Golder, relative to the roadway centrelines and the geodetic elevations of the roadway centrelines were obtained from the survey drawing (Hwy 17-MR55.dwg) provided by AECOM. The NAD 83 MTM CSRS CBNv6-2010.0 (Zone 12) northing and easting coordinates, World Geodetic System 1984 (WGS 84) geographical coordinates, ground surface elevations referenced to Geodetic datum, and borehole depths at each borehole location are presented on the borehole records in Appendix A and summarized below.

Borehole Number	Location (MTM NAD 83 Zone 12)		Location (WGS 84)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting	Latitude	Longitude		
21-01	5137454.0	278240.7	46.376275	-81.345249	241.7	9.8
21-02	5137448.2	278224.5	46.376222	-81.345459	244.0	15.9
21-07	5137704.9	278285.0	46.378534	-81.344687	243.3	15.9
21-08	5137709.2	278297.9	46.378574	-81.344521	241.0	10.9

<sup>4</sup> ASTM D2435 Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

Based on the Northern Ontario Engineering Geology Terrain Study (NOEGTS)<sup>5</sup> mapping, the ground terrain within the vicinity of the Highway 17 and MR55 intersection is generally comprised of jagged, ruffed and cliffed bedrock knobs, outcrops and ridges with glaciolacustrine plain, alluvial plain, and organic terrain deposits in the lower lying wetland areas immediately adjacent the highway/roadway. The glaciolacustrine plain and alluvial plain deposits primarily consist of silts, sands and clays, and the organic terrain deposit primarily consists of peat/muck. The surface water drainage in the area varies from dry to wet, corresponding to areas of moderate to low relief.

Based on geological mapping by the Ministry of Natural Resources (Map 2543)<sup>6</sup>, the site is underlain by rocks of the Paleoproterozoic Era belonging to the Huronian Supergroup and Elliot Lake Group consisting of conglomerate, wacke, arkose, quartz arenite, argillite, limestone and dolostone. Areas of mafic and related intrusive rocks comprised of diabase sills, dykes and related granophyre are also present in the vicinity of the site. Based on geological mapping by the Ontario Department of Mines (Map 2170)<sup>7</sup>, this site area is characterized by extensive faults including the Murray Fault, which generally runs parallel to the Highway 17 alignment.

### 4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes and the results of the in situ and laboratory tests are presented in the Record of Boreholes in Appendix A. The detailed results of geotechnical laboratory testing are contained in Appendix B. The results of the in situ field tests (i.e., SPT 'N'-values and field vane undrained shear strengths), as presented on the Record of Borehole sheets and in Section 4.2, are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profiles in Drawings 1 and 2 are inferred from non-continuous sampling and therefore, represent transitions between soil types rather than exact planes of geological change.

The subsurface conditions will vary between and beyond the borehole locations; however, the factual data presented on the Record of Borehole sheets governs any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawings 1 and 2 is a simplification of the subsurface conditions.

#### 4.2.1 Culvert Extension – MR 55 STA 10+147

A detailed description of the soil deposits encountered in Boreholes 21-01 and 21-02 for the culvert on MR 55 at STA. 10+147 is provided below.

##### 4.2.1.1 Gravelly Sand to Silty Sand and Gravel - Fill

A 2.4 m thick layer of brown, moist, gravelly sand to silty sand and gravel (fill) was encountered from ground surface (Elevation 244.0 m) in Borehole 21-02, which was advanced from the existing roadway shoulder. Auger grinding on an obstruction was noted from a depth of 0.8 m to 2.4 m below ground surface.

SPT 'N' values within the gravelly sand to silty sand and gravel (fill) layer were 37 blows and 45 blows per 0.3 m of penetration indicating a dense compactness condition.

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<sup>5</sup>Northern Ontario Engineering Geology Terrain Study, Ontario Geological Society Digital Map Reference Number 41ISW.

<sup>6</sup> Ministry of Natural Resources, 1991. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey - Map 2543

<sup>7</sup> Ontario Department of Mines, 1969. Sudbury Mining Area, Sudbury District, Map 2170.



The water content measured on a sample of the silty sand and gravel fill was 5%.

The results of a grain size distribution test on a sample of the silty sand and gravel fill are presented in Figure B-1 in Appendix B.

#### **4.2.1.2 Peat**

A 50 mm thick layer of peat was encountered from the ground surface (Elevation of 241.7 m) in Borehole 21-01, which was advanced at the toe of the embankment slope near the existing culvert outlet.

#### **4.2.1.3 Sandy Silt to Sand**

A 2.1 m thick deposit of brown to grey, moist to wet, sandy silt, some clay, trace gravel to sand, trace to some silt, trace to some gravel was encountered below the fill in Borehole 21-02 and below the peat in Borehole 21-01. The surface of the sandy silt to sand deposit was encountered at Elevation 241.6 m. Auger grinding on an obstruction (potential cobble/boulder) was encountered from 3.0 m to 3.8 m depth below ground surface. This granular deposit is considered to be derived from fluvial processes and should be expected to contain interlayers of coarser grained and finer grained materials.

SPT 'N'-values within the sandy silt to sand deposit range from 3 blows to 27 blows per 0.3 m of penetration indicating very loose to compact compactness condition.

The water content measured on three samples of the sandy silt to sand deposit ranges from 12% to 16%.

The results of a grain size distribution test carried out on a sample of the sandy silt are presented on Figure B-2 in Appendix B.

#### **4.2.1.4 Organic Silt to Silt**

A 0.8 m to 1.4 m thick deposit of grey to dark grey, wet, sandy organic silt to silt, trace organics was encountered below the sandy silt to sand deposit in Boreholes 21-01 and 21-02. The surface of the organic silt to silt deposit was encountered at Elevation of 239.5 m.

SPT 'N' values within the organic silt to silt layer were 0 blows (i.e., weight of hammer) and 2 blows per 0.3 m of penetration, indicating a very loose compactness condition.

The water content measured on two samples of the organic silt to silt deposit were 41% to 57%.

The organic content on two samples of the organic silt to silt measured 5.0% and 4.9%.

An Atterberg limits test completed on a sample of the organic silt indicates the deposit is non-plastic.

#### **4.2.1.5 Clayey Silt to Silty Clay**

A 6.8 m and 7.8 m thick deposit of grey, wet clayey silt to silty clay was encountered below the organic silt to silt deposit in Boreholes 21-01 and 21-02 respectively. The surface of the clayey silt to silty clay deposit was encountered at Elevation 238.7 and 238.1 m in Boreholes 21-01 and 21-02, respectively, and the deposit was noted to be varved below about Elevation 237 m. The clayey silt to silty clay deposit extended to the borehole termination depth at Elevation 231.9 m in Borehole 21-01 and to Elevation 230.3 m in Borehole 21-02 where it was fully penetrated.

The SPT 'N'-values measured within the clayey silt to silty clay deposit range from 0 blows (i.e., weight of rods or weight of hammer) to 7 blows per 0.3 m of penetration suggesting a very soft to firm consistency.

In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 24 kPa to 53 kPa, indicating a firm to stiff consistency, but predominately firm. In one instance, the in situ field vane did not shear measuring an undrained shear strength >100 kPa at about Elevation 237.6 m in Borehole 21-01, which may be indicative of a non-cohesive seam/pocket or crust.

The water content measured on seven samples of the clayey silt to silty clay deposit range from 30% and 57%.

The results of a grain size distribution test completed on one sample of the clayey silt portion of the deposit is shown in Figure B-3 in Appendix B.

Atterberg limits tests carried out on five samples of the clayey silt to silty clay deposit returned liquid limits ranging from about 29% to 50%, plastic limits ranging from about 18% to 24%, and plasticity indices ranging from about 8% to 26%, indicating the deposit is a clayey silt of low plasticity to silty clay of intermediate plasticity. The Atterberg limits test results are shown in Figure B-4 in Appendix B.

#### **4.2.1.6 Sandy Silt to Silty Sand**

A 2.2 m thick deposit of grey, wet silty sand to sandy silt was encountered below the clayey silt to silty clay deposit in Borehole 21-02. The surface of the sandy silt to silty sand deposit was encountered at Elevation 230.3 m and Borehole 21-02 was terminated within this deposit at Elevation 228.1 m.

SPT 'N'-values measured within the sandy silt to silty sand deposit were 0 blows (i.e., weight of rod) and 6 blows per 0.3 m of penetration indicating very loose to loose compactness condition.

The water content measured on one sample of the silty sand to sandy silt deposit was 31%.

### **4.2.2 Culvert Extension – Hwy 17 EBL STA 14+563**

A detailed description of the soil deposits encountered in Boreholes 21-07 and 21-08 for the culvert extension in the Hwy 17 EBL at STA. 14+563 is provided below.

#### **4.2.2.1 Snow / Ice**

A 0.3 m thick layer of snow/ice was encountered at the time of drilling at Elevation 241.0 m in Borehole 21-08.

#### **4.2.2.2 Sand and Gravel to Gravelly Sand - FILL**

A 3.7 m and 0.7 m thick layer of brown, moist sand and gravel to gravelly sand (fill) was encountered from ground surface in Boreholes 21-07 (advanced from the existing shoulder), and below the snow/ice in Borehole 21-08 (which was advanced from the toe of the existing highway embankment). The surface of the fill deposit was encountered at Elevation 243.3 m and 240.7 m in Boreholes 21-07 and 21-08, respectively. An interlayer of sandy silt fill was encountered within the gravelly sand fill layer, as discussed in the next section. An obstruction was encountered at a depth of 2.7 m and is inferred to be a possible cobble/boulder.

SPT 'N'-values within the sand and gravel to gravelly sand (fill) range from 2 blows to 105 blows per 0.3 m of penetration indicating a very loose to very dense compactness condition.

The water content measured on two samples of the sand and gravel to gravelly sand (fill) layer were 5% and 9%.

#### 4.2.2.3 *Sandy Silt - FILL*

Interrupting the sand and gravel to gravelly sand fill in Borehole 21-07 was a 0.8 m thick layer of brown, moist sandy silt fill, some clay, some gravel. The sandy silt fill layer was encountered between Elevations 241.9 m and 241.1 m.

An SPT 'N'-value of 23 blows per 0.3 m of penetration was measured within the sandy silt fill layer indicating a compact compactness condition.

The water content measured on a sample of the sandy silt fill interlayer was 12%.

The results of a grain size distribution test completed on one sample of the sandy silt fill layer is shown in Figure B-5 in Appendix B.

#### 4.2.2.4 *Clayey Silt to Silty Clay*

An 8.4 and 9.9 m thick deposit of grey, moist to wet clayey silt to silty clay was encountered below the sand and gravel to gravelly sand (fill) in Boreholes 21-07 and 21-08, respectively. The surface of the clayey silt to silty clay deposit was encountered at Elevation 239.6 m and 240.0 m in Boreholes 21-07 and 21-08, respectively, and the deposit was noted to be varved below about Elevation 235 m. The clayey silt to silty clay deposit extended to Elevation 231.2 m in Borehole 21-07 and to the borehole termination depth at Elevation 230.1 m in Borehole 21-08.

SPT 'N'-values measured within the clayey silt to silty clay deposit range from 0 blows (i.e., weight of hammer) to 8 blows per 0.3 m of penetration suggesting a very soft to firm/stiff consistency. In situ field vane tests carried out within the deposit measured an undrained shear strength ranging from about 26 kPa to 82 kPa, indicating a firm to stiff consistency, but predominately firm.

Atterberg limits tests carried out on six samples of the clayey silt to silty clay deposit returned liquid limits ranging from about 32% to 44%, plastic limits ranging from about 19% to 21%, and plasticity indices ranging from about 11% to 23%, indicating the deposit is a clayey silt of low plasticity to silty clay of intermediate plasticity. The Atterberg limits test results are shown in Figure B-6 in Appendix B.

Two consolidation (oedometer) tests were carried out on selected specimens of the silty clay from a Shelby tube sample in Borehole 21-08. One of the tests was carried out in the horizontally trimmed orientation to evaluate the deformation parameters of the cohesive deposit and one test was completed in the vertically trimmed orientation to allow for the evaluation of horizontal drainage parameters. The preconsolidation stress was estimated from the void ratio versus logarithmic pressure plot for the horizontally trimmed test. The bulk unit weight measured from the specimens was 17.1 kN/m<sup>3</sup> and 17.2 kN/m<sup>3</sup>, with a measured specific gravity of 2.77. The detailed results of the oedometer test are shown in the separate foundation investigation report for the embankment widening and are summarized below.

Borehole / Sample No.	Sample Elevation (m)	w <sub>n</sub> (%)	γ (kN/m <sup>3</sup> )	σ <sub>vo</sub> ' (kPa)	σ <sub>p</sub> ' (kPa)	OCR	e <sub>o</sub>	C <sub>c</sub>	C <sub>r</sub>	C <sub>v</sub> (cm <sup>2</sup> /s)
21-08 / 8A	232.3	53.5	17.2	70	150	2.1	1.4	0.66	0.046	0.006
21-08 / 8B	232.0	54.4	17.1	70	N/A	N/A	N/A	N/A	N/A	N/A

Notes: Parameters presented calculated within the operative stress range for this project.

Where: w<sub>n</sub> Natural Moisture content (%)  
 γ Unit weight (kN/m<sup>3</sup>)  
 σ<sub>vo</sub>' Effective overburden pressure (kPa)  
 σ<sub>p</sub>' Preconsolidation pressure (kPa)  
 OCR Overconsolidation Ratio  
 e<sub>o</sub> Initial void ratio  
 C<sub>c</sub> Compression index  
 C<sub>r</sub> Recompression index  
 C<sub>v</sub> Coefficient of consolidation in the normally consolidated range (cm<sup>2</sup>/s)

#### 4.2.2.5 Silt

A 2.7 m thick deposit of grey, wet silt, trace clay, trace sand was encountered below the clayey silt to silty clay deposit in Borehole 21-07. The surface of the silt deposit was encountered at Elevation 231.2 m.

SPT 'N'-values measured within the silt deposit were 1 blow and 4 blows per 0.3 m of penetration indicating a very loose to loose compactness condition.

The water content measured on one sample of the silt deposit was 31%.

The results of a grain size distribution test completed on one sample of the silt deposit is shown in Figure B-7 in Appendix B.

An Atterberg limits test completed on one sample of the silt indicates the deposit is non-plastic.

#### 4.2.2.6 Sand

A 1.1 m thick deposit of grey, wet sand, some silt was encountered below the silt deposit in Borehole 21-07. The surface of the sand deposit was encountered at Elevation 228.5 m and Borehole 21-07 was terminated in the sand deposit at Elevation 227.4 m.

An SPT 'N'-value measured within the sand deposit was 3 blows per 0.3 m of penetration indicating a very loose compactness condition.

The water content measured on a sample of the sand deposit was 23%.

### 4.3 Groundwater Conditions

The unstabilized groundwater levels were measured in the open boreholes upon completion of drilling. In addition, a more stabilized groundwater level was measured within the temporary piezometer installed within Borehole 21-01. The observed groundwater conditions near each culvert location are summarized below.

Culvert Location	Borehole No.	Depth to Groundwater Level (m)	Approximate Groundwater Elevation (m)	Notes
MR55 STA.10+147	21-01	Dry	-	open borehole (Feb. 1, 2021)
		1.4	240.3	piezometer (Feb. 3, 2021)
		0.1	241.6	piezometer (Feb. 8, 2021)
		0.1	241.6	piezometer (Feb. 9, 2021)
	21-02	9.0	235.0	open borehole (Feb. 1, 2021)
Hwy 17 EBL STA. 14+563	21-07	6.8	236.5	open borehole (Feb. 2, 2021)
	21-08	2.8	238.2	open borehole (Feb. 8, 2021)

The existing pipe culverts both drain into the nearby Fairbank Creek, which crosses the Highway 17 EBL at about STA. 14+384 and flows towards the southeast. The water level in Fairbanks Creek was measured by others to be at Elevation 240.9 m adjacent to Highway 17 on January 7, 2021. The water level at the outlet of the existing MR55 culvert was observed to be below the invert elevation (i.e., about Elev. 241.0 m) during a site visit in June 2021. The water level at the outlet of the existing Hwy 17 EBL culvert was observed to be above the obvert (i.e., about Elevation 240.9 m) during a site visit in June 2021 and as shown in Photograph 10. Groundwater and creek water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

#### 4.4 Analytical Laboratory Testing Results

A sample of the native subgrade soils at each culvert site near the approximate culvert invert elevation was submitted to Bureau Veritas Laboratories, an accredited analytical testing laboratory. The results of the analytical testing are detailed in the laboratory testing report (Certificate of Analysis) included in Appendix C and are summarized below.

Borehole No.	Sample No.	Elevation (m)	Parameters					
			Resistivity (ohm-cm)	Conductivity (µmho/cm)	Sulphate (SO <sub>4</sub> ) Content (µg/g)	Chloride (Cl) Content (µg/g)	Sulphide (mg/kg)	pH
21-02	5	240.6	580	1,720	49	980	<0.5 <sup>(1)</sup>	6.65
21-07	6	239.2	1600	639	25	410	<0.5 <sup>(1)</sup>	6.27

(1) The sulphide concentrations are below the reportable detection limit of 0.5 mg/kg.

#### 5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Tibor Berecz, EIT, under the overall direction of Mr. Matthew Thibeault, P.Eng. This report was prepared by Mr. David Muldowney, P.Eng., and Mr. Kevin Bentley, P.Eng., an MTO Foundations Designated Contact and Associate with Golder, conducted an independent quality control review of this report.

## Signature Page

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**PART B**

FOUNDATION DESIGN REPORT

PIPE CULVERT EXTENSIONS

MUNICIPAL ROAD 55 – STATION 10+147 DENISON TOWNSHIP

HIGHWAY 17 EBL – STATION 14+563 DENISON TOWNSHIP

DISTRICT OF SUDBURY

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5032-19-00

## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides Foundation design recommendations for two plastic pipe culvert extensions associated with the Highway 17 and MR55 intersection improvements / widening. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface exploration. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the culvert extensions. The Foundation Investigation and Design Report, including the discussion and recommendations are intended for the use of the MTO and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

### 6.1 General

#### 6.1.1 Culvert Extension – MR55 STA 10+147

The existing plastic pipe culvert along MR55 is located at about STA 10+147 in the Denison Township, in the District of Sudbury, Ontario (i.e., about 123 m south of the Highway 17 intersection; latitude 46.376178, longitude -81.345475). The existing culvert consists of a 900 mm diameter pipe culvert with inverts at about Elev. 241.5 m and 241.0 m at the west (inlet) and east (outlet) ends, respectively.

