

Foundation Investigation and Design Report

Hwy 401 / Sixteen Mile Creek Culvert Replacement, Site No. 10-592/C (Latitude: 43.50832; Longitude: -79.95266); Detail Design for Structural Culvert Rehabilitation/Replacement; Ministry of Transportation, Ontario; G.W.P. 2219-14-00

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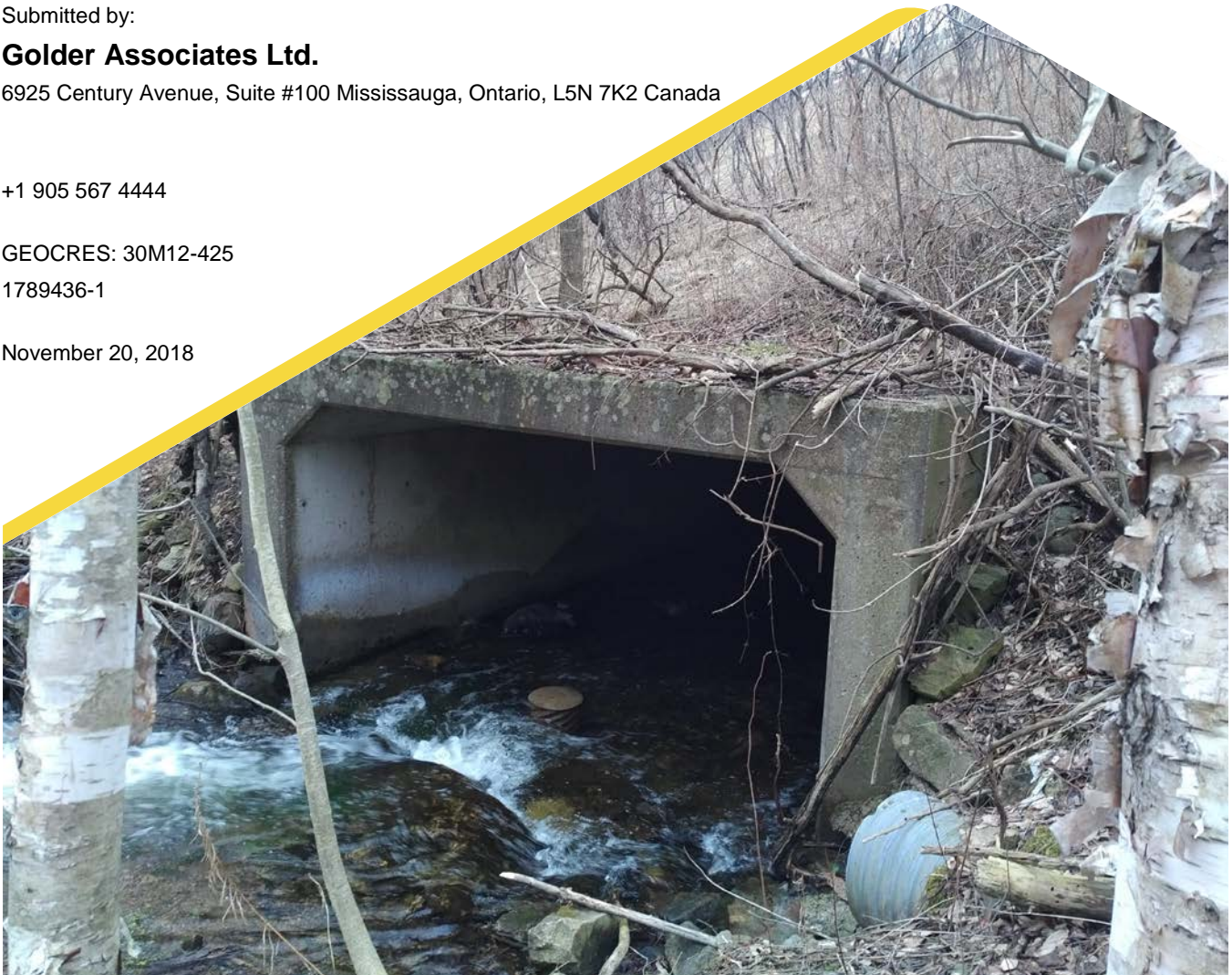
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PART A

**FOUNDATION INVESTIGATION REPORT
HWY 401 / SIXTEEN MILE CREEK CULVERT REPLACEMENT, SITE NO. 10-
592/C (LATITUDE: 43.50832; LONGITUDE: -79.95266);
DETAIL DESIGN FOR STRUCTURAL CULVERT REHABILITATION/
REPLACEMENT;
MINISTRY OF TRANSPORTATION, ONTARIO; G.W.P. 2219-14-00**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AIA Engineers Ltd. (AIA) on behalf of the Ministry of Transportation, Ontario (MTO) to provide Foundation Engineering services for the replacement of the Sixteen Mile Creek structural culvert located at about Station (STA) 17+940 on Highway 401 in the Regional Municipality of Halton (MTO Structural Site No. 10-592/C), approximately 450 m east of Appleby Line in Milton, Ontario (see key plan on Drawing 1).

The Terms of Reference and the Scope of Work for the foundation investigation are outlined in MTO's Request for Quotation, dated September 2017. Golder's proposal for the foundation engineering services associated with the culvert replacement is contained in Section 3.5 of AIA's Technical Proposal for this assignment.

2.0 SITE DESCRIPTION

The existing open box concrete culvert is about 122 m in total length, about 3.1 m wide and 1.6 m high. The upstream and downstream invert elevations of the existing culvert are at about Elevation 238.9 m and 234.5 m. The structure is located below the Highway 401 embankment that is up to approximately 11 m high, with up to about 10.2 m of soil cover above the existing culvert. Details of the existing culvert are summarized in Table 1 following the text of this report.

In general, the culvert is located along the Niagara Escarpment and the topography in the area varies from rolling terrain to nearby exposed bedrock faces (i.e. cliffs) and lakes. The area is generally used for agriculture and recreational purposes with the outlet end of the culvert located within the Region of Halton's Kelso Conservation Area. Sixteen Mile Creek flows through the culvert from northwest to southeast and is formed within a valley incised into the natural terrain and eventually drains into Kelso Lake just beyond the outlet of the culvert. The lands adjacent to the creek are generally well vegetated with trees. The ground surface at the borehole locations advanced for the culvert investigation, including through the existing Highway 401 embankment, varies between about Elevation 234.6 m to 249.4 m, referenced to Geodetic datum. The Highway 401 alignment in the project area is generally oriented in a southwest-northeast direction; for the purposes of this report, the direction of east-west along the highway, and north and south for the upstream and downstream ends of the culvert, respectively have been assumed.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the foundations investigation associated with the structural culvert Site No.10-592/C was carried out between March 11 and April 4, 2018, during which time a total of six boreholes were advanced in the immediate vicinity of the culvert alignment as shown in plan on Drawing 1.