We understand that an approximately 3 m long culvert extension may be required at the east (outlet) end to accommodate a new Hwy 17 / MR55 south-east (S-E) ramp; however, the actual length of the extension will be confirmed once the final configuration of the intersection improvements, including the extent of the MR55 widening, has been finalized.

The proposed extension could consist of a concrete box culvert, open footing culvert, or pipe culvert; however, considering the additional / complex tie-in details for a box or open footing culvert, a similar sized pipe culvert extension compatible with the existing culvert is considered to be the most practical option. A pipe culvert is also preferred from a foundations perspective given the relative ease of construction compared to an open footing culvert or box structure that would require additional excavation and more elaborate dewatering and/or shoring support.

Based on our discussions with AECOM, we understand that that given the relatively small diameter of the existing pipe culvert, a similarly sized pipe culvert extension is preferred.

#### 6.1.2 Culvert Extension – Hwy 17 EBL STA 14+565

The existing plastic pipe culvert is located along the Highway 17 EBL at about STA 14+565 in the Denison Township, in the District of Sudbury, Ontario (i.e., about 265 m east of the MR 55 intersection; latitude 46.378611, longitude -81.344722). The existing culvert consists of a 900 mm diameter pipe culvert with inverts at about Elev. 240.0 m and 239.9 m at the north (inlet) and south (outlet) ends, respectively.

We understand that an approximately 3.5 m to 5 m long culvert extension may be required to accommodate the 3.5 m to 5 m embankment widening for the new eastbound acceleration lane; however, the actual length of the extension / width of embankment widening will be confirmed once the final configuration of the Highway 17 / MR55 interchange has been determined.

The proposed extension could consist of a concrete box culvert, open footing culvert, or pipe culvert; however, considering the additional / complex tie-in details for a box or open footing culvert, a similar sized pipe culvert extension compatible with the existing culvert is considered to be the most practical option. A pipe culvert is also preferred from a foundations perspective given the relative ease of construction compared to an open footing culvert or box structure that would require additional excavation and more elaborate dewatering and/or shoring support.

Based on our discussions with AECOM, we understand that that given the relatively small diameter of the existing pipe culvert, a similarly sized pipe culvert extension is preferred.

### 6.1.3 Consequence and Site Understanding Classification

As Highway 17 carries a relatively large volume of traffic, with a projected average annual daily traffic (AADT) of about 9,000 vehicles per day for the year 2024 (as outlined in the Planning, Preliminary Design, and Environmental Report by Stantec dated August 2008), and has the potential to impact alternative transportation corridors, a “typical consequence level” is considered appropriate for the foundation design at this site, as outlined in Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2019) and its Commentary. The traffic volumes along MR55 are significantly lower (AADT of about 1,450 vehicles per day) and has limited impact to alternative transportation corridors, thus, consideration could be given the utilizing a “low consequence level” compared to Highway 17. However, for the purpose of this report, a “typical consequence level” has been adopted for design on MR55.

Further, given the scope of work of the foundation field investigation and laboratory testing program, as presented in Sections 3.0 and 4.0, a “typical degree of site and prediction model understanding” has been utilized. Accordingly, the appropriate corresponding ULS and SLS consequence factor,  $\Psi$ , and geotechnical resistance factors,  $\Phi_{gu}$  and  $\Phi_{gs}$ , from Tables 6.1 and 6.2 of the CHBDC have been used for design.

## 6.2 Embankment Stability at Culvert Locations

Based on our site observations at the time of the field investigation and a review of the available satellite images, the existing highway/roadway embankments in the culvert areas appear to be performing satisfactorily with no visual evidence of instability (i.e., soil movement) and no tension cracks near the embankment crest that would be indicative of instability.

The following sections outline the methodology used to interpret geotechnical parameters for the foundation soils and evaluate embankment stability at the culvert locations.

### 6.2.1 Methodology

Limit equilibrium slope stability analyses were carried out to assess the Factor of Safety (FoS) of the existing embankment side slopes at the culvert locations as well as the proposed widened embankments. The slope stability analyses were performed using the commercially available program GeoStudio 2021 (Version 11.0.1.21429), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For the analyses, the FoS of numerous potential surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure.

For the purpose of the stability analyses, and in the context of the CHBDC (2019), the target FoS is defined as being equal to the inverse of the product of the consequence factor,  $\Psi$ , and the geotechnical resistance factor,  $\Phi_{gu}$  (i.e.,  $FoS = 1 / (\Psi * \Phi_{gu})$ ). Accordingly, for a “typical consequence level” and “typical degree of site and prediction model understanding”, a target minimum FoS of 1.33 and 1.54 has been used for the design of the embankment slopes, considering global stability for temporary (short-term) and permanent (long-term) conditions, respectively.

The stability analyses were carried out using the cross-section embankment geometries at the culvert locations (i.e., MR55 at STA 10+147 and Hwy 17 EBL at STA 14+563) taken from the base plan survey drawings provided by AECOM and using the idealized soil stratigraphy shown in Drawings 1 and 2 (as appropriate). Due to the transient nature of traffic loading, traffic loads have not been included in the slope stability analyses.

For the analyses, it assumed the groundwater level is located at the creek water level which is estimated to be about Elevation 241.6 m at the MR55 culvert site and about Elevation 240.9 m at the Hwy 17 culvert site.

### 6.2.2 Parameter Selection

For the non-cohesive granular embankment fill materials and native deposits of sands and silts, effective stress parameters were employed for analysing both the short-term (undrained) and the long-term (drained) conditions. The effective stress parameters (i.e.,  $c'$  and  $\phi'$ ) were estimated from empirical correlations based on the in situ SPT 'N'-values. The correlations proposed in literature (i.e., NavFac Design Manual 7.02 (1986) and Bowles (1997)) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive native clayey silt to silty clay soils, total stress parameters were employed for analysing the short-term, undrained condition. The total stress parameters (i.e., average mobilized undrained shear strength,  $s_u$ ) for the cohesive soils are based on the results of the in situ field vane test data and the consolidation test and correlated with the SPT results and laboratory test data.

For the long-term, drained condition, effective stress parameters (i.e.,  $c'$  and  $\phi'$ ) were assigned to the cohesive soils based on empirical correlations with index properties and engineering judgement.

Summarized below are the simplified stratigraphy and the associated soil parameters employed for the stability analyses at the two culvert locations.

**Soil Parameters for Stability Analyses at Culvert Extension - MR55 STA 10+147**

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Short-Term Condition		Long-Term Condition	
		Effective Friction Angle (°)	Undrained Shear Strength (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
New Granular Fill (compacted)	21	35	-	-	35
Existing Granular Fill (dense)	20	35	-	-	35
Sandy Silt to Sand (very loose to compact)	19	29	-	-	29
Organic Silt to Silt (very loose)	18	28	-	-	28
Clayey Silt to Silty Clay above Elev. 237 m (stiff)	18	-	55	0	31
Clayey Silt to Silty Clay Elev. 237 m to 236 m (stiff to firm)	18	-	55-30 <sup>1</sup>	0	29
Clayey Silt to Silty Clay - varved Below Elev. 236 m (firm)	18	-	30 <sup>1</sup>	0	29
Sandy Silt to Silty Sand (very loose to loose)	19	29	-	-	29

(1) corrected values for  $\mu_{(avg)} = 0.85$  to account for varves and plasticity.

**Soil Parameters for Stability Analyses for Culvert Extension - Hwy 17 EBL STA 14+563**

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Short-Term Condition		Long-Term Condition	
		Effective Friction Angle (°)	Undrained Shear Strength (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
New Granular Fill (compacted)	21	35	-	-	35
Existing Granular Fill (compact to very dense)	20	35	-	-	35
Clayey Silt to Silty Clay above Elev. 237 m (stiff)	18	-	55	-	31
Clayey Silt to Silty Clay - varved Elev. 237 – 236 m (stiff to firm)	18	-	55 - 30 <sup>1</sup>	-	29
Clayey Silt to Silty Clay - varved below Elev. 236 m (firm)	18	-	30 <sup>1</sup>	-	29
Silt (very loose to loose)	18	28	-	-	28
Sand (very loose)	19	30	-	-	30

(1) corrected values for  $\mu_{(avg)} = 0.85$  to account for varves and plasticity.

In general, the lower portion of the cohesive deposit (i.e., below Elev. 235 m to 237 m) is considered to be varved; therefore, an anisotropic correction factor has been applied to the measured cross-varve strength properties (i.e., undrained shear strength measured from in situ vane testing and correlated effective friction angle) of the silty clay deposit below about Elevation 237 m to account for a lower mobilized strength that would result from horizontal shearing along the weak clay laminae.

Different correction factors have been reported in literature but vary depending on the nature of the laminated clay soil (i.e., structure as well as thickness and plasticity of the laminae). Based on our experience and judgement, for this site, a 0.85 correction factor ( $\mu_{(avg)}$ ), resulting in a 15% reduction applied to the strength properties is considered appropriate for a varved clayey silt to silty clay stratum encountered below Elevation 237 m based on an upper bound plasticity index of about 25%. More information on selection of the correction factors and referenced literature is provided in the separate Highway 17 Widening Foundation Investigation and Design Report included as part of this project.

For the upper portion of the deposit, where varving was not observed (i.e., above about Elevation 237) a lesser correction based on Bjerrum (1975) was applied to this portion of the deposit to calculate the mobilized shear strength in accordance with the recommendations provided in (ASTM D2573).



### 6.2.3 Results of Analyses

#### **Culvert Extension – MR 55 STA 10+147**

Based on the results of the analyses, the critical slip surface extends through the non-cohesive embankment fill and into the native sandy silt to sand deposit. As such, the strength parameters for both the short-term (undrained) and long-term (drained) cases are identical and the long-term scenario with the higher target minimum FoS will govern for embankment design at this culvert location.

The existing west embankment side slope, which is inclined at about 2.9H:1V, has a FoS of 1.79 and the existing east embankment side slope, which is slightly lower in height and inclined at about 2.7H:1V, has a FoS of 1.91. As such, the existing west and east embankments both satisfy the minimum target FoS requirement.

Based on discussions with AECOM, we understand that an approximately 3 m widening of the east embankment will be required to accommodate the proposed S-E ramp and a 2H:1V side slope will be utilized for the proposed widening. For the approximately 3 m widened east embankment (assumed to consist of granular fill) inclined at 2H:1V, the calculated FoS against global stability is 1.84 in both the short-term (undrained) and long-term (drained) conditions (see Figure 1), which satisfies the minimum target FoS requirement of 1.54 for a “typical consequence level” and “typical degree of site and prediction model understanding”. As such, slope stability mitigation measures are not required at this site.

#### **Culvert Extension – Hwy 17 EBL STA 14+563**

Based on the results of the analyses, the existing north embankment side slope, which is inclined at about 3H:1V, has a FoS of 3.79 and 2.0 in the short-term (undrained) and long-term (drained) conditions, respectively, which satisfies the minimum target FoS requirements. Similarly, the existing south embankment side slope, which is inclined at about 2H:1V, has a FoS of 4.16 and 1.85 in the undrained and drained conditions, respectively, which also satisfies the minimum target FoS requirements.

Based on discussions with AECOM, we understand that an approximately 3.5 m to 5.0 m widening of the south embankment will be required to accommodate the proposed eastbound acceleration lane and a 2H:1V (or possibly 3H:1V) side slope will be utilized for the proposed widening. For the proposed approximately 5 m widening of the south embankment inclined at 2H:1V (or flatter), the calculated FoS against global stability is 3.83 in the short-term (undrained) condition and 1.80 in the long-term (drained) condition as shown in Figures 2 and 3, respectively, which satisfies the minimum target FoS requirements for a “typical consequence level” and “typical degree of site and prediction model understanding”. As such, slope stability mitigation measures are not required for the embankments.

### 6.3 Embankment and/or Culvert Settlement

Based on discussions with AECOM, we understand that an approximately 3 m widening may be required along the east side of MR55 in the vicinity of the existing pipe culvert at STA. 10+147 to accommodate a new S-E ramp. Similarly, an approximately 3.5 m to 5 m widening is being proposed along the south side of the Highway 17 EBL in the vicinity of the pipe culvert at STA. 14+563 to accommodate a new acceleration lane.

The embankment widenings will introduce additional loads to the compressible foundation soils that will cause settlement adjacent to and below the proposed culvert extensions. The following sections outline the methodology and results of the settlement assessment carried out at the proposed culvert extensions.

### 6.3.1 Methodology

To estimate the magnitude of the settlement as a result of the proposed embankment widenings, analyses were carried out using the commercially available computer program *Settle-3* (Version 5.010) from Rocscience Inc. The settlement analysis assumes that the organics (i.e., topsoil, peat and mixed organic soils containing excessive organics) have been removed and replaced in accordance with OPSD 203.020 (Embankments Over Swamp) prior to construction of the embankment widenings.

The sources of settlement are considered to include:

- immediate settlement of the native granular soils;
- primary time dependent consolidation of the cohesive deposits (using Terzaghi's one dimensional consolidation theory); and,
- secondary time dependent (creep) compression of the cohesive deposits (long term)

As noted above, in addition to primary consolidation being evaluated using Terzaghi's one dimensional consolidation theory within the cohesive deposits (i.e., clayey silt to silty clay), secondary compression was also assessed. The following relationship has been employed for estimating the magnitude of creep settlement following the completion of primary settlement at each location:

$$S_c = HC_{\alpha\epsilon} \log \left( \frac{t}{t_{EOP}} \right)$$

where:  $S_c$  = secondary compression (creep) settlement (mm)

$C_{\alpha\epsilon}$  = modified secondary compression index

$H$  = initial thickness of compressible clay deposit (mm)

$t$  = post construction period of interest (20 years)

$t_{EOP}$  = time to reach end of primary consolidation (years)

Based on experience from other sites in Northeastern Region, the secondary compression was applied to the settlement models after a degree of primary consolidation (U) of 90% was achieved.

### 6.3.2 Parameter Selection

The simplified stratigraphy together with the associated deformation and time rate consolidation parameters employed for the different native soil types were evaluated based on in situ field testing, laboratory testing and engineering judgement. A summary of the foundation engineering parameters employed in the settlement models for the cohesive deposits at both culvert locations is presented on Figure 4. Figure 4 includes all data and laboratory testing from the surrounding boreholes advanced as part of the overall foundation investigation for the Hwy 17 / MR55 intersection improvement and widening project (including previous investigations in the area) and is included in the separate Hwy 17 Embankment Widening Foundation Investigation and Design Report (FIDR). The reference data / information used to develop Figure 4 are included in the Hwy 17 Embankment Widening FIDR.

The immediate compression of the cohesionless deposits (i.e., silt, sand or gravel) were modelled by estimating an elastic modulus of deformation from engineering judgement based on similar soils in Northeastern Ontario.

The primary consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests and, where appropriate, in situ field vane tests to estimate the deformation parameters. In addition, for the clayey soils the results of the laboratory index tests were also employed to further assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Koppula (1981) and Azzouz et al. (1976), as well as an empirical correlation developed from research performed by Golder for the MTO in Northeastern Region (Geocres No. 32D-35). The literature correlations were compared to the results of the consolidation testing for this site (by elevation and void ratio) to select an appropriate site-specific correlation for use in the selection of design lines.

For clayey soils, the following correlation relating in situ mobilized undrained shear strength to preconsolidation pressure (Mesri, 1975) was employed:

$$\sigma_p' = \tau_{mobilized}/0.22$$

where:  $\sigma_p'$  = preconsolidation pressure (kPa)

$$\tau_{mobilized} = \mu_v(S_u)_{FV}$$

where:  $\tau_{mobilized}$  = the mobilized shear strength ( $S_{u(mob)}$ ) for geotechnical analysis

$\mu_v$  = an empirical correction factor that has been related to plasticity index (PI) and/or liquid limit ( $w_L$ ) and/or other parameters based on back calculation from failure case history records of full-scale projects.

The coefficient of consolidation,  $c_v$  (cm<sup>2</sup>/s), required in the time rate settlement analysis, was established for the site using the combined results of the laboratory consolidation tests and the estimated  $c_v$  values based on the Unified Facilities Criteria (U.S. Navy, NAVFAC 1986) correlation with liquid limit for normally consolidated and overconsolidated soils.

The secondary compression index was evaluated using a method proposed by Mesri et al. (1994), which indicates that for every soil, a constant value of the ratio of  $C_{\alpha}/C_c$  holds at all combinations of consolidation pressure and time. Specifically, for this site a ratio of  $C_{\alpha(e)}/C_c = 0.04 \pm 0.01$  for inorganic soils (Mesri et al. 1994).

### 6.3.3 Results of Analyses

Based on the results of the analyses, the anticipated settlements associated with the proposed widenings will be less than about 25 mm at the specific culvert extension locations. Given the relatively minor estimated magnitude of post-construction settlement that will occur under the proposed embankment widenings at the culvert extension locations, settlement mitigation measures are not anticipated to be required.

## 6.4 Culvert Foundation Design Recommendations

### 6.4.1 Founding Level and Geotechnical Resistance

Prior to placing the bedding for the new culvert extensions, it is recommended that all organic material (i.e., topsoil, peat, and/or mixed soils containing excessive organics) and existing fill encountered below the footprint of the culvert extension be sub-excavated and replaced with Ontario Provincial Standard Specification, Provincial Oriented (OPSS.PROV) 1010 Granular 'A or Granular 'B' Type II material, as amended by Standard Special Provision (SSP) 110S06. Given the fine-grained nature of the founding soils at this site and the potential for subgrade disturbance/softening, sub-aqueous fill placement is not recommended for the culvert extension on Hwy 17 at STA 14+563.

Based on the proposed invert elevations provided in Section 6.1 and taking into account the 300 mm thick bedding layer (as discussed in Section 6.5.4), the culvert bedding for the pipe culvert extensions will be founded within the loose sand deposit for the culvert extension on MR55 at STA 10+147 and the stiff clayey silt deposit for the culvert extension on the Hwy 17 EBL at STA 14+563, both of which are considered suitable for supporting a pipe culvert extension provided the subgrade is properly prepared and the bedding/embedment material is properly placed and compacted.

#### 6.4.2 Frost Protection

The estimated frost penetration depth in the vicinity of the Highway 17 and MR55 intersection is 2.1 m as interpreted from OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario). As the culvert extensions are to be founded below the estimated 2.1 m depth of frost penetration for this site within the roadway, and the recommended backfill materials (including the existing granular embankment fill) are classified as having a low susceptibility to frost heaving (as per the MTO Pavement Design and Rehabilitation Manual), a frost taper is not required as per OPSD 803.030 (Frost Treatment – Pipe Culverts). In this regard, we do not recommend re-using the existing sandy silt embankment fill encountered at the culvert site in the Highway 17 EBL at STA. 14+563 (see Borehole 21-07).

Where the risk of differential heaving at culvert ends is high, which is not the case at these culvert sites, consideration can be given to sub-excavating and replacing the frost susceptible soils with non-frost susceptible fill materials (i.e., OPSS.PROV 1010 Granular 'A' or Granular 'B' Type II) and/or incorporating polystyrene insulation into the design. However, these measures are typically not considered to be practical or cost effective, and if a flexible pipe culvert extension is selected, generally not necessary as flexible culverts are typically tolerant to freeze-thaw movements. As such, measures to mitigate the risk of differential heaving occurring at the culvert ends at these sites are not considered necessary.

### 6.5 Construction Considerations

#### 6.5.1 Temporary Excavations / Support Systems

All excavations must be carried out in accordance with Ontario Regulation 213, Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended.

Based on the encountered subsurface conditions and the limited excavation extents/depths that are anticipated to be required to facilitate the culvert extension installations, temporary open cut excavations are considered feasible at both sites. Excavations for the culvert extension are anticipated to extend through a portion of the existing granular embankment fill materials (i.e., within the existing shoulder) and/or the upper portion of the native soil deposits. The granular fill and native soils within the anticipated excavation depths can be classified as Type 3 soil above the groundwater table and Type 4 soil below the groundwater table as per the OHSA. Temporary open-cut excavations in Type 3 and Type 4 soils can be sloped no steeper than 1H:1V and 3H:1V, respectively.

Temporary shoring systems, if required, could consist of sheet piles and/or soldier piles and lagging and could be incorporated into a cofferdam enclosure for dewatering purposes (as discussed in the next section). Consideration should be given to the potential for cobble and/or boulder obstructions, as inferred to be present within the existing embankment fill materials and/or native soils. Horizontal support to the system could be in the form of struts, walers, rakers, or anchors if a cantilevered system is not sufficient. Temporary protection/dewatering systems (if utilized) are the responsibility of the Contractor and should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems), as amended by SSP 105S09. Temporary protection systems should be designed to Performance Level 2 for any excavation adjacent to an existing roadway. Design of the temporary support system should include an evaluation of base stability, soil squeezing stability, and hydraulic uplift stability, as defined in the Canadian Foundation Engineering Manual (CFEM 2006).

Consideration could be given to either partial or full removal of the temporary protection system(s) upon completion of construction, as noted in OPSS.PROV 539. As noted above, there is a risk that the installation and/or subsequent removal of the temporary protection system(s) could result in subgrade disturbance/softening of the clayey silt portions of the cohesive deposits at these sites depending on type of system and installation methodology utilized. There is also a risk of soil adhesion along the piles (CFEM 2006), which could create a void in the subsoil after removal. From our perspective, the risk is considered relatively low at this site and, as such, partial and full removal options are both considered to be feasible. The Contractor will need to evaluate these risks based on the type of system and installation methodology ultimately adopted as part of their temporary protection system design. Further, the Contractor will need to re-evaluate these risks prior to removing the temporary protection system based on site observations during installation of the temporary protection system related to subgrade, culvert, and embankment performance.

Although the design of the temporary protection and/or dewatering (i.e., cofferdam) system(s), if required, will be carried out by the Contractor, the following soil parameters are provided to enable the structural designer to develop a conceptual design and assess the approximate construction costs for the project system, if adopted at this site.

### **Culvert Extension – MR 55 STA 10+147**

Fill / Soil Type	Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	Effective Stress Parameters <sup>(1)</sup>				Total Stress Parameters <sup>(1)</sup>	
		Internal Angle of Friction, $\phi$ (degrees)	Lateral Earth Pressure Coefficients <sup>(2)</sup>			Undrained Shear Strength, $s_u$ (kPa)	Lateral Earth Pressure Coefficients <sup>(2)</sup> $K_p=K_o=K_a$
			Active, $K_a$	At Rest, $K_o$	Passive, $K_p$ <sup>(3)</sup>		
New Granular Fill (compacted)	21	35	0.27	0.43	3.69	-	-
Existing Granular Fill (dense)	20	35	0.27	0.43	3.69	-	-
Sandy Silt to Sand (very loose to compact)	19	29	0.35	0.52	2.88	-	-
Organic Silt to Silt	18	28	0.36	0.53	2.77	-	-
Clayey Silt to Silty Clay above Elev. 237m (stiff)	18	31	0.32	0.48	3.12	50	1.0
Clayey Silt to Silty Clay Elev 237-236 (stiff to firm)	18	29	0.35	0.52	2.88	43 (average)	1.0
Clayey Silt to Silty Clay below Elev 236 m (firm)	18	29	0.35	0.52	2.88	30	1.0
Sandy Silt to Silty Sand (very loose to loose)	19	29	0.36	0.53	2.77	-	-

## Notes:

- (1) The temporary shoring design should be assessed for both the effective stress, drained ( $\phi'$ ) and total stress, undrained ( $s_u$ ) cases and the design should be based on the more conservative earth pressure conditions.
- (2) The lateral earth pressure coefficients presented above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected, the coefficients should be corrected accordingly.
- (3) The total passive resistance below the base of the excavation adjacent to the temporary protection system may be calculated based on the values of  $K_p$  indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.27 of the CHBDC (2019) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

**Culvert Extension – Hwy 17 EBL STA 14+563**

Fill / Soil Type	Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	Effective Stress Parameters <sup>(1)</sup>				Total Stress Parameters <sup>(1)</sup>	
		Internal Angle of Friction, $\phi$ (degrees)	Lateral Earth Pressure Coefficients <sup>(2)</sup>			Undrained Shear Strength, $s_u$ (kPa)	Lateral Earth Pressure Coefficients <sup>(2)</sup> $K_p=K_o=K_a$
			Active, $K_a$	At Rest, $K_o$	Passive, $K_p$ <sup>(3)</sup>		
New Granular 'A' Fill (compacted)	22	35	0.27	0.43	3.69	-	-
New Granular 'B' Type I or II Fill (compacted)	21	35	0.27	0.43	3.69	-	-
Existing Granular Fill (dense)	20	35	0.27	0.43	3.69	-	-
Clayey Silt to Silty Clay above Elev. 237 m (stiff)	18	31	0.32	0.49	3.12	55	1.0
Clayey Silt to Silty Clay Elev 237-236 (stiff to firm)	18	29	0.35	0.52	2.88	43 (average)	1.0
Clayey Silt to Silty Clay below Elev 236 m (firm)	18	29	0.35	0.52	2.88	30	1.0
Silt (very loose to loose)	18	28	0.36	0.53	2.77	-	-
Sand (very loose)	19	30	0.33	0.50	3.00	-	-

## Notes:

- (1) The temporary shoring design should be assessed for both the effective stress, drained ( $\phi'$ ) and total stress, undrained ( $s_u$ ) cases and the design should be based on the more conservative earth pressure conditions.
- (2) The lateral earth pressure coefficients presented above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected, the coefficients should be corrected accordingly.
- (3) The total passive resistance below the base of the excavation adjacent to the temporary protection system may be calculated based on the values of  $K_p$  indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.27 of the CHBDC (2019) to account for the fact that a large strain would be required for mobilization of the full passive resistance.



### 6.5.2 Control of Groundwater and Surface Water

At both culvert locations, temporary excavations to reach the design founding level (i.e., bottom of culvert bedding) will extend below the watercourse (i.e., creek/stream water level) and ditch invert level, and groundwater flow / seepage into the excavation should be expected. Therefore, control of the surface water and groundwater will be required to facilitate the culvert extensions as the bedding and culvert placement is recommended to be carried out in-the-dry.

Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade and to allow for placement and compaction of bedding / embedment soils. Depending on the water level / groundwater level at the time of construction, cofferdams (e.g., sheet pile box) may be required and are considered feasible at both culvert sites. Consideration should be given to the potential for cobble and/or boulder obstructions, as inferred to be present within the existing embankment fill materials and/or native soils. Provided the cofferdam is installed with adequate penetration / depth into the native soils, water pumping volumes within the excavation are anticipated to be manageable. Depending on the water flows at the time of construction, the water could potentially be pumped from behind the cofferdam or be diverted through a temporary diversion pipe/channel.

The culvert in the Highway 17 EBL provides cross-drainage from the highway median towards the ditch on the south side of the highway. Given the limited anticipated surface water flows and the relatively low permeability of the silty clay to clayey silt subgrade at this culvert location, a sandbag or inflatable cofferdam system is considered more practical from a cost perspective and from a constructability perspective as the sandbag or inflatable cofferdam system would have less potential impacts associated with subgrade disturbance during construction.

For the culvert extension on MR55 at STA. 10+147, a sandbag or inflatable cofferdam may be not feasible given the presence of the relatively permeable sandy silt to sand foundation soils and the higher anticipated surface water flows from the watercourse (creek/stream). At this site, a sheet pile cofferdam is preferred from a foundation perspective and may need to penetrate below the sandy silt to sand and silt deposits to limit groundwater seepage into the excavation.

Unwatering / dewatering of all excavations should be carried out in accordance with OPSS.PROV 517 (Dewatering), as modified by SSP 517F01, a copy of which is included in Appendix D. Given the general lack of infrastructure in the vicinity of the culvert, the conceptual dewatering strategy (i.e., pumping behind/within cofferdam systems that provide adequate groundwater cutoff) and limited depth of excavation, a preconstruction survey is not anticipated to be required and, as such, the designer fill-in in Table A of SSP 517F01 should indicate that a preconstruction survey is not applicable (N/A). The requirements for a preconstruction survey distance should, however, be reviewed/confirmed by the Hydrology and Drainage Engineer(s) and/or by a professional hydrogeologist. Further, referencing the fill-ins for SSP 517F01, the Designer Option requiring that the dewatering system design engineer and design-checking engineer have a minimum five-years experience designing similar systems is not considered to be required for these sites. The remaining fill-in information related to the minimum design storm return period and return period flow estimates should be input by AECOM's Hydrology and Drainage Engineer(s).

An Environmental Activity Section Registry (EASR) may not be required to temporarily pump surface water flows from behind a cut-off wall or cofferdam system, provided the water is returned back to the same watercourse and the prescribed discharge requirements are met. However, an EASR will be required to unwater/dewater the excavation area if pumping volumes are anticipated to be greater than 50 m<sup>3</sup>/day and a Permit to Take Water (PTTW) will be required if pumping volumes are anticipated to be greater than 400m<sup>3</sup>/day. Based on the soil conditions at this site and the anticipated culvert invert elevation, pumping volumes to unwater/dewater the excavation areas are anticipated to be less than 50 m<sup>3</sup>/day if an appropriate cofferdam system with sufficient embedment into the underlying cohesive deposit is utilized. The Contractor will need to evaluate the estimated seepage and groundwater removal quantity, based on their proposed construction methods/procedures and the groundwater conditions at the time of construction, to make the final assessment/determination whether an EASR or PTTW is ultimately required.

### 6.5.3 Subgrade Preparation

Prior to placing the levelling pad/bedding layer and culvert extension, all existing fill, exposed organic materials (including topsoil, peat, and/or mixed organic soil with excessive organics), and any disturbed/softened native soils should be sub-excavated from below the plan limits of the proposed works to expose the undisturbed native subgrade soil within the plan limits of the culvert extensions. Note that sub-excavation of the organic silt to silt deposit encountered at 2.2 m below ground surface (below the silty sand to sand deposit) in Borehole 21-01 near the toe of the existing embankment slope on MR55, which contains trace organics (i.e., 4.9%) is not considered necessary in advance of the culvert extension.

The subgrade shall be inspected following sub-excavation, to ensure that all organics (if encountered) and other unsuitable materials have been removed, in accordance OPSS.PROV 421 (Pipe Installation in Open Cut), as amended by SSP104S06 if necessary. Following inspection and approval of the exposed subgrade, any additional fill material required to raise the grade up to the underside of the proposed bedding layer shall consist of granular material meeting the requirements of an OPSS.PROV 1010 Granular 'A' or Granular 'B' Type II, as amended by SSP 110S06. As the native silty clay at the culvert invert elevation on Hwy 17 is generally fine grained, a non-woven geotextile shall be placed between the native soil and the granular backfill / bedding material(s). The geotextile shall meet the specifications for OPSS.PROV 1860 (Geotextiles) Class II and have a filtration opening size (FOS) not greater than 212 µm. The granular fill shall be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD) of the material in accordance with OPSS.PROV 501 (Compacting), as amended by SSP 105S22. Sub-aqueous fill placement is not recommended at this site.

### 6.5.4 Bedding

Culvert construction should be in accordance with OPSS.PROV 421 (Pipe Culvert Installation in Open Cut), as amended by SSP 104S06, and OPSS.PROV 401 (Trenching, Backfill, and Compacting). The bedding and embedment for the replacement pipe culvert should be constructed in accordance with OPSD 802.010 (Flexible Pipe Embedment and Backfill) or OPSD 802.031 or 802.032 (Rigid Pipe Bedding, Cover and Backfill), as appropriate.

### **Culvert Extension – MR 55 STA 10+147**

It is recommended that the bedding be a minimum 300 mm thick. The bedding layer should consist of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' (Type I, II, or III) with 100% passing the 26.5 mm sieve size. Granular 'B' Type II is recommended in wet conditions. The bedding shall be placed in a maximum 200 mm thick loose lifts, be compacted to not less than 98% of the SPMDD in accordance OPSS.PROV 501 (Compacting), as amended by SSP 105S22, and be shaped to receive the bottom of the pipe. Care is required to ensure adequate compaction is achieved below the haunches of the pipe culvert extension.

Inspection of the subgrade and of the placed/compacted bedding shall be carried out by qualified geotechnical personnel during all engineered fill placement operations, to ensure that appropriate materials are used and that adequate levels of compaction by the construction equipment have been achieved.

### **Culvert Extension – Hwy 17 EBL STA 14+563**

As discussed above in Section 6.5.3 "Subgrade Preparation", as the native soil below the bedding for Hwy 17 culvert extension is generally fine grained, it is recommended that a non-woven geotextile be placed between the native soil and the granular backfill and/or granular bedding. The geotextile shall meet the specifications for OPSS.PROV 1860 (Geotextiles) Class II, and have a filtration opening size (FOS) not greater than 212 µm. Given the anticipated relatively large diameter (i.e., 0.9 m) pipe extension and the potential for disturbance of the clayey silt to silty clay subgrade, it is recommended that the bedding be a minimum 300 mm thick. The bedding layer shall consist of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' (Type I, II, or III) with 100% passing the 26.5 mm sieve size. Given the potential for subgrade disturbance, we recommend the use of Granular 'B' Type II for bedding at this culvert site. The bedding shall be placed in maximum 200 mm thick loose lifts, be compacted to not less than 98% of the SPMDD in accordance OPSS.PROV 501 (Compacting), as amended by SSP 105S22, and be shaped to receive the bottom of the pipe. Care is required to ensure adequate compaction is achieved below the haunches of the pipe culvert extensions. Given the fine-grained nature of the founding soils at this site and the potential for subgrade disturbance/softening, sub-aqueous fill placement is not recommended at this site.

Inspection of the subgrade and of the placed/compacted bedding shall be carried out by qualified geotechnical personnel during all engineered fill placement operations, to ensure that appropriate materials are used and that adequate levels of compaction by the construction equipment have been achieved.

### **6.5.5 Embedment and Cover**

Embedment / cover around/over the pipe culvert extensions should consist of granular fill meeting the specifications for OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' (Type I, II or III) with 100% passing the 26.5 mm sieve, as per OPSS.PROV 401 (Trenching, Backfilling, and Compacting). The embedment and backfill around/over the culvert should be placed in a similar configuration to OPSD 802.010 (Flexible Pipe Embedment and Backfill) or OPSD 802.031 or 802.032 (Rigid Pipe Bedding, Cover and Backfill), as appropriate. The granular embankment backfill should be placed in maximum 200 mm thick loose lifts and be compacted to at least 98% of the SPMDD of the materials in accordance with OPSS.PROV 501 (Compacting), as amended by SSP 105S22. The fill should also be placed concurrently on both sides of the culvert, ensuring that the backfill depth on one side does not exceed the other side by more than 200 mm as per OPSS.PROV 401 (Trenching, Backfilling, and Compacting).

Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations, to ensure that appropriate materials are used and that adequate levels of

compaction have been achieved. It is particularly important that the adequate compaction be achieved for the embedment material supporting the culvert haunches.