The field investigation was carried out using a variety of drilling equipment due to the varying levels of accessibility and restrictions associated with the terrain at the culvert site. The details of the drilling equipment and suppliers are listed below.

Drilling Equipment	Supplied and Operated By
Truck-Mounted Diedrich D90	Walker Drilling Ltd. of Utopia, Ontario
Portable Equipment	OGS Drilling Inc. of Almonte, Ontario

The boreholes drilled by the truck-mounted D90 drill rig were advanced through the overburden using 210 mm outer diameter (O.D.), 108 mm inner diameter (I.D.) hollow stem augers. The boreholes completed with the portable equipment were advanced through the overburden using BW sized casing (73 mm O.D.) with wash boring techniques utilizing AW (57 mm O.D.) casing with a diamond casing shoe to advance between samples. As a result, the recovery within the AW casing between SPT sampling intervals was collected and placed in core boxes and photographs taken to supplement the investigation results (see Figure 1). Soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m using a 50 mm O.D. split-spoon sampler operated by an automatic hammer on the drill rigs, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Boreholes advanced by portable equipment employed a full-weight hammer lifted manually and dropped from the SPT height.

The results of the SPT testing as presented on the Record of Borehole Sheets are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). The samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 mm. Therefore, particles that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. For the site stratigraphy, these larger sized particles may include glacial erratics such as cobbles and boulders.

A piezometer was installed in Boreholes 18-1 and 18-4 to allow monitoring of the groundwater level at this site. The piezometer consists of a 25 mm diameter PVC pipe, with a slotted screen sealed within the sandy clayey silt and sand deposit in Borehole 18-1 and within the sand and gravel and silty sand till deposits in Borehole 18-4. The borehole and annulus surrounding the piezometer pipe above the screen and sand pack were backfilled with bentonite pellets to ground surface. The piezometer installation details and water level readings are noted on the Record of Boreholes in Appendix A. All other boreholes were backfilled with bentonite upon completion of drilling in accordance with Ontario Regulation 903 (Wells) (as amended). The groundwater conditions and water levels in the open boreholes were observed during and immediately following the drilling operations and are described on the Record of Borehole sheets in Appendix A.

The fieldwork was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined the soil samples. The soil samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO Laboratory Standards and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. Where gravel / cobble nests were encountered and sampled, point load strength index tests were carried out, both perpendicular to the core axis (diametral) and along the core axis (axial), to provide an indication of the point load strength index (Is_{50}) of the rock fragment type. The rock strength classification is based on the Is_{50} values as suggested in Table 3.5 of the Canadian Foundation Engineering Manual (CFEM)¹.

The results of the laboratory testing are summarized in Section 4.2, on the Record of Borehole sheets in Appendix A and on the laboratory test sheets in Appendix B.

¹ Canadian Foundation Engineering Manual. 2006. Fourth Edition, Canadian Geotechnical Society.

Four soil samples were submitted to a specialist analytical laboratory under chain of custody procedures for corrosivity package testing. The results of the analytical testing are provided in Appendix C and summarized in Section 4.4.

Borehole locations and Geodetic elevations were surveyed in the field at the as-drilled location using a GPS (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and 0.1 m in the horizontal directions and/or relative to a fixed benchmark location on site. Where boreholes were surveyed relative to a fixed benchmark location, the as-drilled borehole locations were measured from this point and subsequently converted into MTM NAD 83 (Zone 10) coordinates and elevations obtained from the digital terrain model provided by AIA. The borehole locations, ground surface elevations and depths drilled are provided below.

Culvert Location	Borehole I.D.	Location		Ground Surface Elevation (m)	Depth of Borehole (m)
		Northing (m) / Latitude (°)	Easting (m) / Longitude (°)		
Highway 401 – STA 17+940 (STA 17+884 to STA17+966)	18-1	4,818,779.1 / 43.508038	268,079.0 / -79.954156	240.0	6.1
	18-2	4,818,791.9 / 43.508154	268,085.2 / -79.954070	239.7	5.5
	18-3	4,818,781.0 / 43.508057	268,109.3 / -79.953772	249.4	20.1
	18-4	4,818,812.1 / 43.508339	268,167.6 / -79.953053	247.7	21.6
	18-5	4,818,804.8 / 43.508276	268,208.6 / -79.952545	234.8	9.8
	18-6	4,818,824.6 / 43.508454	268,214.9 / -79.952469	234.6	9.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The section of Highway 401 where Sixteen Mile Creek crosses is located within the Niagara Escarpment physiographic region, as delineated by *The Physiography of Southern Ontario* (Chapman and Putman, 1984)². This area of the Niagara Escarpment is bordered by the Flamborough Plain and the Peel Plain physiographic regions to the west and east, respectively. The Niagara Escarpment region is characterized by its vertical dolostone cliffs. At the culvert location, Highway 401 crosses an old river valley cut through the escarpment. The Peel Plain physiographic region is located to the east of the Niagara Escarpment and is generally characterized by a thin clay deposit overlying till deposits. The Flamborough Plain physiographic region is located on top of the escarpment to

² Chapman, L.J. and Putnam, D.F., 1984: *The Physiography of Southern Ontario*, Third Edition. Ontario Geological Survey, Special Volume 2.

the west of the site. This region has few drumlins and multiple swamps. It is a limestone plain with shallow cover generally consisting of bouldery glacial tills or sands and gravels.

4.2 General Overview of Local Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the Record of Borehole sheets and the laboratory test sheets in Appendices A and B, respectively. The stratigraphic boundaries shown on the Record of Boreholes are inferred from non-continuous sampling, observations of drilling progress and in situ testing and are approximate. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

In general, the soil stratigraphy at the culvert site consists of surficial layers of embankment fill (Highway 401) or topsoil underlain by an upper granular deposit primarily consisting of sand to sand and gravel which is in turn underlain by a non-cohesive to cohesive till deposit. The till deposit is typically underlain by a lower granular deposit on the north side of the site. On the south side of the site, closer to Kelso Lake, the topsoil is underlain directly by a clayey silt deposit which is in turn underlain by a cohesive till deposit. A detailed description of the main soil deposits / stratum encountered during the investigation is provided below.

4.2.1 Asphalt

In Boreholes 18-3 and 18-4 advanced at the Highway 401 grade, an asphalt layer approximately 200 mm to 225 mm thick was encountered.

4.2.2 Topsoil to Sandy Clayey Silt containing Organics

A 0.1 m to 0.6 m thick layer of topsoil in Boreholes 18-2 and 18-6 and sandy clayey silt containing organics in Boreholes 18-1 and 18-5 was encountered at ground surface.

The SPT 'N'-values measured within the surficial organic deposit range between 4 blows and 10 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.

The natural water content measured on two samples of the sandy clayey silt containing organics were about 28 and 29 per cent.

4.2.3 Embankment Fill

Embankment fill, approximately 10.4 m and 11.7 m thick, was encountered below the asphalt in Boreholes 18-3 and 18-4 at Elevations 249.2 m and 247.5 m, respectively. The embankment fill consists of gravelly sand to sand and gravel, with variable amounts of silt. Although not specifically observed in the samples collected, our recent experience with trenchless crossings of major MTO highways suggests that there may be debris consisting of abandoned temporary works associated with the original culvert construction. This debris buried in the fill may consist of logs, stumps, and brush from the clearing and grubbing operations, and cobbles and/or boulders as inferred from auger grinding in Boreholes 18-3 and 18-4 (see Borehole Record for details).

The SPT 'N'-values measured within the non-cohesive fill range between 10 blows and 64 blows per 0.3 m of penetration, indicating a compact to very dense level of compactness.

The natural water content measured on select samples of the non-cohesive embankment fill ranges between about 4 per cent and 20 per cent.

The results of grain size distribution tests completed on two samples of the sand and gravel fill are shown on Figure B1 in Appendix B.

Atterberg limits tests were carried out on two samples of the embankment fill and measured liquid limits of 21 per cent and 22 per cent, plastic limits of 14 per cent and 16 per cent, corresponding to plasticity indices of 7 per cent and 6 per cent. The test results, which are plotted on a plasticity chart on Figure B2 in Appendix B, indicate that the fines component of the material tested is a clayey silt of low plasticity.

4.2.4 Upper Granular Deposit

An upper granular deposit (1.0 m to 3.3 m thick) consisting of sand, sand and gravel, silty sand and gravel and sandy gravel, was encountered below the topsoil / sandy clayey silt with organics layer in Boreholes 18-1 and 18-2 and below the embankment fill in Boreholes 18-3 and 18-4 between Elevations 239.6 and 235.8 m. The upper 1.1 m of the deposit encountered in Borehole 18-2 contains organics. A 0.6 m to 0.7 m thick layer of sandy clayey silt with gravel was encountered within the upper granular deposit in Boreholes 18-1 and 18-2 at Elevations 238.9 m and 238.5 m, respectively. Auger grinding in Borehole 18-4 suggests the presence of cobbles and/or boulders within the deposit and cobble sized rock fragments (between 75 mm and 150 mm) were recovered in the core samples obtained during advancement of Boreholes 18-1 and 18-2 within this deposit (See Figure 1).

The SPT 'N'-values measured within the upper granular deposit range between 7 blows and 136 blows per 0.3 m of penetration, and up to 100 blows per 0.05 m of penetration, indicating a loose to very dense level of compactness. The SPT 'N'-values measured within the sandy clayey silt with gravel layer are 26 blows and 107 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

The natural water content measured on select samples of the upper granular deposit are between about 9 per cent and 17 per cent. The natural water content measured on select samples of the sandy clayey silt and gravel layer were about 11 per cent and 15 per cent.

The results of grain size distribution tests completed on five samples of the upper granular deposit are shown on Figure B3 in Appendix B.

Point load strength index tests (axial and diametral) were carried out on cobble sized rock fragment recovered in the core during advancement of Boreholes 18-1 and 18-2. The corrected point load strength index values (Is_{50}) are shown on the Record of Boreholes in Appendix A. The axial and diametral Is_{50} values of 2.6 MPa and 5.0 MPa (axial) and 1.4 MPa and 2.2 MPa (diametral) were measured, respectively. Based on the laboratory Is_{50} values and the field measurement techniques, the estimated intact strength of the cobble sized fragments tested is classified as medium strong (R3, 25 MPa < UCS < 50 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa).

4.2.5 Glacial Till

A glacial till deposit about 2.1 m to 6.3 m thick consisting of non-cohesive and cohesive till was encountered below the upper granular deposit in Boreholes 18-1 to 18-4 and below the clayey silt deposit in Borehole 18-5. Auger grinding in Boreholes 18-3 and 18-4 suggests the presence of cobbles and/or boulders within the till deposit and cobbles (between 75 mm and 250 mm in size) were confirmed / recovered during coring to advance Boreholes 18-1, 18-2 and 18-5. Photos of the core recovery from Boreholes 18-1, 18-2 and 18-5 are provided on Figure 1 following the text of the report. Boreholes 18-1 and 18-5 were terminated in this deposit after penetrating it for lengths of 3.3 m and 5.5 m, respectively.

Non-Cohesive Glacial Till

Non-cohesive till comprised of silty sand and silty sand and gravel was encountered in Boreholes 18-1, 18-3 and 18-4.

The SPT 'N'-values measured within the non-cohesive till range between 30 blows per 0.3 m of penetration and 100 blows per 0.05 m of penetration, indicating a compact to very dense level of compactness.

The natural water content measured on samples of the non-cohesive till ranges between about 8 per cent and 17 per cent.

The results of grain size distribution tests completed on three samples of the predominantly non-cohesive till are shown on Figure B4 in Appendix B.

Atterberg limits tests were carried out on three samples of the non-cohesive till and measured liquid limits between 15 per cent and 18 per cent, plastic limits between 11 per cent and 12 per cent, and plasticity indices of 4 per cent and 6 per cent. The test results, which are plotted on a plasticity chart on Figure B5 in Appendix B, indicate that the fines component of the material tested ranges from a silt of slight plasticity to a clayey silt of low plasticity.

Cohesive Glacial Till

Cohesive till comprised of sandy clayey silt to gravelly clayey silt with sand was encountered in Boreholes 18-2, 18-3 and 18-5.

The SPT 'N'-values measured within the cohesive till range between 28 blows per 0.3 m of penetration, and 100 blows per 0.05 m of penetration, suggesting a very stiff to hard consistency.

The natural water content measured on samples of the cohesive till ranges from about 7 per cent to 21 per cent.

The results of a grain size distribution test completed on three samples of the cohesive till are shown on Figure B4 in Appendix B.