### 6.5.6 Backfill

The backfill for embankment re-instatement / widening (i.e., between the culvert embedment / cover material and the pavement structure) could consist of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' (Type I, II, or III) and/or the excavated granular embankment fill materials. However, we do not recommend re-using the existing sandy silt fill from the culvert site on Highway 17 as culvert backfill material. The backfill over the culvert should be placed in a similar configuration to OPSD 802.010 (Flexible Pipe Embedment and Backfill) or OPSD 802.031 or 802.032 (Rigid Pipe Bedding, Cover and Backfill), as appropriate. The granular fill should be placed in maximum 300 mm thick loose lifts and compacted to not less than 98% of the SPMDD in accordance with OPSS.PROV 501 (Compacting), as amended by SSP105S22. Embankment restoration after completion of the culvert extension should be carried out in accordance with OPSS.PROV 206 as amended by SSP 102S05, 206F04 and 206F06. Further, it is recommended that the widened embankment fill be benched into the existing embankment as per OPSD 208.010 (Benching).

As discussed in Section 6.4.2 "Frost Protection", given that the culvert extensions are to be founded below the estimated 2.1 m depth of frost penetration for this site, and the recommended backfill and existing embankment fill materials are generally classified as having a low susceptibility to frost heaving (as per the MTO Northern Region Pavement Design Practices and Guidelines), a frost taper as per OPSD 803.030 (Frost Treatment – Pipe Culverts) is not required.

Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations, to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

### 6.5.7 Erosion Protection

Provision should be made for erosion protection of the embankment side slopes near the outlet of the culvert extension locations. The requirements for, and design of, erosion protection measures for the widened embankment side slope and new culvert outlets should be assessed by the Hydrology and Drainage Engineer(s). As a minimum, the exposed embankment side slope near the culvert extensions should be seeded and covered in accordance with OPSS.PROV 804 (Temporary Erosion Control), as amended by SSP 804F02 (if applicable). If additional erosion protection is required, consideration could be given to the use of rip-rap, rock protection, or granular sheeting, meeting the requirements of OPSS.PROV 1004 (Aggregates – Miscellaneous), as amended by SSP 110S16, which is placed/constructed in accordance with OPSS.PROV 511 (Rip-Rap, Rock Protection, and Granular Sheeting).

The requirements for and design of erosion protection measures for the inlet and outlet of the culvert should be assessed by the Hydrology and Drainage Engineer(s). As a minimum, rip-rap treatment for the outlet of the culvert extensions should be consistent with the standard presented in OPSD 810.010 (Rip-Rap Treatment).

### 6.5.8 Obstructions

The Contractor shall be alerted to the potential for cobble and boulder obstructions within the embankment fill and/or native soils as inferred to be present in Borehole 21-02 and Borehole 21-07. It is recommended that a Notice to Contractor be included in the Contract Documents to alert the Contractor to the potential presence of these obstructions. A sample Notice to Contractor is included in Appendix D. Note that the extent and depth of the obstruction(s) may vary beyond and between the borehole locations.

## 6.6 Corrosion Assessment and Protection

The results of analytical testing on soil samples recovered in Boreholes 21-02 and 21-07 are summarized in Section 4.4 “Analytical Laboratory Testing Results” and are included in Appendix C. The potential for sulphate attack and corrosion are discussed in the following sub-sections; however, it is ultimately up to the designer to determine the appropriate construction materials, including the appropriate type of cement for concrete elements (if required) and/or the need for corrosion protection for steel elements (if required).

### 6.6.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 “Additional requirements for concrete subjected to sulphate attack” for potential sulphate attack on concrete. The measured sulphate concentrations on the soil samples from Boreholes 21-02 and 21-07 are 0.0049% (i.e., 49 µg/g) and 0.0025% (i.e., 25 µg/g), respectively, which is below the S-3 (Moderate) exposure class and is considered negligible according to Table 7.2 in the MTO Gravity Pipe Guidelines (2014).

However, given that the culvert locations will be exposed to de-icing salts, it is recommended that a C-1 (reinforced concrete) or C-2 (non-structurally reinforced concrete) class exposure concrete be considered for any concrete elements required at these sites.

### 6.6.2 Potential for Corrosion

The pH of the soil is 6.65 and 6.27 for the samples obtained from Boreholes 21-02 and 21-07, respectively and according to the MTO Gravity Pipe Design Guidelines (2014), soil pH levels between 5.5 and 8.5 are generally not considered detrimental to culvert durability. However, the measured resistivity ranges from 580 to 1600 ohm-cm, which indicates that the soil has a “severe” corrosiveness potential ( $R < 2,000$  ohm-cm), as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014).

For this reason, consideration should be given to using a plastic pipe culvert as opposed to steel or concrete that are considered to be more susceptible to corrosion.

It should be noted that the water levels are subject to seasonal fluctuations and variations, due to precipitation events, and the soil chemistry could also be variable. These recommendations are provided as guidance only. The culvert designer should take the results of the laboratory testing and the potential for corrosion into consideration as part of the ultimate material selection.

## 7.0 CLOSURE

This Foundation Design Report was prepared by Mr. David Muldowney, P.Eng. Mr. Kevin Bentley, P. Eng., an MTO Foundations Designated Contact and Associate of Golder, conducted an independent quality control review and technical audit of this report.

## Signature Page

### **Golder Associates Ltd.**



David Muldowney, P.Eng.  
*Senior Geotechnical Engineer*



Kevin Bentley, M.E.Sc, P.Eng.  
*MTO Foundations Designated Contact, Associate*

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#### **ASTM International**

ASTM D1586	Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method of Field Vane Shear Test in Saturated Fine-Grained Soils
ASTM D2435	Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Incremental Loading

#### **Ontario Provincial Standard Specifications (OPSS)**

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 401	Construction Specification for Trenching, Backfilling, and Compacting
OPSS.PROV 421	Construction Specification for Pipe Culvert Installation in Open Cut
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Temporary Erosion Control
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS.PROV 1860	Material Specification for Geotextiles

#### **OPSS Standard Special Provisions**

SSP 102S05	Amendment to OPSS 206
SSP 104S06	Amendment to OPSS 421
SSP 105S09	Amendment to OPSS 539
SSP 105S22	Amendment to OPSS 501
SSP 110S06	Amendment to OPSS 1010
SSP 110S16	Amendment to OPSS 1004



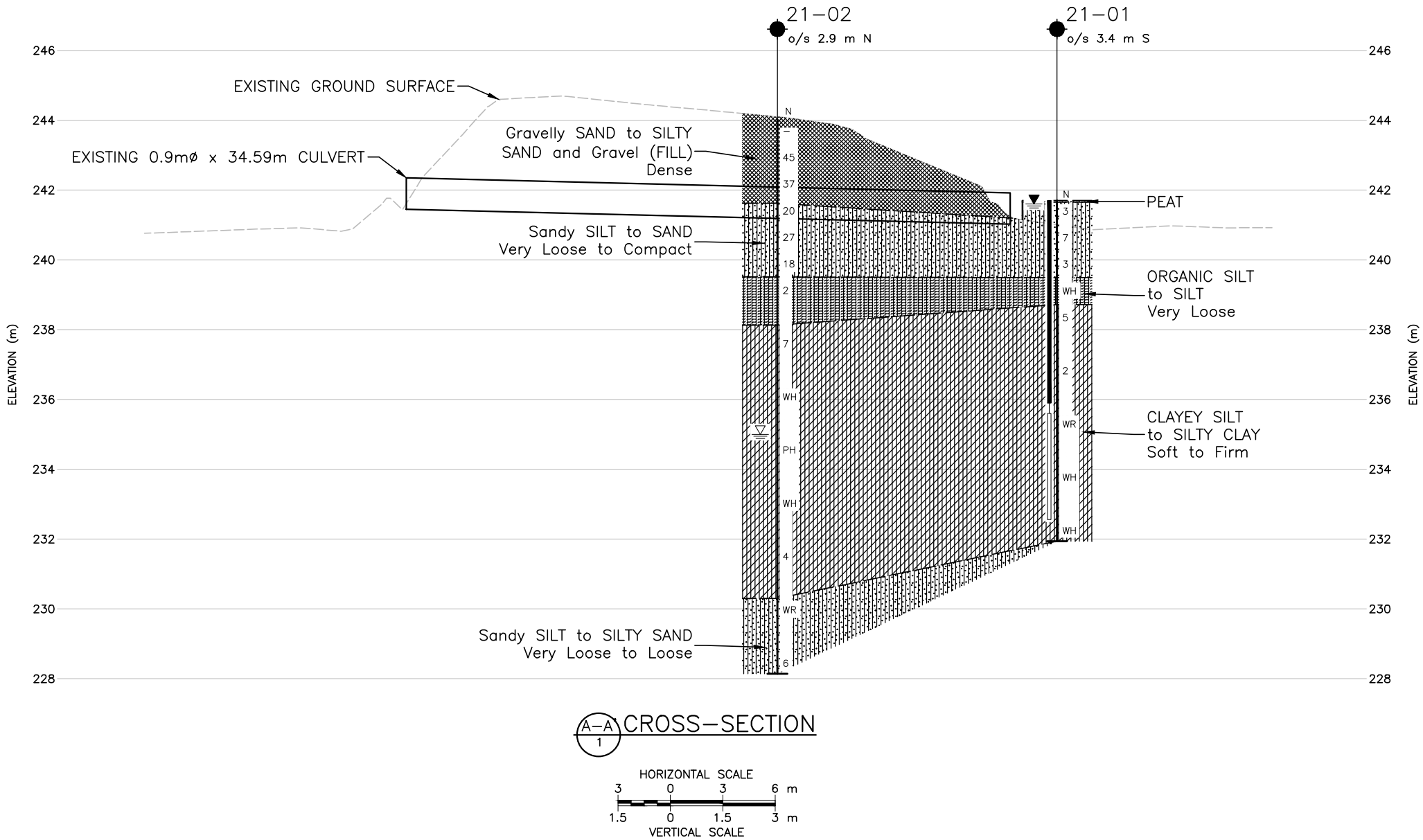
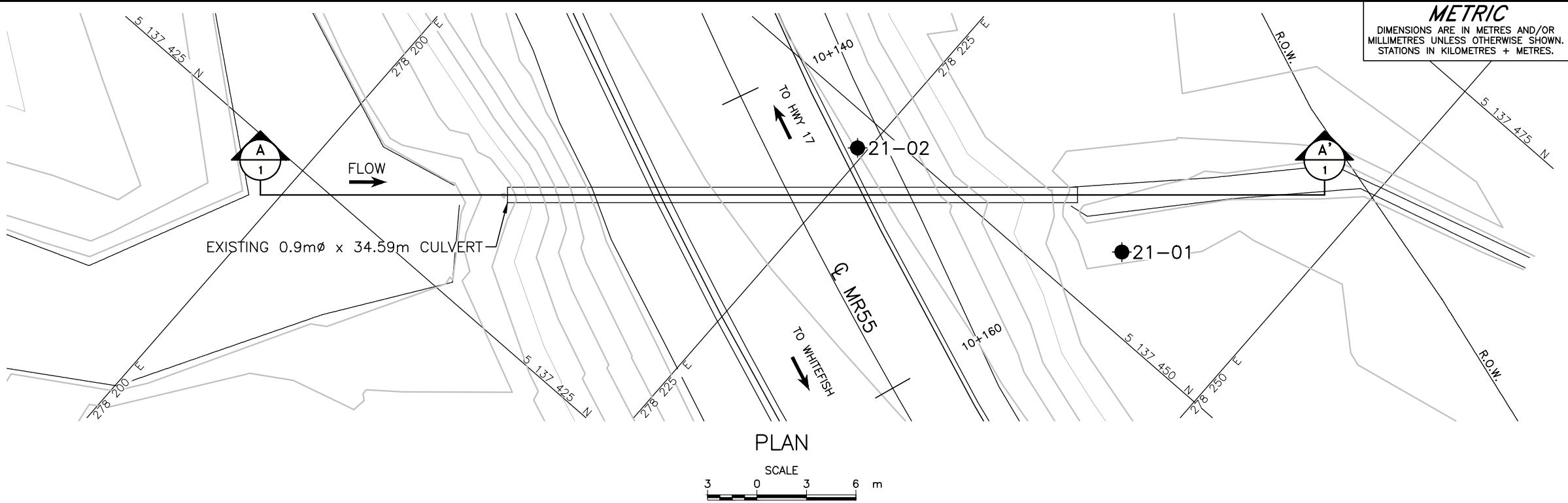
SSP 206F04	Amendment to OPSS 206
SSP 206F06	Amendment to OPSS 206
SSP 517F01	Amendment to OPSS 517
SSP 804F02	Amendment to OPSS 804

**Ontario Provincial Standard Drawings (OPSD)**

OPSD 203.020	Embankments Over Sump, Existing Slope Excavated
OPSD 208.010	Benching of Earth Slopes
OPSD 802.010	Flexible Pipe, Embedment and Backfill, Earth Excavation
OPSD 802.031	Rigid Pipe Bedding, Cover and Backfill, Type 3 Soil Earth Excavation
OPSD 802.032	Rigid Pipe Bedding, Cover and Backfill, Type 4 Soil Earth Excavation
OPSD 803.030	Frost Treatment, Pipe Culverts, Frost Penetration Line Below Bedding Grade
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.100	Foundation Frost Penetration Depths for Northern Ontario


**Ontario Water Resource Act**


Regulation 903	Wells (as amended)
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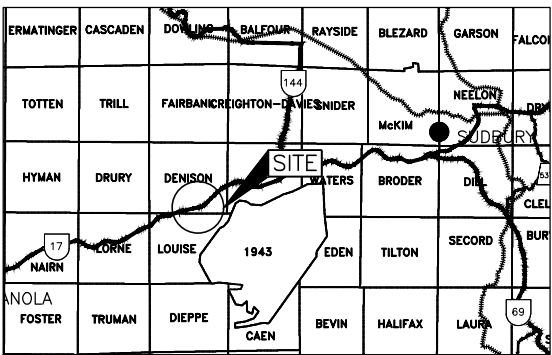
CONT No. GWP No. 5032-19-00

MUNICIPAL ROAD 55  
CULVERT EXTENSION AT STATION 10+147  
BOREHOLE LOCATION AND SOIL STRATA


**GOLDER**  
MEMBER OF WSP





SHEET





KEY PLAN  
SCALE 10 0 10 20 km


 Borehole – Current Investigation


 Seal

 Piezometer

 Standard Penetration Test Value

 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)

 WL in piezometer, measured on FEB 9, 2021

 WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 12)			
No.	ELEVATION	NORTHING	EASTING
21-01	241.7	5137454.0	278240.7
21-02	244.0	5137448.2	278224.5

DRAFT

NOTES

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

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

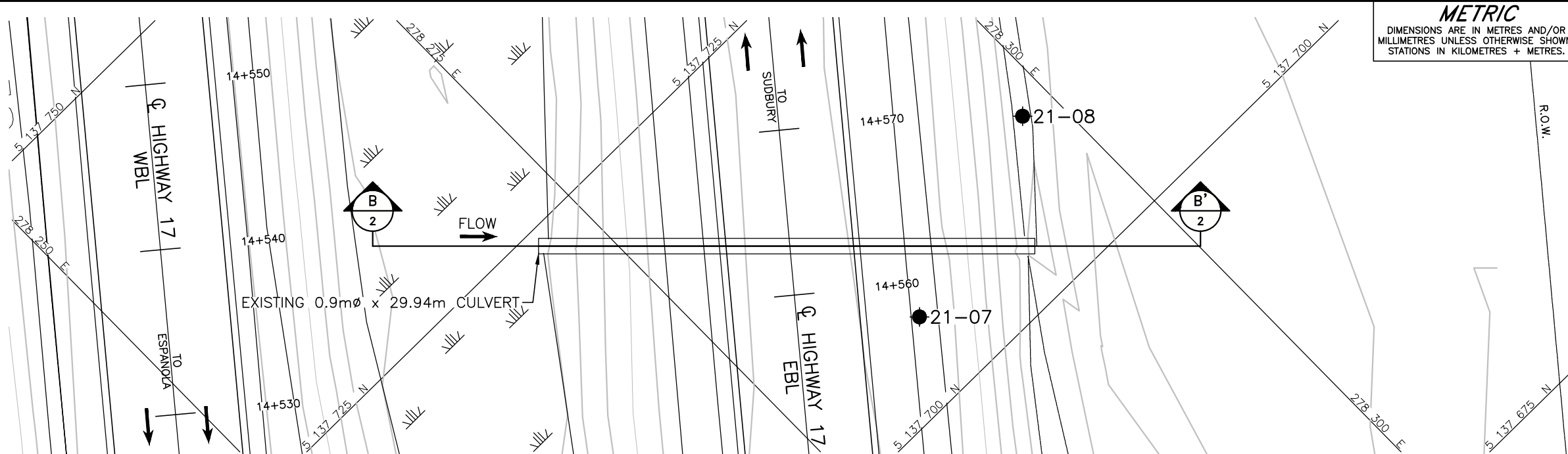
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Base plans provided in digital format by AECOM CANADA LTD., drawing file no. Hwy 17-MR55.dwg, received MARCH 8, 2021.