Atterberg limits tests were carried out on three samples of the cohesive till and measured liquid limits between 16 per cent and 24 per cent, plastic limits between 11 per cent and 15 per cent, and plasticity indices between 5 per cent and 9 per cent. The test results, which are plotted on a plasticity chart on Figure B5 in Appendix B, indicate that the material tested is a clayey silt of low plasticity.

Point load strength index tests (axial and diametral) were carried out on a sample of the cobble sized rock fragment recovered in the core during advancement of Borehole 18-5. The corrected point load strength index values (Is_{50}) are shown on the Record of Boreholes in Appendix A. The axial and diametral Is_{50} values measured were 12.2 MPa and 11.1 MPa, respectively. Based on the laboratory Is_{50} values and the field measurement techniques, the estimated intact strength of the cobble sized fragment tested is classified as extremely strong (R6, UCS > 250 MPa).

4.2.6 Clayey Silt

A 3.7 m and 8.2 m thick deposit of clayey silt was encountered below the topsoil to sandy clayey silt with organics deposit in Boreholes 18-5 and 18-6 and below the till deposit in Borehole 18-4 between Elevation 234.2 m and 228.5 m. The lower 1.2 m of the deposit encountered in Borehole 18-5 is described as clayey silt with gravel. Auger grinding in Borehole 18-4 suggests the presence of cobbles and/or boulders within the deposit and cobbles (between 75 mm and 100 mm in size) were confirmed during coring in Boreholes 18-5 and 18-6. Photos of the

cored soil retrieved from Boreholes 18-5 and 18-6 are provided on Figure 1 following the text of the report. Borehole 18-4 was terminated in this deposit after penetrating it for a length of 2.4 m.

The SPT 'N'-values measured within the clayey silt deposit range from 18 blows to 132 blows per 0.3 m of penetration, and up to 100 blows for 0.13 m of penetration, suggesting a very stiff to hard consistency.

The natural water content measured on samples of the clayey silt deposit range between about 14 per cent and 27 per cent.

The results of grain size distribution tests completed on six samples of the clayey silt deposit are shown on Figure B6 in Appendix B.

Atterberg limits tests were carried out on seven samples of the clayey silt deposit and measured liquid limits ranging between 24 per cent and 31 per cent, plastic limits ranging between 15 per cent and 18 per cent and plasticity indices ranging between about 6 per cent and 16 per cent. The test results, which are plotted on a plasticity chart on Figure B7 in Appendix B, indicate that the material tested is a clayey silt of low plasticity.

4.2.7 Lower Granular Deposit

A lower granular deposit consisting of sand and gravel to sandy gravel was encountered in Boreholes 18-2, 18-3 and 18-6 between Elevations 234.8 m and 225.8 m. Auger grinding in Borehole 18-3 suggests the presence of cobbles and/or boulders within the deposit. Boreholes 18-2, 18-3 and 18-6 were terminated within this deposit after penetrating it for lengths between 0.6 m and 2.4 m.

The SPT 'N'-values measured within the lower granular deposit range between 41 blows per 0.3 m of penetration to 100 blows per 0.1 m of penetration, indicating a dense to very dense level of compactness.

The natural water content measured on select samples of the lower granular deposit are between about 7 per cent and 10 per cent.

The results of grain size distribution tests completed on two samples of the lower granular deposit are shown on Figure B8 in Appendix B.

4.3 Groundwater Conditions

The water level was measured in Boreholes 18-2, 18-3, 18-5 and 18-6 upon completion of drilling operations at depths between 0.1 m and 12.5 m below ground surface, between Elevations 238.7 m and 233.6 m. The water level observed in the open boreholes during and/or upon completion of drilling may not represent the longer-term, stabilized groundwater level at the site.

Standpipe piezometers were installed in Boreholes 18-1 and 18-4 located on the north and south side of the Highway 401 embankment, respectively. The observed groundwater levels in the standpipe piezometers are shown on the Record of Borehole sheets and summarized below.

Borehole	Depth to Water Level (m)	Groundwater Elevation (m)	Date of Measurement
18-1	1.8	238.2	March 13, 2018
	0.8	239.2	April 12, 2018
18-4	12.5	235.2	March 22, 2018

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation. Given the close proximity of the outlet to Lake Kelso, the water level in the lake also has the potential to influence the groundwater level at the culvert outlet. The water level in Lake Kelso is controlled by a dam and as such, the water level in the lake has the potential to change rapidly.

4.4 Analytical Testing of Soil Samples

Analytical testing was carried out on selected soil samples from four boreholes to assess the corrosivity and concrete degradation potential of the soils for the new culvert structure. The analytical test results from the specialist analytical laboratory are presented in Appendix C and are summarized below.

Borehole (Sample)	Depth / Elevation (m)	Parameters				
		Soil Resistivity (ohm-cm)	Soil Conductivity (mS/cm)	Sulphate Concentration (%)	Chloride Concentration (%)	Soil pH
18-2 (SA 4)	2.0 / 237.7	11,100	0.09	<0.01	0.002	8.58
18-3 (SA 16)	15.4 / 234.0	6,250	0.16	0.02	0.003	8.43
18-4 (SA 8)	9.2 / 238.2	2,330	0.43	<0.01	0.026	8.33
18-6 (SA 13)	7.6 / 227.0	3,570	0.28	0.02	0.004	8.13

5.0 CLOSURE

Messrs. Jeremy Lebow and Erik Giles supervised the borehole investigation program. Mr. Matthew Kelly, P.Eng., the project engineer, oversaw the overall field investigation. This report was prepared by Ms. Madison Kennedy, B.A.Sc., and was reviewed by Mr. Matthew Kelly, P.Eng., a geotechnical engineer. Mr. Kevin J. Bentley, P.Eng., an Associate with Golder and Designated MTO Foundations Contact conducted an independent quality control review of this report.

Signature Page

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PART B

**FOUNDATION DESIGN REPORT
HWY 401 / SIXTEEN MILE CREEK CULVERT, SITE NO. 10-592/C (LATITUDE:
43.50832; LONGITUDE: -79.95266);
DETAIL DESIGN FOR STRUCTURAL CULVERT REHABILITATION/
REPLACEMENT;
MINISTRY OF TRANSPORTATION, ONTARIO; G.W.P. 2219-14-00**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the structural culvert at Sixteen Mile Creek under Highway 401 (STA 17+940) in the Regional Municipality of Halton (MTO Structural Site No. 10-592/C) which is to be replaced. The existing culvert is a 3.1 m wide by 1.6 m high open footing rigid concrete structure which is about 122 m long. The culvert is an approximately 106 m long, 3000 mm diameter pipe culvert with upstream and downstream elevations at Elevation 238.9 m and 234.5 m, respectively. At this time the staging for construction and whether the existing culvert will remain in use is unknown.

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

6.1 General

This report addresses potential construction concerns and geotechnical problems associated with installation of the replacement culvert by means of a trenchless method.

Based on the General Arrangement (GA) drawings provided by AIA on May 16, 2018, the proposed culvert will be approximately 107 m long, 3000 mm inside diameter smooth-walled pipe, installed with upstream and downstream invert elevations at about Elevation 238.9 m and 234.5 m, respectively, which is consistent with the existing culverts invert elevations. The Highway 401 alignment in the project area is generally oriented in a southwest-northeast direction; for the purposes of this report, the direction of east-west along the highway, and north and south for the upstream and downstream ends of the culvert, respectively have been assumed. The current proposed design indicates that the additional culvert will be installed as near as about 1 m away from the existing culvert. If space and hydraulic requirements permit, the new culvert should be installed at least two diameters (about 6 m) away from the existing structure. There is an increased risk of striking the existing culvert, its footings, or debris left after the original stripping of ground surfaces and construction with a separation distance of only 1 m, particularly if the trenchless installation is deflected by an obstruction within the granular fill or native deposits.

The contractor should be fully responsible for the selection of the trenchless technology which best fits the Contract requirements and subsurface conditions. All trenchless work should be carried out in accordance with MTO's Non-Standard Special Provision (NSSP), titled "Pipe Installation by Trenchless Method" dated December 2014, a copy of which is included in Appendix D (as modified by the recommendations provided in this report), and by an experienced specialist contractor employing only qualified workers skilled in their trade under the direction of an experienced foreman. The work plan should include a provision for grouting around the outside of any temporary or permanent ground support systems should the need arise. The geotechnical aspects of the contractor's work plan for the trenchless undercrossing should be reviewed by a qualified geotechnical engineer prior to construction.

In general, when crossing beneath highways, trenchless operations should be carried out continuously (i.e., 24 hours per day) from the start until the installation is complete. Continuous operations assist with minimizing risks of equipment becoming bound in the excavation by time-dependent increases in friction and/or adhesion, uncontrolled ground losses, worsening of seepage flows and other critical problems that may occur while the work

area is unattended. Recommendations specific to the methodologies appropriate for this site are provided in the following report sections.

6.2 Anticipated Ground Conditions

Progressing from north to south, the subsurface conditions encountered along the proposed alignment generally consist of loose to compact to very dense sand to sandy gravel embankment fill materials overlying very dense silty sand to silty sand and gravel glacial till, firm and hard clayey silt with sand glacial till and, near the outlet, very stiff to hard clayey silt. The embankment consists of gravelly sand to silty sand and gravel fill. If the existing alignment is maintained, embankment fill will occupy most of the face with the native deposits near and above the culvert and tunnel invert. The presence of cobbles and boulders in the fill and native granular deposits has been inferred from auger resistance/grinding encountered in the boreholes advanced through Highway 401 and have been cored in the boreholes advanced at the culvert inlet and outlet. In addition to cobbles and boulders in the fill, there is the potential for encountering debris in the fill as noted in Sections 4.2.3 and 6.3. Generally, the groundwater level along the tunnel profile is near the culvert invert, ranging between about Elevations 239.2 m and 233.6 m.

The anticipated behavior of the subsurface materials as described above and in Section 4.2 can be classified using Terzaghi's Tunnelman's Ground Classification system as modified by Heuer (1974). The behaviour of the materials within the tunnel alignment is summarized as follows.

Material	Tunnelman's Ground Classification	
	Above Groundwater Level	Below Groundwater Level
Gravelly Sand to Sand and Gravel Embankment Fill	Cohesive Running to Fast Ravelling	Flowing
Sand to Sand and Gravel	Cohesive Running to Fast Raveling	Flowing to Fast Ravelling
Clayey Silt to Clayey Silt with Sand	Firm	Firm
Glacial Till (Silty Sand to Silty Sand and Gravel to Clayey Silt with Sand)	Firm to Slow Raveling	Firm to Fast Raveling

In the absence of dewatering, where the embankment fill and native granular soils are saturated, these will collapse and flow into any unsupported excavation or tunnel face. The native cohesive soils and glacial tills would have a stand-up time ranging from a few minutes to several hours depending on the degree of seepage, disturbance and localized gradation of the deposits. The stand-up time of this materials will likely be unpredictable.

6.3 Subsurface Conditions Significantly Influencing Tunnelling Conditions

Trenchless installations will be primarily affected by five factors associated with the subsurface and groundwater conditions, namely:

- The nature of the embankment fill: the fill along the tunnel alignment is granular and can be excavated with several trenchless methods provided that the appropriate precautions are taken to preserve face stability, control water pressures and seepage, prevent void formation and potential loss of ground.

- Remnants of the original construction buried in the fill particularly along the original ground interface: cobbles are inferred to be present within the fill based on auger resistance and grinding encountered when advancing the boreholes and were cored through in the native deposits encountered at the inlet and outlet. Our recent experience with trenchless crossings of major MTO highways suggests that there may be debris consisting of abandoned temporary works associated with the original construction; logs, stumps and brush from the clearing and grubbing operations; and cobbles and boulders buried in the fill. Such obstructions have the potential to damage/clog/obstruct machinery and halt trenchless operations, particularly if there is no person-access to the excavation face to clear the obstruction.
- Tunnel invert at or near the interface of the embankment fill and native materials: the vertical alignment will generally encounter granular embankment fill and native compact to very dense glacial till. Both the fill and native material are expected to include cobbles and boulders. At the culvert outlet the vertical alignment may encounter native hard/dense to very dense glacial till deposits and hard clayey silt deposits. In addition to the potential for encountering obstructions buried in the fill, the selected trenchless methodology must be adaptable to varying subsurface conditions which can change from flowing/running or raveling in the granular materials to firm to hard cohesive deposits. The selected methods and equipment must provide sufficient face support for the granular materials which could be flowing at the tunnel invert or near the more resistance materials near the invert and into the looser fills. Similar conditions, with hard or very dense soils near the invert and granular materials above, have been known to result in ground losses and excessive surface settlement when using mechanized tunnelling systems.
- Presence of cobbles and boulders: cobbles and boulders should be anticipated in the fill, native sand and gravel and glacial till deposits. The advance of microtunneling tunnel boring machines (TBMs) and jack and bore equipment can be hindered by cobble nests or boulders. When conditions are ideal and a boulder is within the pipe opening, pipe ramming can accommodate boulders up to 90% of the casing diameter. The diameter of the proposed replacement culvert at this site is such that person entry for removal of obstructions may be possible if the tunnelling equipment can permit safe access to the face.
- Groundwater at or near invert level: this groundwater condition is somewhat favourable as compared to groundwater at or above the tunnel crown since it may permit a wider range of trenchless methodologies to be used. However, effective face support for raveling and flowing granular fill and native materials will still be critical for success.