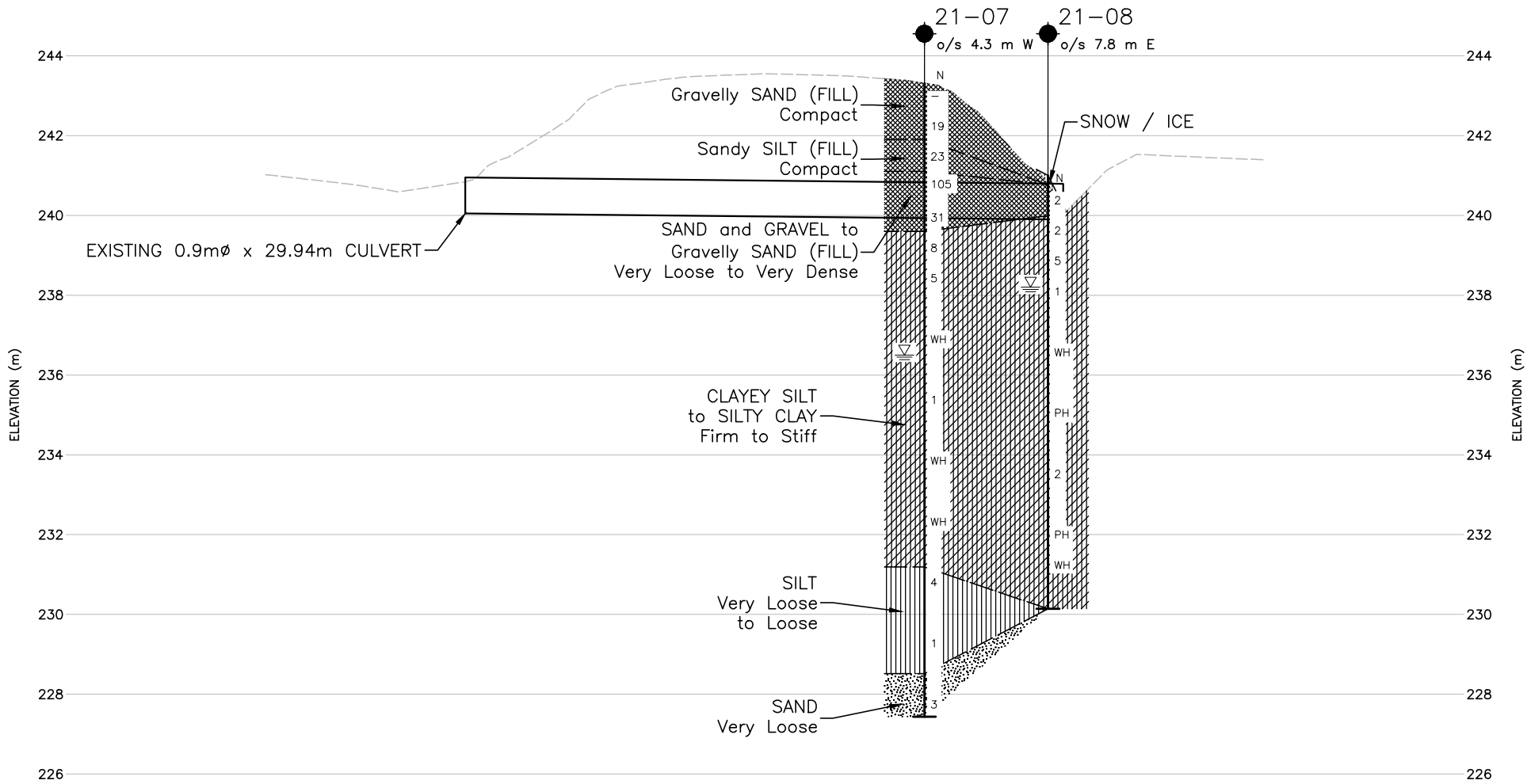
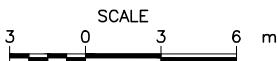
NO.	DATE	BY	REVISION

Geocres No. PROJECT NO. 20253807 DIST.

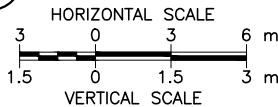
MR. 55	CHKD. MT	DATE: 7/20/2021	SITE: <span> </span>
SUBM'D.	CHKD. DAM	APPD. KB	DWG. 1



PLAN



B-B CROSS-SECTION



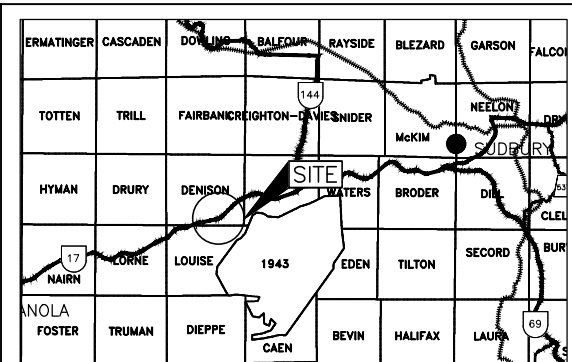
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No. .  
GWP No. 5032-19-00

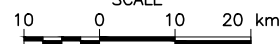


HIGHWAY 17 EASTBOUND LANE  
CULVERT EXTENSION AT STATION 14+563  
BOREHOLE LOCATION AND SOIL STRATA

SHEET



KEY PLAN



LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
21-07	243.3	5137704.9	278285.0
21-08	241.0	5137709.2	278297.9

**DRAFT**

NOTES

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

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

REFERENCE

Base plans provided in digital format by AECOM CANADA LTD., drawing file no. Hwy 17-MR55.dwg, received MARCH 8, 2021.

NO.	DATE	BY	REVISION
1			
Geocres No. .			
HWY. 17	PROJECT NO. 20253807		DIST. .
SUBM'D.	CHKD. MT	DATE: 7/23/2021	SITE: .
DRAWN: TR	CHKD. DAM	APPD. KB	DWG. 2



Photographs: Municipal Road 55 and Highway 11 Culvert Extension



**Photograph 1: Municipal Road 55, Facing North from Culvert Location (June 2021)**



**Photograph 2: Municipal Road 55, Facing South towards culvert (June 2021)**



Photographs: Municipal Road 55 and Highway 11 Culvert Extension

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**Photograph 3: Municipal Road 55 East Embankment Side Slope, Facing North (June 2021)**



**Photograph 4: Municipal Road 55 East Embankment Side Slope, Facing South (June 2021)**



Photographs: Municipal Road 55 and Highway 11 Culvert Extension

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**Photograph 5: Municipal Road 55 Culvert Outlet, Facing East (June 2021)**



Photographs: Municipal Road 55 and Highway 11 Culvert Extension



**Photograph 6: Highway 17 EBL, Facing East at culvert location (June 2021)**



**Photograph 7: Highway 17 EBL, Facing West at culvert location (June 2021)**



Photographs: Municipal Road 55 and Highway 11 Culvert Extension

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**Photograph 8: Highway 17 EBL South Embankment Side Slope, Facing East (June 2021)**



**Photograph 9: Highway 17 EBL South Embankment Side Slope, Facing West (June 2021)**



Photographs: Municipal Road 55 and Highway 11 Culvert Extension

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**Photograph 10: Highway 17 Submerged Culvert Outlet, Facing South (June 2021)**

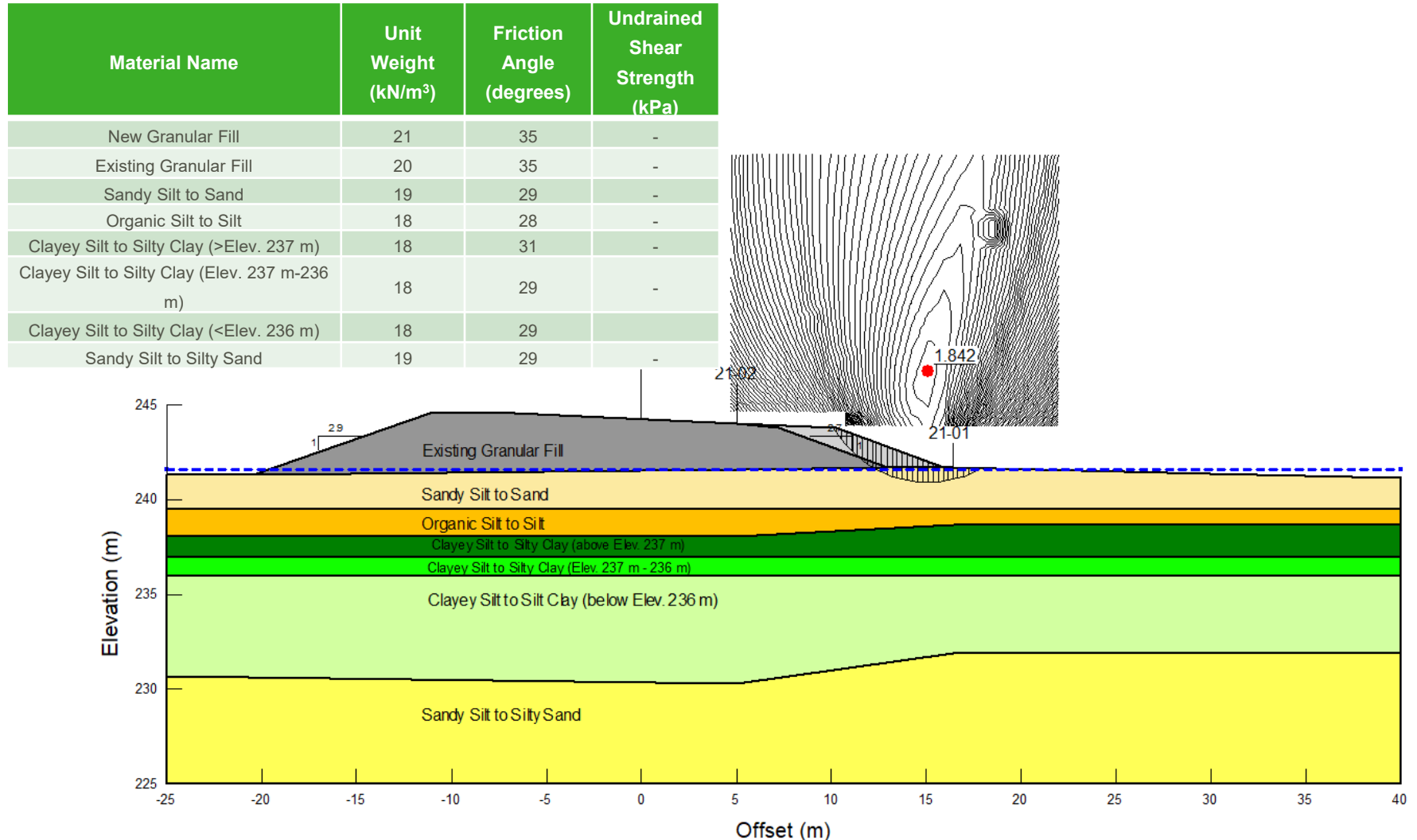


**Table 1: Comparison of Alternative Culvert Extension Types**

Option	Advantages	Disadvantages	Risks/Consequences
Pipe Culvert Extension	<ul style="list-style-type: none"> <li>Compatible with existing plastic pipe culverts.</li> <li>Minimizes foundation stresses and potential post-construction settlement compared to heavier concrete structures.</li> <li>Minimizes depth of excavation, protection systems (if required), and dewatering requirements compared to open footing option.</li> <li>Allows for faster construction resulting in shorter duration for unwatering and surface water pumping compared to an open-footing and to a lesser extent, a box culvert option.</li> <li>More tolerant to differential settlement compared to an open footing or closed bottom concrete box culvert.</li> </ul>	<ul style="list-style-type: none"> <li>Reduced flow-through capacity compared to box culvert and open-footing culvert options with a similar span.</li> <li>Difficult to compact backfill materials to level of culvert springline (below the haunches) if not constructed in dry conditions.</li> <li>If CSP extensions are used, CSPs have a shorter design life compared to concrete or plastic options; however, asphalt coatings or polymer linings can be incorporated to increase design life.</li> </ul>	<ul style="list-style-type: none"> <li>Lower risk of disturbance of the native subgrade soils during construction; can be mitigated with use of a granular working pad/bedding layer.</li> <li>Lower risk related to total and differential settlements compared to box or open-footing options, especially if flexible pipe is used.</li> <li>Plastic pipe preferred over concrete or steel due to corrosive environment.</li> </ul>
Pre-Cast Box Culvert Extension	<ul style="list-style-type: none"> <li>Typically has a longer design life compared to steel culverts.</li> <li>Minimizes depth of excavation, protection system, and dewatering requirements compared to an open-footing option.</li> <li>Allows for faster construction, resulting in shorter duration for dewatering and surface water pumping compared to an open-footing culvert.</li> <li>More tolerant of total and differential settlement compared to an open-footing culvert but less tolerant compared to a pipe culvert.</li> <li>Allows for greater flow volume compared to a similar sized pipe culvert extension.</li> </ul>	<ul style="list-style-type: none"> <li>High foundation stresses compared to a pipe and less tolerant to differential settlement.</li> <li>Transportation to and on-site lifting of pre-cast sections will be required.</li> <li>Specialized connection / tie-in to existing plastic pipes required.</li> </ul>	<ul style="list-style-type: none"> <li>Lower risk of dewatering issues as box culverts do not require as much effort to shape/compact the bedding compared to a pipe option.</li> <li>Higher risk related to settlement performance; however, box segments can accommodate some total and differential settlement.</li> </ul>

Option	Advantages	Disadvantages	Risks/Consequences
Open Footing Culvert	<ul style="list-style-type: none"> <li>■ Typically has a longer design life compared to steel culverts.</li> <li>■ May be feasible to construct the culvert on pre-cast footing sections to accelerate construction schedule and reduce time for dewatering/unwatering (pumping).</li> <li>■ Readily suitable for construction using concrete or metal sections.</li> <li>■ Allows for greater flow volume than a pipe with a similar span.</li> </ul>	<ul style="list-style-type: none"> <li>■ Highest foundation stresses and least tolerant to total and differential settlement.</li> <li>■ Excavation depths are greater than for a box culvert or pipe culvert, resulting in increased excavation support and dewatering requirements and additional spoil material to be disposed off-site.</li> <li>■ Transportation to and on-site lifting of sections if precast segments are used.</li> <li>■ Longer anticipated schedule for construction; however, precast footings and/or open box segments can be used to expedite the schedule.</li> <li>■ Specialized connection / tie-in to existing plastic pipes required.</li> </ul>	<ul style="list-style-type: none"> <li>■ Higher risk of disturbance of the native subgrade soils during construction.</li> <li>■ Highest risk related to settlement performance; culvert joints may be required to accommodate the total and differential settlement.</li> </ul>

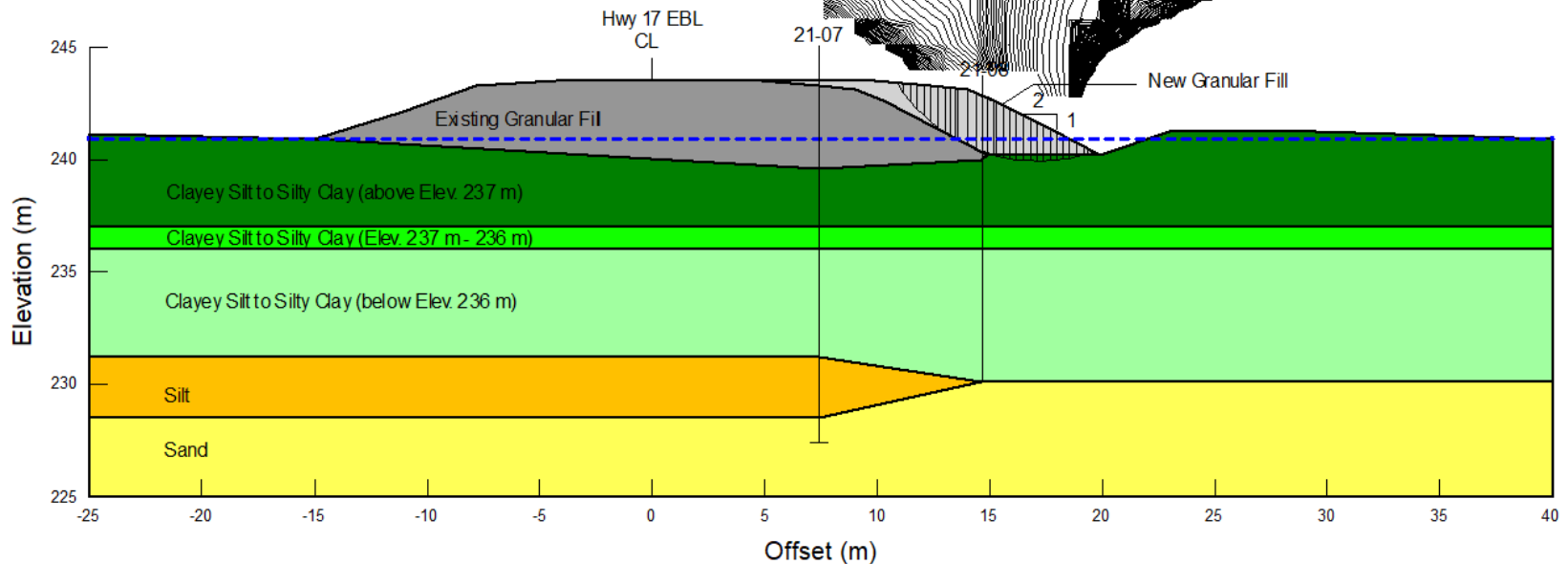
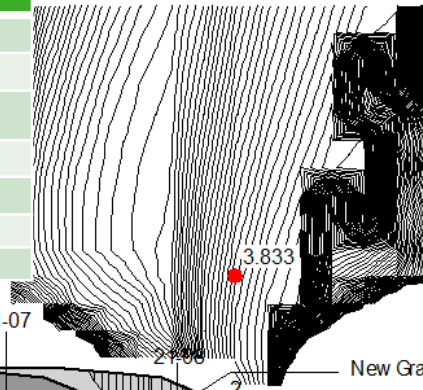
### Culvert Extension – Municipal Road 55 Station 10+147 Long-term (Drained) Condition



### Culvert Extension – Highway 17 EBL Station 14+563

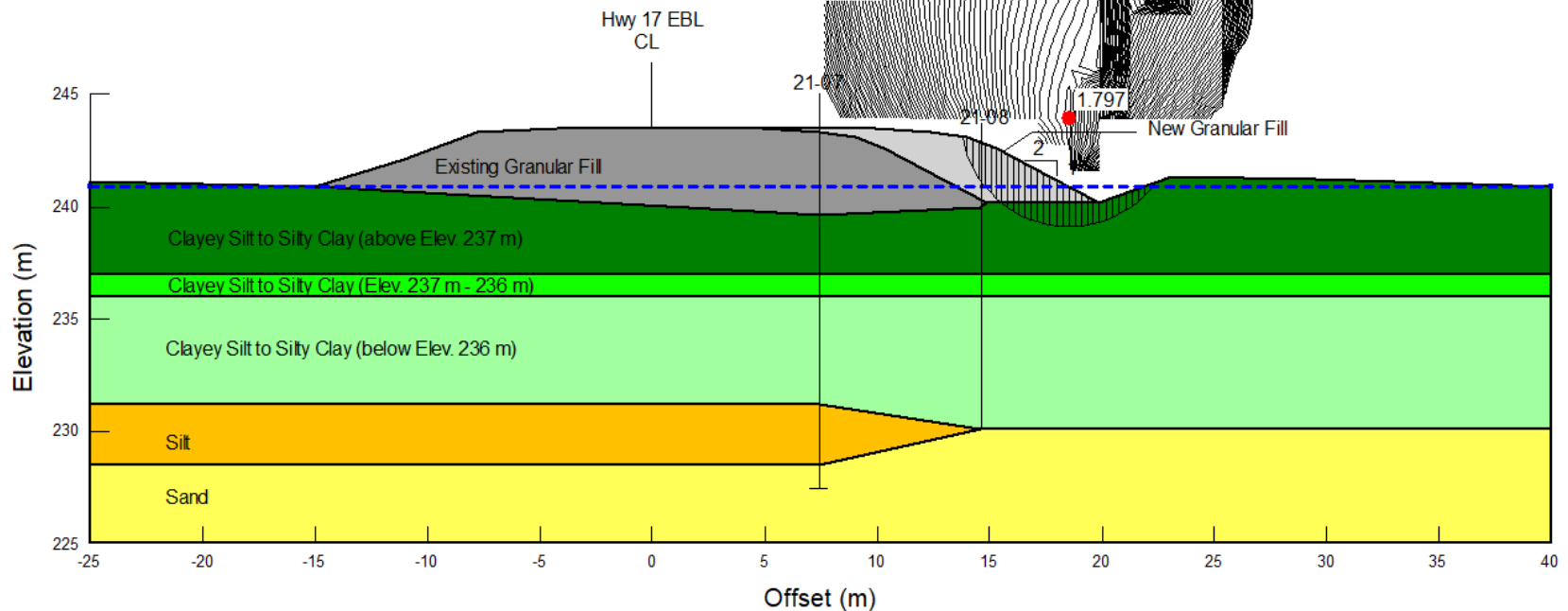
#### Short-term (Undrained) Condition

Material Name	Unit Weight (kN/m³)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
New Granular Fill	21	35	-
Existing Granular Fill	20	35	-
Clayey Silt to Silty Clay (>Elev. 237m)	18	--	55
Clayey Silt to Silty Clay (Elev. 237-236m)	18	-	55-30
Clayey Silt to Silt Clay (<Elev. 236m)	18	-	30
Silt	18	28	-
Sand	19	30	-



### Culvert Extension – Highway 17 EBL Station 14+563 Long-term (Drained) Condition

Material Name	Unit Weight (kN/m³)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
New Granular Fill	21	35	-
Existing Granular Fill	20	35	-
Clayey Silt to Silty Clay (>Elev. 237 m)	18	31	-
Clayey Silt to Silty Clay (Elev. 237-236 m)	18	29	-
Clayey Silt to Silty Clay (<Elev. 236 m)	18	29	-
Silt	18	28	-
Sand	19	30	-

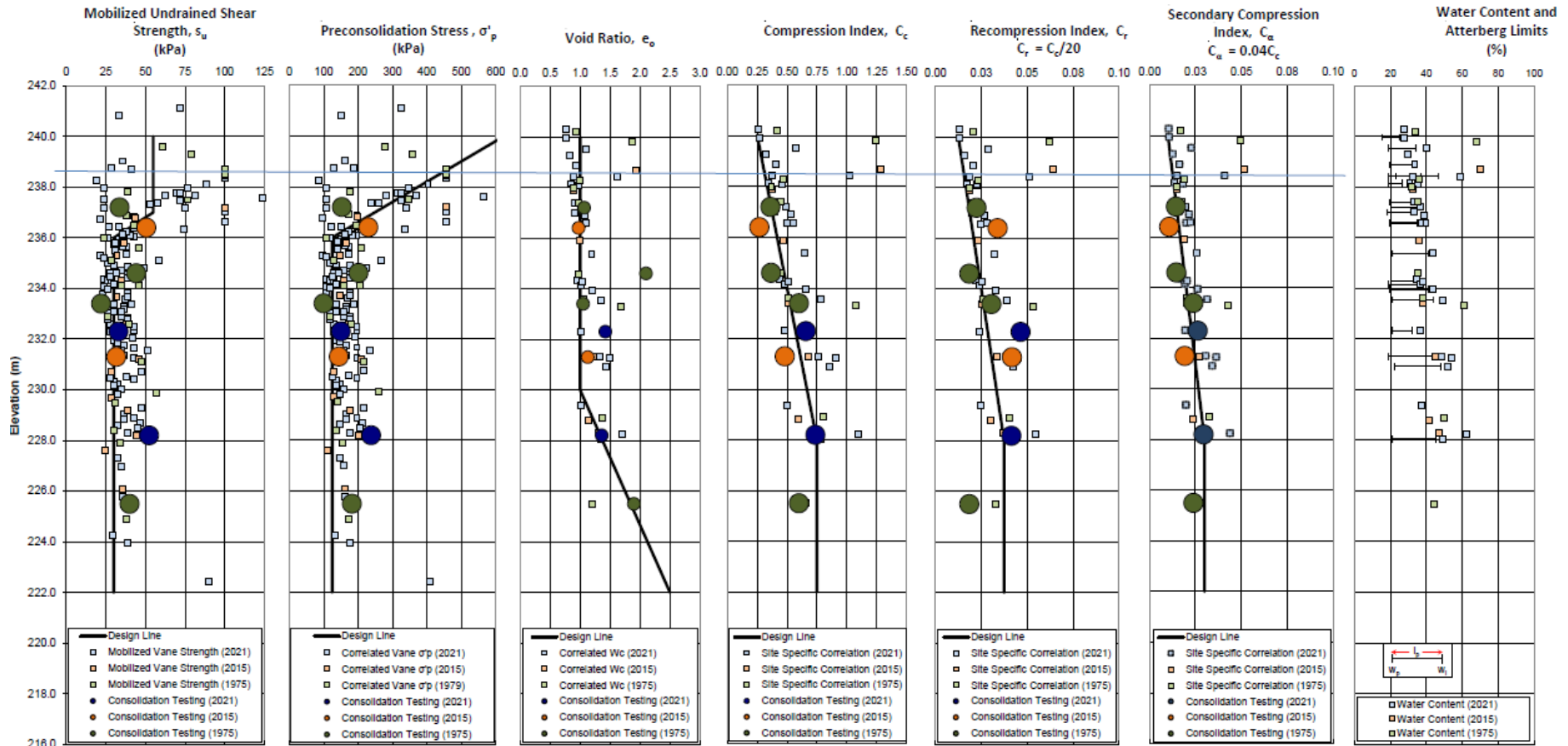


# DRAFT

## Summary of Engineering Parameters for Cohesive Deposits

### Highway 17 and MR55 Widening

Figure 4



## APPENDIX A

# Record of Boreholes



# ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

## MINISTRY OF TRANSPORTATION, ONTARIO

### PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

### MODIFIERS FOR SECONDARY COMPONENTS<sup>1,2</sup>

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component ( <i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some ( <i>i.e.</i> , some sand)
≤ 10	trace ( <i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

### PENETRATION RESISTANCE

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

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

#### Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q<sub>t</sub>*), porewater pressure (*u*) and sleeve friction (*f<sub>s</sub>*) are recorded electronically at 25 mm penetration intervals.

#### Dynamic Cone Penetration Resistance (DCPT); N<sub>d</sub>:

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

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

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

### SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

### SOIL TESTS

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

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

### COARSE-GRAINED SOILS

#### Compactness<sup>1</sup>

Term	SPT 'N' (blows/0.3m) <sup>2</sup>
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

- Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
- SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

### FINE-GRAINED SOILS

#### Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' <sup>1,2</sup> (blows/0.3m)
Very Soft	< 12	0 to 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	> 200	> 30

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
- SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

### Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

**LIST OF SYMBOLS**  
**MINISTRY OF TRANSPORTATION, ONTARIO**

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

**I. GENERAL**

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

**II. STRESS AND STRAIN**

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta\sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

$\sigma_{oct}$	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

**III. SOIL PROPERTIES**

**(a) Index Properties**

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

**(a) Index Properties (continued)**

w	water content
$w_L$ or LL	liquid limit
$w_P$ or PL	plastic limit
$I_P$ or PI	plasticity index = $(w_L - w_P)$
NP	non-plastic
$w_s$	shrinkage limit
$I_L$	liquidity index = $(w - w_P) / I_P$
$I_c$	consistency index = $(w_L - w) / I_P$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

**(b) Hydraulic Properties**

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

**(c) Consolidation (one-dimensional)**

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

**(d) Shear Strength**

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

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

**Notes:** 1  
 2

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

## LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

## WEATHERING CLASSIFICATION

**Fresh (W1):** no visible sign of rock material weathering.

**Slightly Weathered (W2):** discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

**Moderately Weathered (W3): less than 50%** of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

**Highly Weathered (W4): more than 50%** of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

**Completely Weathered (W5): 100%** of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

**Residual Soil (W6): all rock material is converted to soil.** The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

## BEDDING THICKNESS

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

## JOINT OR FOLIATION SPACING

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

## GRAIN SIZE

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

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

## CORE CONDITION

## Total Core Recovery (TCR)

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

## Solid Core Recovery (SCR)

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

## Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

## DISCONTINUITY DATA

## Fracture Index

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

## Dip with Respect to Core Axis

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

## Description and Notes

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

## Abbreviations

AXJ Axial Joint	KV Karstic Void
BD Bedding	K Slickensided
BC Broken Core	LC Lost Core
CC Continuous Core	MB Mechanical Break
CL Closed	PL Planar
CO Contact	PO Polished
CU Curved	RO Rough
CT Coated	SA Slightly Altered
FLT Fault	SH Shear
FOL Foliation	SM Smooth
FR Fracture	SR Slightly Rough
GO Gouge	SY Stylolite
IN Infilled	UN Undulating
IR Irregular	VN Vein
JN Joint	VR Very Rough

## ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250

PROJECT 20253807		RECORD OF BOREHOLE No. 21-01				1 OF 1 METRIC	
G.W.P. 5032-19-00		LOCATION N 5137454.0; E 278240.7 NAD83 MTM ZONE 12 (LAT. 46.376275; LONG. -81.345249)				ORIGINATED BY AD	
DIST HWY 17		BOREHOLE TYPE CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers				COMPILED BY TR	
DATUM GEODETIC		DATE February 1, 2021				CHECKED BY DAM	

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							
241.7	GROUND SURFACE							20 40 60 80 100							
241.7	PEAT (PT) (50 mm) Brown Moist		1	SS	3			○ UNCONFINED + FIELD VANE							
	SAND (SP), trace to some silt to SILTY SAND (SM), trace to some gravel Very loose to loose Grey Moist to wet		2	SS	7			● QUICK TRIAXIAL × REMOULDED							
			3	SS	3										
239.5															
239.5	Sandy ORGANIC SILT (OL) Very loose Dark grey Wet		4	SS	WH										NP, OC=5.0%
238.7															
238.7	SILTY CLAY (CI), trace sand Firm Grey Wet		5	SS	5										
	- Varved below 4.6 m depth.		6	SS	2										
			7	SS	WR										
			8	SS	WH										
233.0															
233.0	CLAYEY SILT (CL) Firm Grey Wet		9	SS	WH										0 0 85 15
231.9															
231.9	END OF BOREHOLE														
NOTES:  1. Borehole dry upon completion of drilling.  2. Water level in piezometer measured below ground surface as follows: Date Depth (m) Feb 3, 2021 1.4 Feb 8, 2021 0.1 Feb 9, 2021 0.1															


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PROJECT <u>20253807</u>			RECORD OF BOREHOLE <b>No. 21-02</b>			1 OF 2 <b>METRIC</b>															
G.W.P. <u>5032-19-00</u>			LOCATION <u>N 5137448.2; E 278224.5 NAD83 MTM ZONE 12 (LAT. 46.376222; LONG. -81.345459)</u>			ORIGINATED BY <u>TB</u>															
DIST <u>          </u> HWY <u>17</u>			BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>			COMPILED BY <u>TR</u>															
DATUM <u>GEODETIC</u>			DATE <u>February 1, 2021</u>			CHECKED BY <u>DAM</u>															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub>	W	W <sub>L</sub>	WATER CONTENT (%)	γ	GR	SA	SI	CL
244.0	GROUND SURFACE							20	40	60	80	100									
0.0	Gravelly SAND (SP), some silt to SILTY SAND (SM) and gravel (FILL) Dense Brown Moist  - Auger grinding between 0.8 m and 2.4 m depth.		1	AS	-																
			2	SS	45																
			3	SS	37																
241.6	Sandy SILT (ML), some clay, trace gravel Compact Brown Moist to wet  - Auger grinding between 3.0 m and 3.8 m depth.		4	SS	20																
2.4			5	SS	27																
			6	SS	18																
239.5	SILT (ML), trace organics Very loose Grey Wet		7	SS	2																
4.5																					
238.1	SILTY CLAY (CI) Soft to firm Grey Wet  - Varved below 7.2 m depth.		8	SS	7																
5.9																					
			9	SS	WH																
			10	TO	PH																
233.4	CLAYEY SILT (CL) Firm Grey Wet		11	SS	WH																
10.6																					

Continued Next Page


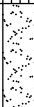
+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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PROJECT <u>20253807</u>			RECORD OF BOREHOLE <b>No. 21-02</b>			2 OF 2 <b>METRIC</b>												
G.W.P. <u>5032-19-00</u>			LOCATION <u>N 5137448.2; E 278224.5 NAD83 MTM ZONE 12 (LAT. 46.376222; LONG. -81.345459)</u>			ORIGINATED BY <u>TB</u>												
DIST <u>          </u> HWY <u>17</u>			BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>			COMPILED BY <u>TR</u>												
DATUM <u>GEODETIC</u>			DATE <u>February 1, 2021</u>			CHECKED BY <u>DAM</u>												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub>	W	W <sub>L</sub>	WATER CONTENT (%)	γ	GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100										
230.3	CLAYEY SILT (CL) Firm Grey Wet		12	SS	4		231											
13.7	SILTY SAND (SM) to Sandy SILT (ML) Very loose to loose Grey Wet		13	SS	WR		230											
228.1			14	SS	6		229											
15.9	END OF BOREHOLE  NOTE:  1. Water level measured at a depth of 9.0 m below ground surface (Elev. 235.0 m) inside augers upon completion of drilling.																	

SUD-MTO 001 R:\SUDBURY\SIM\CLIENTS\IMTO\HWY17\_MR\_5502\_DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/23/21 TR

PROJECT		RECORD OF BOREHOLE No. 21-07				1 OF 2		METRIC				
G.W.P. 5032-19-00		LOCATION N 5137704.9; E 278285.0 NAD83 MTM ZONE 12 (LAT. 46.378534; LONG. -81.344687)				ORIGINATED BY TB/NP						
DIST _____ HWY 17		BOREHOLE TYPE CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers				COMPILED BY TR						
DATUM GEODETIC		DATE February 2, 2021				CHECKED BY DAM						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED				
								WATER CONTENT (%)				
243.3	GROUND SURFACE						20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
0.0	Gravelly SAND (SP) (FILL) Compact Brown Moist		1	AS	-							
			2	SS	19							
241.9												
1.4	Sandy SILT (ML), some clay, some gravel (FILL) Compact Brown Moist		3	SS	23							
241.1												
2.2	Gravelly SAND (SP) (FILL) Compact to very dense Brown Moist to wet  - Split-spoon refusal (i.e. hammer bouncing) on inferred cobble / boulder at 2.7 m depth.		4	SS	105							
			5	SS	31							
239.6												
3.7	SILTY CLAY (CI) Firm to stiff Grey Wet		6	SS	8							
			7	SS	5							
			8	SS	WH							
			9	SS	1							
			10	SS	WH							

PROJECT <u>20253807</u>			RECORD OF BOREHOLE <b>No. 21-07</b>			2 OF 2 <b>METRIC</b>														
G.W.P. <u>5032-19-00</u>			LOCATION <u>N 5137704.9; E 278285.0 NAD83 MTM ZONE 12 (LAT. 46.378534; LONG. -81.344687)</u>			ORIGINATED BY <u>TB/NP</u>														
DIST <u>          </u> HWY <u>17</u>			BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>			COMPILED BY <u>TR</u>														
DATUM <u>GEODETIC</u>			DATE <u>February 2, 2021</u>			CHECKED BY <u>DAM</u>														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub>	W	W <sub>L</sub>	γ	GR	SA	SI	CL
231.2 12.1	SILT (ML), trace clay, trace sand Very loose to loose Grey Wet		12	SS	4		231													
							230													
			13	SS	1		229													
228.5 14.8	SAND (SP), some silt Very loose Grey Wet		14	SS	3		228													
227.4 15.9	END OF BOREHOLE  NOTE:  1. Water level measured at a depth of 6.8 m below ground surface (Elev. 236.5 m) in augers upon completion of drilling.																			

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PROJECT <u>20253807</u>			RECORD OF BOREHOLE <b>No. 21-08</b>			1 OF 1 <b>METRIC</b>												
G.W.P. <u>5032-19-00</u>			LOCATION <u>N 5137709.2; E 278297.9 NAD83 MTM ZONE 12 (LAT. 46.378574; LONG. -81.344521)</u>			ORIGINATED BY <u>TB</u>												
DIST <u>          </u> HWY <u>17</u>			BOREHOLE TYPE <u>Portable Equipment, NW Casing with Wash Boring</u>			COMPILED BY <u>TR</u>												
DATUM <u>GEODETIC</u>			DATE <u>February 8, 2021</u>			CHECKED BY <u>DAM</u>												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub>	W	W <sub>L</sub>	WATER CONTENT (%)	γ	GR SA SI CL
241.0	TOP OF SNOW							20 40 60 80 100										
0.0	SNOW / ICE							20 40 60 80 100										
240.7																		
0.3	SAND (SP) and gravel, trace silt (FILL) Very loose		1	SS	2													
240.0	- No recovery in Sample No. 1.																	
1.0	CLAYEY SILT (CL) Stiff Grey Moist to wet		2	SS	2													
			3	SS	5													
			4	SS	1													
	- Vane could not be advanced at 4.0 m depth.		5	SS	WH													
236.0	SILTY CLAY (CI) Firm to stiff Grey Wet																	
5.0			6	TO	PH													
	- Varves of clayey silt encountered below 7.2 m depth.		7	SS	2													
			8A	TO	PH													
			8B															
			9	SS	WH													
231																		
230.1	END OF BOREHOLE																	
10.9	NOTES:  1. Water level measured at a depth of 2.8 m below ground surface (Elev. 238.2 m) in casing upon completion of drilling.																	