The Contract Documents should contain a NSSP warning the contractor of obstructions within the fills, sand and gravel deposits and glacial tills and the difficulties associated with tunneling along the interface of the embankment fill and the underlying native soils; an example NSSP is provided in Appendix D.

6.4 Suitable Trenchless Methodologies

Several methodologies were considered for execution of this crossing: horizontal auger boring (“jack and bore”), pipe ramming, microtunnelling, hand mining or mechanically-assisted excavation within a shield with jacked pipe or steel liner plate or steel ribs and lagging, and conventional tunnelling with tunnel boring machine (TBM).

6.4.1 Jack-and-Bore

Conventional jack and bore methods involve forming a horizontal borehole through the ground from an entry shaft to an exit shaft by means of a rotating cutting head attached to continuous-flight helical augers and operated within a steel casing. The continuous flight augers transport cuttings from the face to the entry shaft. There are very few

contractors in Ontario with suitable equipment capable of installing 3 m inside diameter pipes or experienced with installing large diameter (>1.8 m) bores. Considering that an approximately 107 m long tunnel/pipe is required, this installation is beyond the upper limit of a typical jack and bore operation and this method is not considered reasonably feasible. The combination of traditional jack and bore systems and the ground conditions at this location are also prone to excessive ground losses that could be problematic for the highway, even if suitable equipment were available. Therefore, this method is not considered further in this report.

6.4.2 Pipe Ramming

Pipe ramming involves the use of a steel casing, inserted from a launch pit, and driven by a pneumatic percussion hammer or a hydraulic jacking system. The leading edge or head of the initial steel casing is fitted with a cutting shoe/band to reinforce the exterior face of the pipe for open-face pipe ramming and reduce friction by using a slight overcut. As the casing advances towards the exit pit, additional lengths of steel casing are welded on to the preceding piece. Bentonite or polymer lubricants may be used to facilitate the advancement of the casing where the soil conditions dictate. The material within the casing is removed by augering after the casing is installed. The accuracy of the line and grade are comparable to the jack-and-bore method.

As noted above for a conventional jack and bore installation, there are very few pipe ramming contractors in Ontario capable of installing a 3 m diameter pipe or who possess large-scale ramming equipment (greater than 2 m in diameter). Since the length of the new culvert is beyond the upper limit of a typical pipe ramming operation, and given the subsurface conditions, use of pipe ramming is not considered feasible for this site.

6.4.3 Microtunnelling

Microtunnelling is a guided pipejacking process which uses a remotely controlled TBM to provide continuous support to the excavation face. Microtunnelling relies on a horizontal jacking force applied to the pipe to propel the remotely controlled microtunnel boring machine (MTBM) along with the pipe string through the ground. The pipe is typically installed while the bore is being advanced and serves both as temporary ground support and the final culvert. Specially designed jacking pipe made from steel, glass fibre reinforced plastic (GFRP) or reinforced concrete, and capable of transferring the jacking forces from the jacking reaction frame in the shaft to the MTBM, will be required. Entry and exit shafts are required for microtunnelling operations. Dewatering will be required only at the shafts since MTBMs can operate in saturated soils below the groundwater level. Microtunnelling is typically able to maintain high accuracy ($\pm 25\text{mm}$) with line and grade control. The design diameter of the tunnel for this project is at the upper end of the range for available microtunnelling equipment and this size is somewhat atypical for microtunnelling in Ontario.

There are two main types of microtunnelling TBMs (MTBMs), classified by the mechanisms for providing support to the cut face and transport of cuttings to the entry shaft/surface. While some specialized rotating cutting heads are manufactured to be fitted to the lead end of the pipe with cuttings transport via continuous flight augers these do not provide controllable and pressurized support to the cut face and are not considered microtunnelling TBMs in this report. A slurry MTBM has a full-face rotating cutting head with openings through which the spoil enters a pressurized slurry chamber behind the head. The slurry is used to balance the earth and water pressures and convey cuttings away from the face. Typically, MTBMs are limited to passing boulders one third of the machine diameter, but some models have a crusher chamber which breaks down the obstructions to a size which can be pumped with the cuttings. However, the volume or size of the boulders may be such that the capacity of the crusher could be exceeded resulting in either abandonment of the bore or advancement of a rescue shaft to remove the obstructions and permit resumption of tunnelling and such crushing mechanisms should not be relied upon for penetrating through cobbles and boulders. Managing cobbles and boulders is expected to also require disc cutters

to be mounted to the face of the MTBM along with face openings that are a size compatible with passing the cuttings through the MTBM spoils handling systems. If woody debris is encountered in the fill, it will likely clog the machine, also necessitating a rescue shaft and possibly repairs.

In general, an overcut is typically utilized to reduce frictional forces along the pipe string, reduce jacking forces, and facilitate steering. The overcut should be minimized by selection of an excavated diameter which results in a total annular space between the excavated hole diameter and outer diameter of the jacking pipe no greater than 40 mm (i.e., difference in diameter). The annulus between the outside of the pipe and the ground should be immediately filled with bentonite slurry of an appropriate viscosity using lubrication ports in the pipes installed at regular intervals.

Care during operation is required to maintain adequate support at the face of the MTBM. In highly permeable deposits where the groundwater level is below the obvert there is the potential for loss of the slurry away from the machine face. The bentonite-based slurry should be appropriately formulated, using suitable polymers and additives if necessary for the anticipated ground conditions to prevent fluid loss and maintain a stable ground condition at the face and around the MTBM. When installing the culvert in the granular fill and native sand to sandy gravel deposits, the slurry should be sufficiently viscous, with a Marsh funnel³ viscosity typically between 60 and 80 seconds, to create a “filter-cake” to support the granular material and fill the annular gap. A seal will be required to close the annular space between the wall of the entry/exit shaft and the shield and pipes to retain soil behind the temporary shoring and stop backflow of the slurry into the pits.

The MTBM should be equipped for mixed face conditions. The selected cutting tools and methods should be compatible with variable ground conditions, including cobbles and boulders and layers of granular materials. In this case, “mixed face” relates to the combination of layered soil and fill deposits as well as the presence of cobbles and boulders. Where very dense or hard soil layers occupy the lower portion of the cut face and granular materials exist within the upper part of the face and immediately above the crown, these conditions can be susceptible large ground losses and excessive settlement. Properly selected rock disc cutters should be used to break cobbles and boulders and potentially cut the glacial tills, if encountered, at the face into smaller enough fragments to pass through the apertures in the face. Only closed-face machines equipped with rock cutters should be used at this site. In addition to cobbles and boulders, the Contractor’s work plan should include a method of dealing with debris in the fill materials.

The fill deposits along the tunnel alignment are generally less dense than the native deposits above or below the invert. Noting that the granular fill and native sand and gravel deposits contain cobbles and boulders, the Contractor should be prepared for steering difficulties, deflection of the machine and increased wear or damage to the cutters or cutter housings due to high impact forces.

Person-entry to remove obstructions is generally not available for MTBMs. Even if equipped with a crusher chamber and rock cutting tools, the MTBM could be stopped by large boulders or other debris. Given the potential for encountering obstructions in the embankment fill and native sand and gravel deposits there is a high to very high risk of these obstructions either impeding or halting the machine. For these reason, a microtunnelling installation at this site is considered only marginally feasible.

³ ASTM D6910/D6910M, Standard Test Method for Marsh Funnel Viscosity of Clay Construction Slurries. American Society for Testing and Materials, West Conshohocken, PA.

6.4.4 Manual or Mechanically-Assisted Mining

Manual mining or mechanically assisted excavation within a shield with jacked pipe, steel liner plate, or steel ribs and lagging is considered a feasible method for the culvert installation, provided that ground water is adequately controlled and an appropriate shield is selected. In this method, the tunnelling process is carried out by removing excavated soil from the front cutting face and installing a liner to form a continuous ground support structure. The liner may be installed using a two-pass system or a single pass where the culvert pipe is jacked in during excavation and provides both temporary and permanent support. With a two-pass system, a conduit is installed between the entry and exit shaft by first installing a segmental temporary or primary liner. Once the full length of the primary liner has been constructed, a permanent or secondary liner is installed. The primary liner may consist of steel ribs and wooden lagging or steel liner plates. The secondary liner is typically of cast-in-place concrete construction but may be a smaller primary conduit (carrier) pipe of any suitable material. If the carrier pipe option is used, the annulus between the primary and secondary liners is grouted. The soil may be excavated using hand mining or mechanically-assisted techniques and shields that include partitioned face openings to allow closing of some parts of the face while others are excavated.

In hand mining, excavation is conducted at the face using picks, shovels, or pneumatic hand-held tools. Mechanically-assisted excavation is accomplished by using special shields equipped with power excavation devices. Such soil cutting devices can be rotary cutter booms mounted on the front of the shield, modified hydraulic backhoes, or rotary boom cutters. The soil excavation rate of open-face mechanical excavation is much faster than that of hand mining. This method is adaptable to the variable ground conditions along the culvert alignment, provided groundwater is controlled. Dewatering at the entry and exit shafts will likely be required. Due to the potential presence of sand to sand and gravel to gravelly sand layers at and near the invert level which may be saturated, the contractor's selected equipment and methods must provide effective control of the stability of the face. As sand and gravel fill is present above the springline, fore-poling or spiles driven into the ground ahead of the face will be necessary to prevent raveling and cohesive running and improve support for the tunnel crown. Use of a hooded shield where the top of the shield extends beyond the invert by providing an angled profile to the leading edge of about 60 degrees from the horizontal is recommended. This angle must be measured from the top of the shield to the invert. Since the tunnel will be advanced through granular materials, the shield should have doors which can close off the entire face and retractable breast plates or horizontal bench plates to provide face support in these areas.

Seepage should be expected from the granular fill and native deposits particularly after heavy precipitation events. The sand to sand and gravel to sandy gravel deposits within approximately 1 m of the invert are expected to be saturated. It may be necessary to use horizontal lances/pipes or vacuum well points to control groundwater and prevent ground loss in these areas.

Over excavation can lead to ground losses particularly where the granular fills extend from the obvert to the pavement, as is the case at this culvert site. Any overcut beyond the outside diameter of the shield should be limited to a maximum of 15 mm within the granular materials. It is important that care be taken with the installation of the primary liner to control ground and pavement settlement.

The alignment is near the interface between the embankment fill and native materials. As such, the jacking forces will be variable and high jacking pressures and difficulty maintaining line and grade if cobble nests, boulders or other obstructions are present. For each separate jacking stage for jacked pipe, it will be necessary to use varying hydraulic pressures and travel movements to improve or correct steering.

Lubricants may be used if and where high jacking pressures are encountered. The use of bentonite-based lubricants is recommended due to the predominance of granular materials. The spacing and number of grout ports should be optimized to result in even distribution of lubricant over the entire length of pipe and facilitate post-installation grouting of the annulus, if necessary.

Face access using this manual or mechanically-assisted tunnelling approach facilitates removal of cobbles, boulders and obstructions in the fill if encountered. The appropriate health and safety precautions associated with confined spaces as outlined in the current Ontario Regulation 213 in the Occupational Health and Safety Act (OSHA) must be observed by the contractor.

6.4.5 Conventional Tunnelling with Tunnel Boring Machine

The culvert may be installed using conventional tunnelling techniques and a tunnel boring machine (TBM). Since the ground conditions require continual support of the face only closed-face tunnelling shields that provide support to the face to balance groundwater and earth pressures should be employed. This method, while it should be feasible for the culvert installation at this site, it may be impractical and not cost effective. Conventional tunnel boring machine systems generally exhibit good accuracy with line and grade performance comparable to microtunnelling, provided that the TBM is equipped with jacking systems that allow for alignment adjustments. Dewatering will likely be required at the entry and receiving shafts. While the design diameter of the tunnel for this project is well within the range for commonly-available tunnelling equipment, the relatively short length of tunnel may render this method not as cost effective as others.