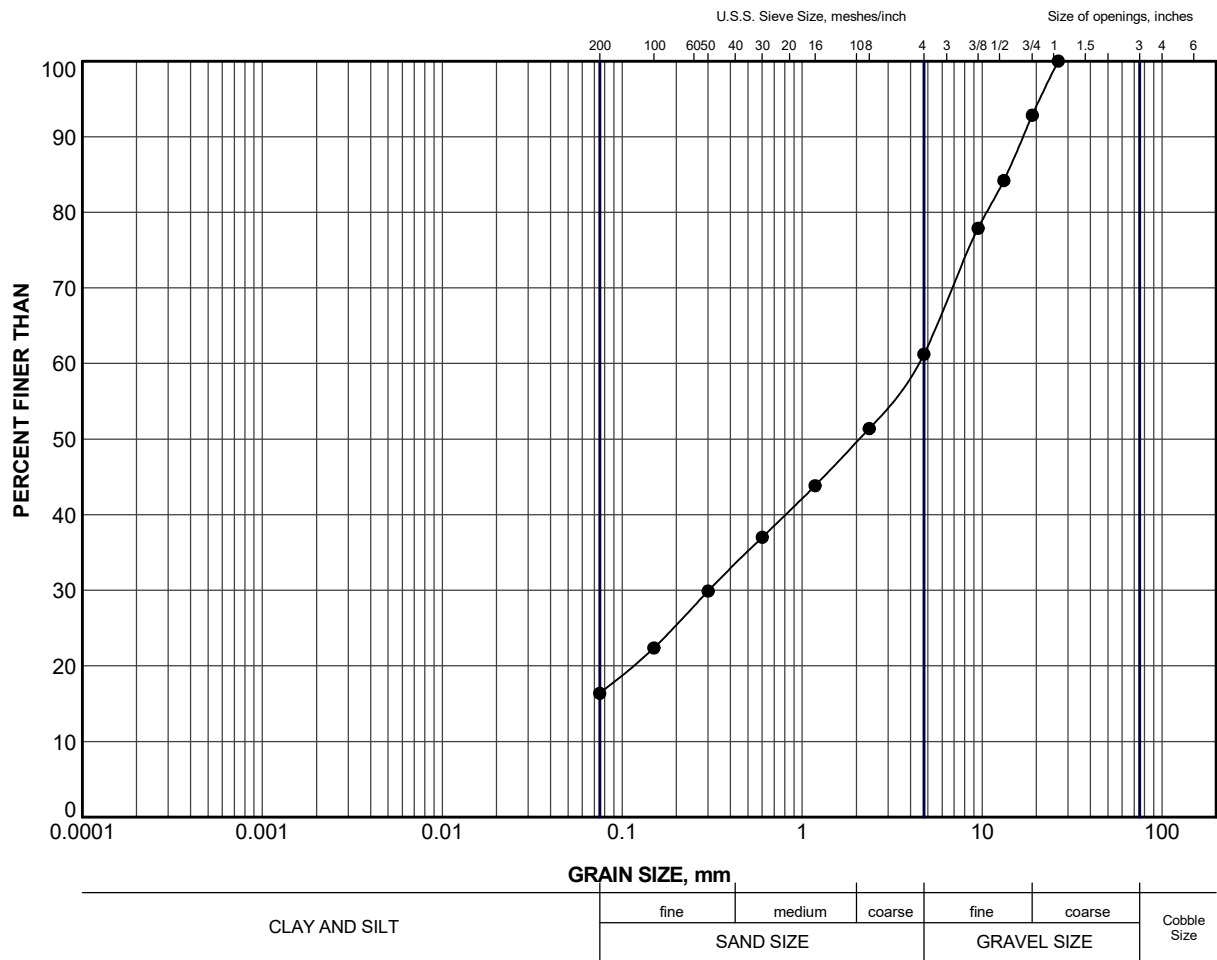
SUD-MTO 001 R:\SUDBURY\CLIENTS\IMTO\HWY17\_MR\_5502\_DATA\GINT\20253807\20253807.GPJ GAL-MISS GDT 7/23/21 TR

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

## APPENDIX B

# Geotechnical Laboratory Test Results

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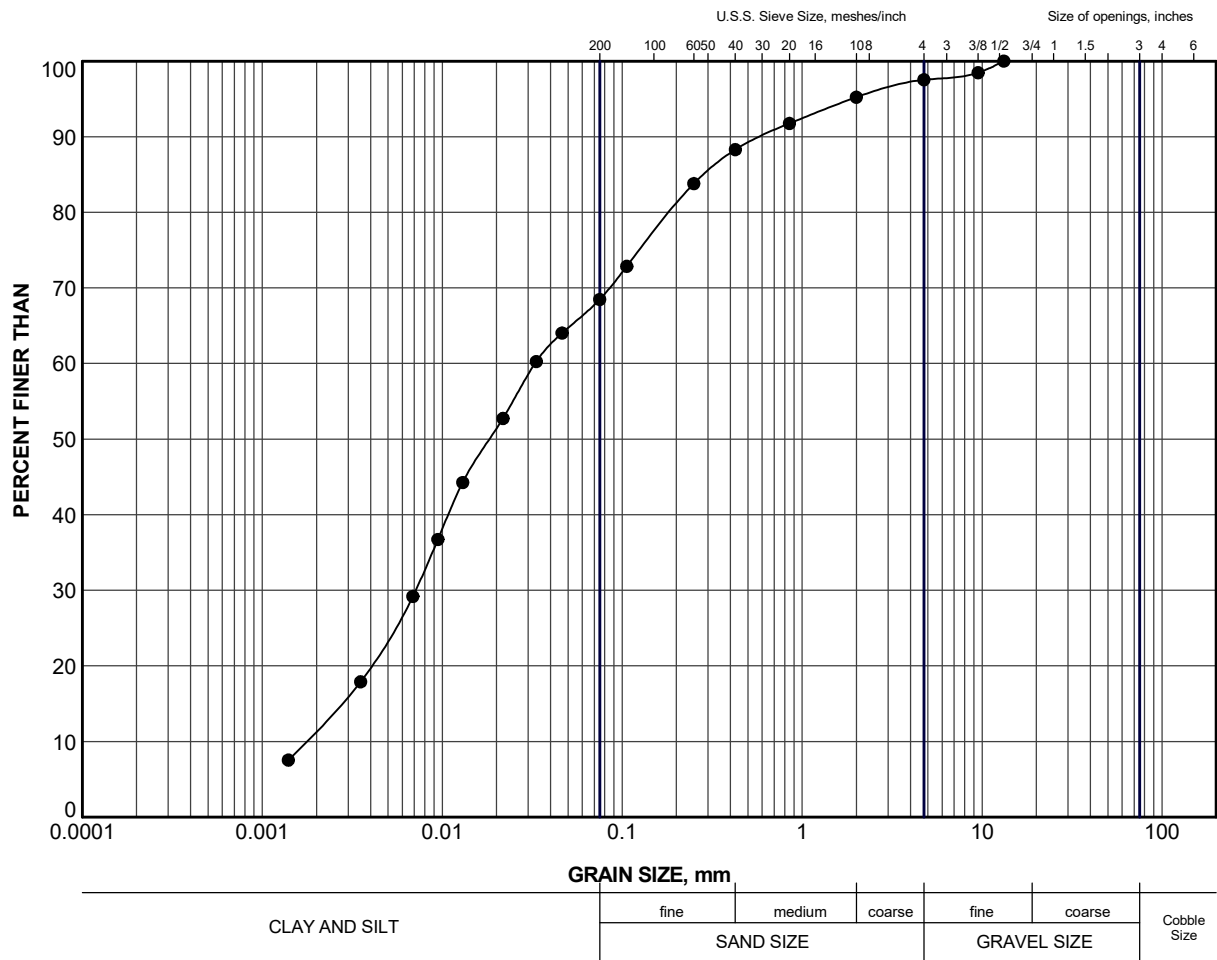


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-02	2	242.9

PROJECT					
MUNICIPAL ROAD 55 CULVERT EXTENSION AT STATION 10+147					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY SAND (SM) and Gravel (FILL)					
PROJECT No.		20253807		FILE No.	
DRAWN		TR		Jul 2021	
CHECK		DAM		Jul 2021	
APPR		KB		Jul 2021	
GOLDER		MEMBER OF WSP		SUDBURY, ONTARIO	
SCALE		N/A		REV.	
FIGURE		B-1			

DRAFT



LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-02	4	241.4

PROJECT

MUNICIPAL ROAD 55  
CULVERT EXTENSION AT STATION 10+147

TITLE

GRAIN SIZE DISTRIBUTION  
Sandy SILT (ML)

 **GOLDER**  
MEMBER OF WSP

SUDBURY, ONTARIO

PROJECT No. 20253807

FILE No. 20253807.GPJ

DRAWN TR Jul 2021

CHECK DAM Jul 2021

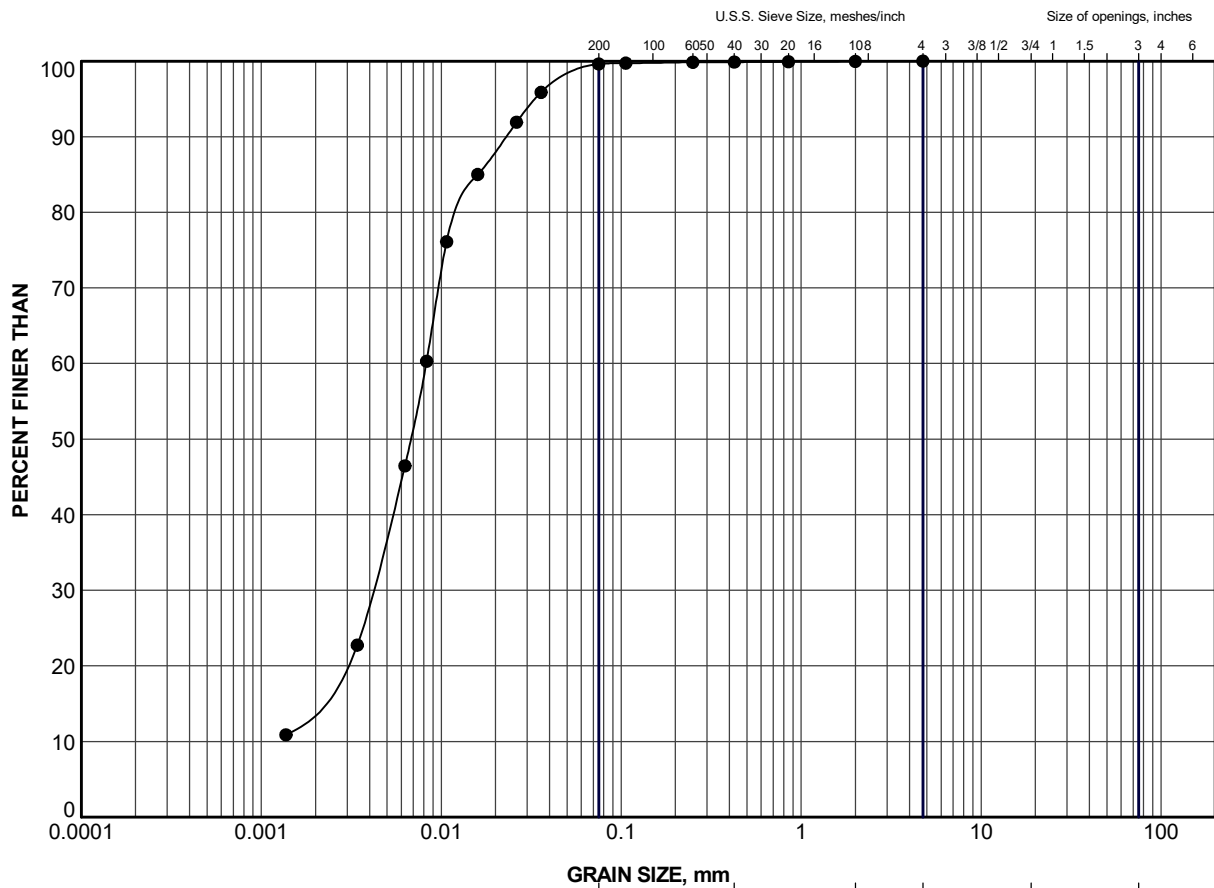
APPR KB Jul 2021

SCALE N/A REV.

FIGURE B-2

SUD-MTO GSD GLDR\_LDN.GDT


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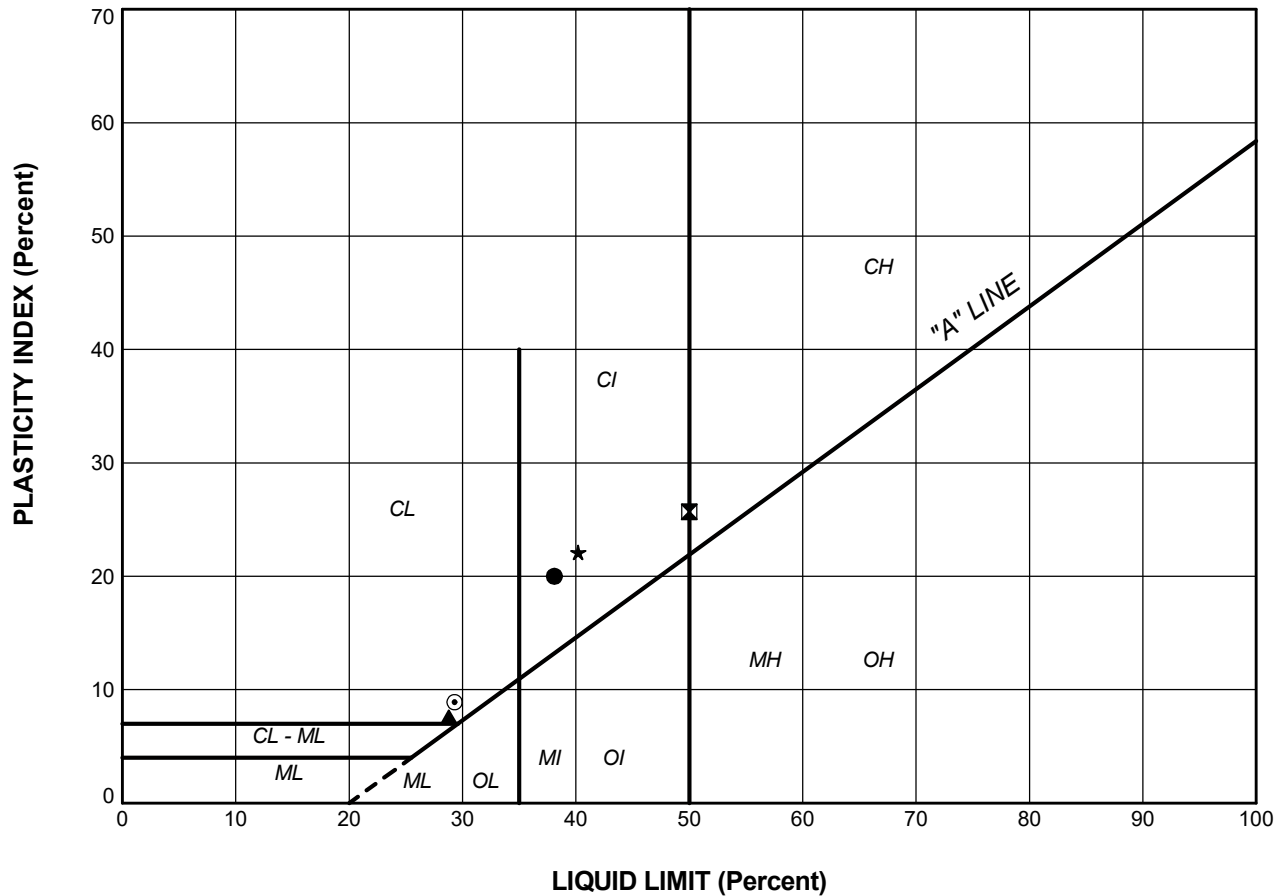
CLAY AND SILT	GRAIN SIZE, mm					
	fine		medium	coarse		
	SAND SIZE			GRAVEL SIZE		Cobble Size

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-01	9	232.3


PROJECT MUNICIPAL ROAD 55 CULVERT EXTENSION AT STATION 10+147										
TITLE GRAIN SIZE DISTRIBUTION CLAYEY SILT (CL)										
 <div><b>GOLDER</b> MEMBER OF WSP</div> <div>SUDBURY, ONTARIO</div>			PROJECT No.		20253807	FILE No.		20253807.GPJ		
			DRAWN		TR	Jul 2021	SCALE		N/A	REV.
			CHECK		DAM	Jul 2021	<b>FIGURE B-3</b>			
			APPR		KB	Jul 2021				

# DRAFT

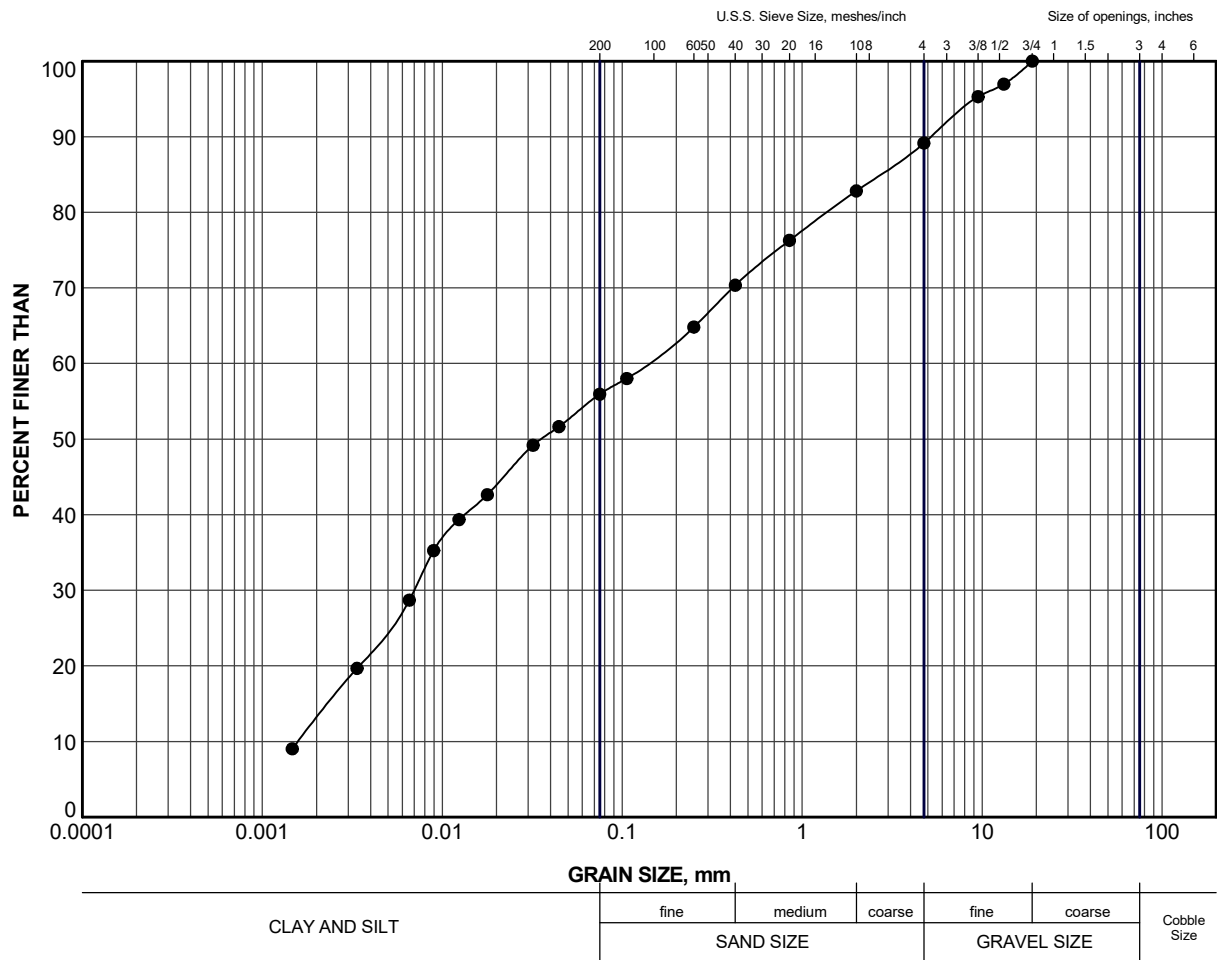


## LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	21-01	5	38.1	18.1	20.0
⊠	21-01	7	50.0	24.3	25.7
▲	21-01	9	28.8	21.2	7.6
★	21-02	8	40.2	18.1	22.1
⊙	21-02	11	29.3	20.4	8.9

PROJECT						MUNICIPAL ROAD 55 CULVERT EXTENSION AT STATION 10+147					
TITLE						<b>PLASTICITY CHART</b> CLAYEY SILT (CL) to SILTY CLAY (CI)					
PROJECT No.			20253807			FILE No.			20253807.GPJ		
DRAWN	TR	Jul 2021	SCALE		N/A	REV.					
CHECK	DAM	Jul 2021	<b>FIGURE B-4</b>								
APPR	KB	Jul 2021									
 <b>GOLDER</b> MEMBER OF WSP						SUDBURY, ONTARIO					


DRAFT



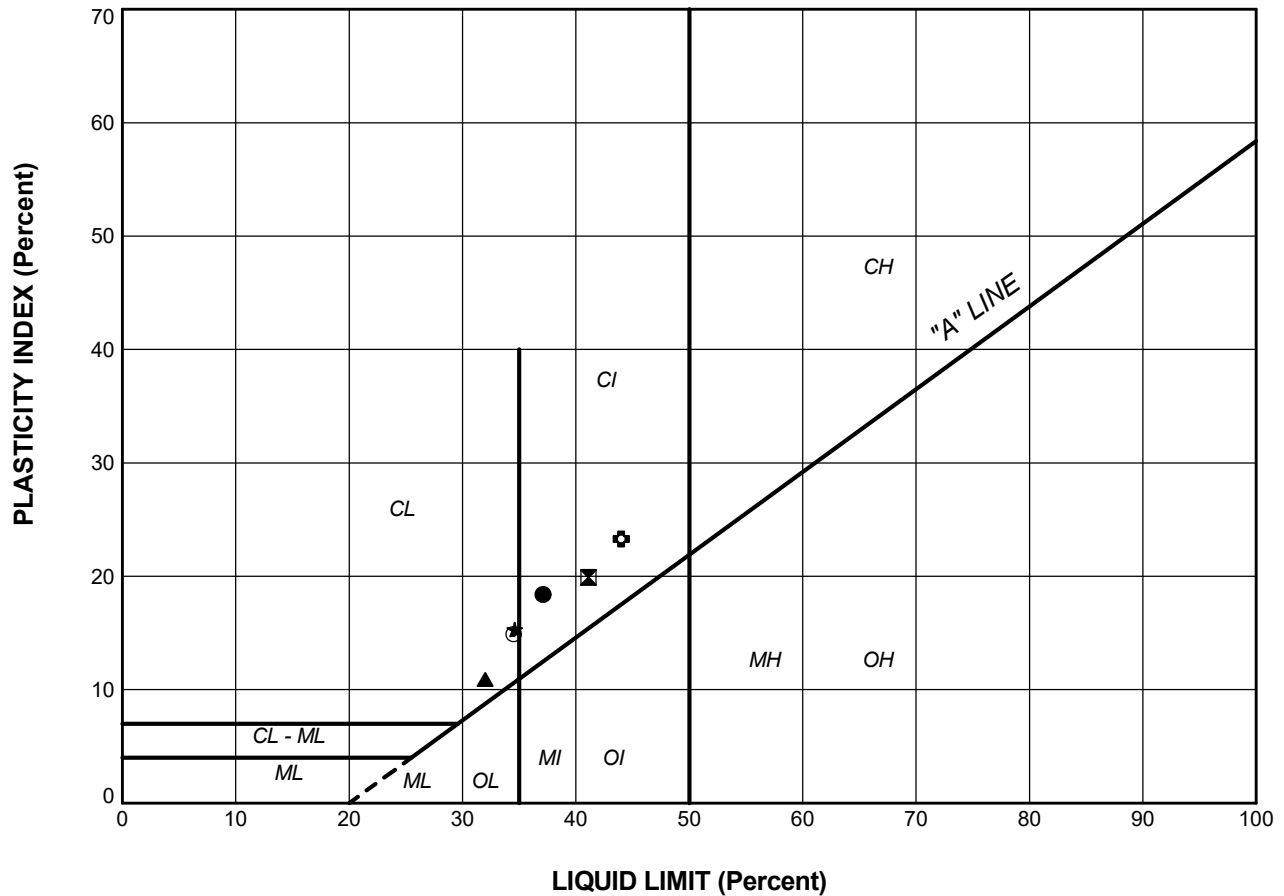
**LEGEND**

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-07	3	241.5

PROJECT						
HIGHWAY 17 EASTBOUND LANE CULVERT EXTENSION AT STATION 14+563						
TITLE						
GRAIN SIZE DISTRIBUTION Sandy SILT (ML) (FILL)						
PROJECT No.		20253807		FILE No.	20253807.GPJ	
DRAWN	TR	Jul 2021	SCALE	N/A	REV.	
CHECK	DAM	Jul 2021	FIGURE B-5			
APPR	KB	Jul 2021				


**GOLDER**  
MEMBER OF WSP  
SUDBURY, ONTARIO

# DRAFT



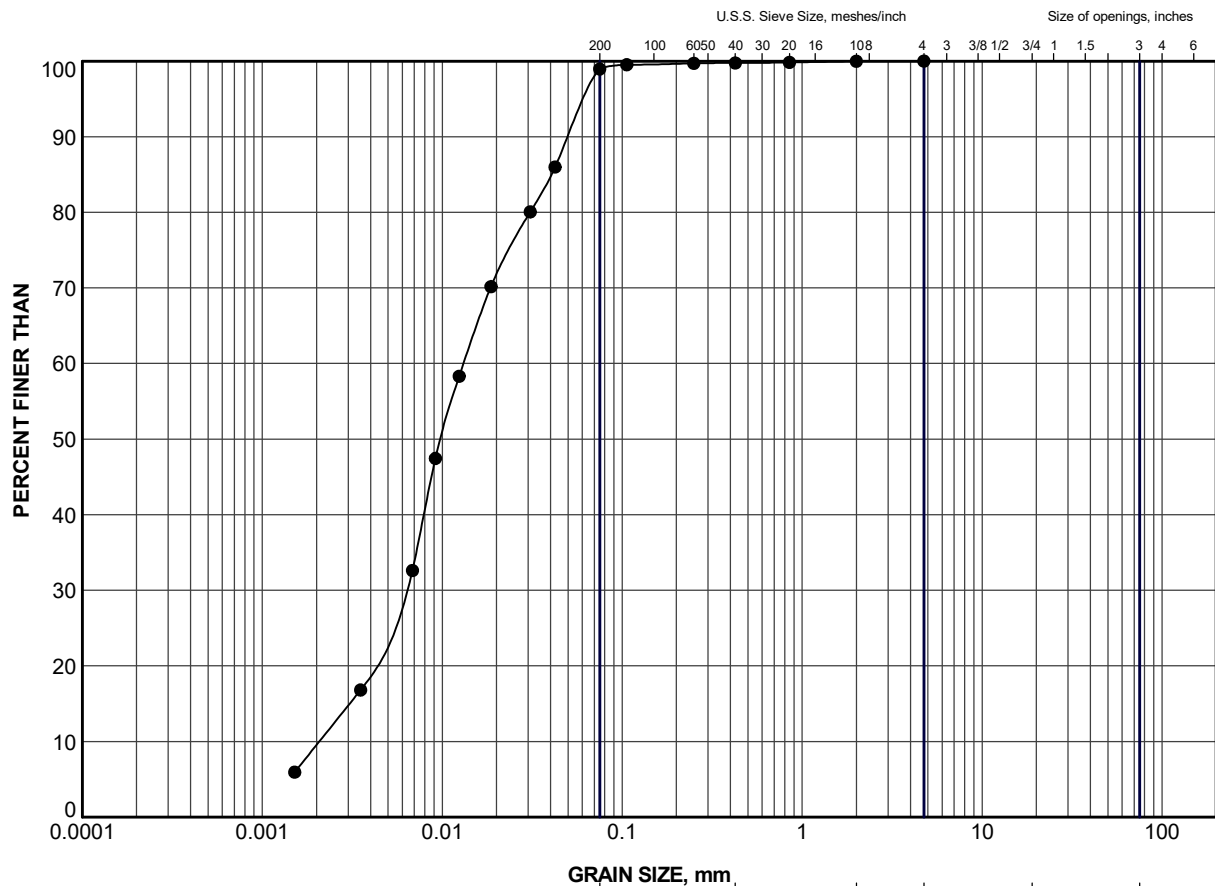
## LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	21-07	7	37.1	18.7	18.4
⊠	21-07	9	41.1	21.2	19.9
▲	21-07	11	32.0	21.1	10.9
★	21-08	3	34.6	19.3	15.3
⊙	21-08	5	34.5	19.6	14.9
⊕	21-08	7	44.0	20.7	23.3

PROJECT				
HIGHWAY 17 EASTBOUND LANE CULVERT EXTENSION AT STATION 14+563				
TITLE				
<b>PLASTICITY CHART</b> CLAYEY SILT (CL) to SILTY CLAY (CI)				
PROJECT No.		20253807		FILE No.
20253807.GPJ				
DRAWN	TR	Jul 2021	SCALE	N/A
CHECK	DAM	Jul 2021	REV.	
APPR	KB	Jul 2021	<b>FIGURE B-6</b>	
 <b>GOLDER</b> MEMBER OF WSP SUDBURY, ONTARIO				



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CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-07	12	230.8

PROJECT HIGHWAY 17 EASTBOUND LANE CULVERT EXTENSION AT STATION 14+563					
TITLE GRAIN SIZE DISTRIBUTION SILT (ML)					
PROJECT No.		20253807		FILE No.	
DRAWN		TR		Jul 2021	
CHECK		DAM		Jul 2021	
APPR		KB		Jul 2021	
SCALE		N/A		REV.	
GOLDER MEMBER OF WSP		SUDBURY, ONTARIO		FIGURE B-7	

## APPENDIX C

# Analytical Laboratory Testing

BUREAU  
VERITASBV Labs Job #: C140122  
Report Date: 2021/02/23

DRAFT

Golder Associates Ltd  
Client Project #: 20253807  
Sampler Initials: TB

## RESULTS OF ANALYSES OF SOIL

BV Labs ID		OVO702	OVO703			OVO703		OVO704		
Sampling Date		2021/02/01	2021/02/08			2021/02/08		2021/02/02		
COC Number		na	na			na		na		
	UNITS	BH21-2 SA#5	BH21-3 SA#7	RDL	QC Batch	BH21-3 SA#7 Lab-Dup	QC Batch	BH21-7 SA#6	RDL	QC Batch
<b>Calculated Parameters</b>										
Resistivity	ohm-cm	580	2000		7201555			1600		7201555
<b>Inorganics</b>										
Soluble (20:1) Chloride (Cl-)	ug/g	980	310	20	7206500			410	20	7206500
Conductivity	umho/cm	1720	502	2	7206535			639	2	7206535
Available (CaCl2) pH	pH	6.65	7.12		7212920	7.07	7212920	6.27		7212920
Soluble (20:1) Sulphate (SO4)	ug/g	49	<20	20	7206511			25	20	7206511
Sulphide	mg/kg	<0.5 (1)	<0.5 (1)	0.5	7211691			<0.5 (1)	0.5	7211691
<b>Physical Testing</b>										
Moisture-Subcontracted	%	11	17	0.30	7211690			24	0.30	7211690
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate (1) Analyzed past method specified hold time Sample contained greater than 10% headspace at time of extraction.										

## APPENDIX D

# Notice to Contractor and Standard Special Provisions

## **EXISTING SUBSURFACE CONDITIONS – Item No.**

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Notice to Contractor

---

The Contactor is alerted to the potential for cobble and boulder obstructions within the embankment fill and native soils as inferred to be present based on instances of auger griding and/or split-spoon refusal as encountered in Borehole 21-02 and 21-07. The extent and depth of obstructions may vary beyond and between the borehole locations.

**DEWATERING SYSTEM - Item No.**  
**TEMPORARY FLOW PASSAGE SYSTEM - Item No.**

Special Provision No. 517F01

November 2016

**Amendment to OPSS 517, November 2016**

**Design Storm Return Period and Preconstruction Survey Distance**

**517.04.01 Design Requirements**

Subsection 517.04.01 of OPSS 517 is amended by deleting the second last paragraph in its entirety and replacing it with the following:

Temporary flow passage systems shall be designed, as a minimum, for a 2 year design storm return period and groundwater discharge, except for the work specified in Table A. For the work specified in Table A, the temporary flow passage system shall be designed, as a minimum, for the design storm return period specified in Table A and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Return period flow estimates are provided in Table A. These estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

**Table A**  
**Return Periods and Flow Estimates**

Site Name / Station Reference	Minimum Design Storm Return Period (Years)	Return Period Flow Estimates (m <sup>3</sup> /s)				Preconstruction Survey Distance (m)
		2 Year	5 Year	10 Year	25 Year	
Culvert on MR55 STA. 10+147 Denison Twp.	**	***	***	***	***	n/a
Culvert on Hwy 17 EBL STA. 14+563 Denison Twp.	**	***	***	***	***	n/a

[\*, \*\*, \*\*\*, \*\*\*\* Designer Fill-Ins, See Notes to Designer]

[\*\*\*\*\* Designer Option - See Notes to Designer]

**NOTES TO DESIGNER:**

**Designer Fill-ins for Table A:**

- \* Fill-in site name, work, and station reference as appropriate for dewatering and temporary flow passage system item locations.
- \*\* For temporary flow passage system item locations only, fill-in the minimum return period for the site based on MTO Drainage Design Standard TW-1. For dewatering system locations, fill-in “n/a”.

- \*\*\* For both dewatering and temporary flow passage system item locations, fill-in the design flow estimates for various return periods.
- \*\*\*\* Fill-in the required distance for preconstruction survey if recommended by the Foundation Engineer. Fill-in “n/a” if not recommended.

**Table A (Sample)**  
**Return Periods and Flow Estimates**

Site Name / Station Reference	Minimum Design Storm Return Period (Years)	Return Period Flow Estimates (m <sup>3</sup> /s)				Preconstruction Survey Distance (m)
		2 Year	5 Year	10 Year	25 Year	
Woods Creek Culvert Rehabilitation	n/a	0.7	3.5	7.5	10.9	100
Brant Drain Lining Rehabilitation	2	0.2	0.6	1.2	1.9	n/a
C/L Culvert Sta. 16+606	2	0.3	1.2	2.7	4.0	n/a
Site 32-145 Robbs Creek Culvert Replacement	10	1.6	7.6	17.4	25.2	250

**\*\*\*\*\* Designer Option**

Insert the following when recommended by the Foundation Engineer:

The dewatering system or temporary flow passage system design for the site(s) / work area(s) listed below shall be completed by a design Engineer and design-checking Engineer, both of whom shall have a minimum 5 years experience in designing systems of similar nature and scope to the required work:

- a) Insert site name / station reference as shown in Table A.
- b) Etc

**WARRANT:** Always with these tender items.

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