Depending on the contractor's available equipment and experience, the size of this installation allows for relatively small diameter conventional TBMs to be used (i.e., operated by personnel stationed within the machine and tunnel). In this case, face control and cuttings transport may be accomplished using slurry methods or using "earth pressure balance" (EPB) technologies in which the mixed groundwater, cuttings and other additives are contained under pressure in a chamber immediately behind the cutting head. Discharge from the chamber is controlled either by pressure relieving gates (doors that open at pre-set pressures or loads) or a screw conveyor to remove materials from the chamber at rates that maintain specified pressures within much of the excavated chamber. Both of these systems should be acceptable for this project. A TBM selected to provide EPB support to the face using pressure relieving gates may provide the most adaptable system for providing face pressure as well as being able to pass small boulders entirely through the TBM. Screw-conveyor systems in EPB machines of the size range necessary for this project can readily become jammed with cobbles and boulder fragments. A slurry TBM, directly analogous to the MTBM systems described above, may be more advantageous given the presence of granular materials along the entire alignment; however, access to the face for obstruction removal may be challenging as for MTBMs. Due to the high permeability of the sand and gravel deposit, extensive soil conditioning (e.g., addition of bentonite, foam, or polymers) may be required if EPB machines are used. Some TBM systems are promoted as being "earth pressure balance" when they do not actually achieve the goals of the EPB technology. Such unacceptable systems rely only on doors that close the face or rely on jacking forces being transmitted to the steel sections of the machine face where this is then interpreted as "face pressure" And do not include a secondary bulkhead and controlled muck discharge rates and pressures. Such systems should be prohibited for this project since they could result in significant ground losses and the consequential highway safety risks and claims. .

The machine must be equipped with hardened cutters, roller bits, and disc cutters to handle compact to very dense sand and gravel fill and native deposits containing cobbles and boulders. The face opening size for any TBM should be restricted so that rock pieces do not pass through the cutterhead and block screw conveyor or slurry systems.

The TBMs could become jammed or clogged with wood and/or cobbles and boulders. The contractor should be prepared to deal with obstructions in the granular fill and native sand to sandy gravel deposits.

Face pressures and cuttings discharge volumes and/or weights should be constantly monitored. Overexcavation above or ahead of the pipe/casing should be strictly avoided to maintain face stability. The overcut should be minimized as described above for MTBM and manual or mechanically-assisted tunnelling. Slurry viscosity and pressure and EPB face pressure should be selected to maintain a face pressure no less than the active earth pressure and groundwater pressure at the elevation of the pressure sensors. If over excavation or ground losses occur, the annulus between the outside of the pipe and the ground and any voids at the face should be immediately filled with bentonite slurry of an appropriate viscosity of no less than about 60 seconds (as described above).

The selected equipment, face tools and methods must be able to adapt to changing ground conditions which include the presence of flowing, running or ravelling granular materials. Alignment may be affected by the presence of soils containing cobbles and boulders or other obstructions associated with embankment construction near the invert.

6.5 Comparison of Tunneling Methods

Trenchless construction methods described in Section 6.4 include various advantages and disadvantages depending on soil conditions, depth of cover, vertical and horizontal alignment, length of pipe installation, cost and availability of equipment, and carry varying levels of risk of successfully completing the installation. The advantages, disadvantages and relative costs and risks are compared in Table 2, following the text of this report.

Conventional “jack and bore” and pipe ramming are not feasible methods due to the pipe diameter and length. Microtunnelling is marginally feasible since the MTBM may be stopped by woody debris in the fill, cobble nests or boulders. The most feasible methods of installing the relief culvert are manual mining or mechanically assisted excavation within a properly designed and selected shield with appropriate groundwater control or conventional tunneling using a TBM fitted with pressure-relieving gates for maintaining face pressures. These methods are more adaptable than microtunnelling or slurry TBM with respect to obstruction removal since person-entry is possible to facilitate removal of obstructions. Manual mining and mechanically assisted excavation is the preferred technical alternative from a foundations engineering perspective since it requires the least work area and support equipment, is less costly compared to use of MTBMs or TBMs, and is readily adapted to the changing ground conditions so long as groundwater can be adequately controlled.

6.6 Entry / Exit Shafts – Temporary Excavation and Groundwater Control

The trenchless methods under consideration require entry and exit pits as part of the tunnel installation. Temporary excavations may be carried out using open cut methods. All excavation work should be carried out in accordance with the Occupational Health and Safety Act and Regulations (OSHA), with local regulations and as outlined in OPSS 902 (Excavating and Backfilling – Structures). The non-cohesive fill and properly dewatered native granular deposits are classified as Type 3 soils. The native glacial till and cohesive deposits above the groundwater level or properly dewatered are considered Type 2 soils. Saturated granular fills and native soils would be Type 4 materials. Excavations at the pits are expected to proceed below the measured groundwater level of Elevations 239.2 m and 234.7 m at the inlet and outlet, respectively.

Dewatering will be required at the entry and exit pits. Groundwater control using sumps may not be adequate in the saturated sand to sand and gravel to sandy gravel and granular till deposits. It may be necessary to use vacuum

well points, eductors or the like. The groundwater level should be lowered to at least 0.5 m below the base of the pit. The excavations should be protected from ingress of surface water. Groundwater control / management for the entry and exit shafts should be carried out in accordance with OPSS.PROV 517 as amended by Special Provision No. 517F01, a copy of which is included in Appendix D. A supplementary NSSP should be included in the Contract Documents to alert the Contractor for the need for effective dewatering and control of surface water; an example NSSP is included in Appendix D. Provided proper groundwater control is in place, conventional temporary type open cuts may be developed with side slopes not steeper than 1 horizontal to 1 vertical (1H:1V) in these materials. Stockpiles of excavated material should be set back from the edge of the excavation by a distance at least equal to the excavation depth. Cobbles may be encountered within the fill and native deposits.

The entry and exit pits will be located near the toes of the highway embankment and must be designed to provide continuous support to preserve the stability of the embankment side slopes. Depending on the topography at the embankment toe, construction of a fully enclosed shaft may not be possible. The heading may have to consist of a cantilevered wall. If the culvert is to be installed using jacking forces for pipe installation or TBM/MTBM propulsion, it would be necessary to incorporate a thrust reaction system into the entry shaft where the reaction frame could consist of drilled piles or ground anchors to transfer the horizontal reaction loads to the ground. The reaction frame or thrust block must be designed to accommodate the high jacking forces required to push the pipe segments through the ground for the entire length of the alignment. Design and planning for the working space at the entry and exit shafts should consider site constraints such as the south end of the culvert being located in a conservation area, a temporary yard nearby may also be required.

The base of all entry/exit shafts should be designed to resist the loading associated with the weight of the pipe and selected tunneling equipment. It should be covered with a geotextile (such as an OPSS 1860 (Geotextiles) Class II with a Fabric Opening Size (FOS) less than or equal to 212 μm), a minimum 300 millimetre thick layer of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II material and a 100 millimetre thick concrete working slab, with concrete having a 28-day compressive strength not less than 20 MPa.

The shafts could be constructed using soldier piles and lagging or a secant pile wall, corner braces and angled braces and raker footings given that the opening will likely only have three sides. Due to the presence of very dense/hard glacial till as well as cobbles and boulders it will likely be difficult to install sheet piles. Steel H-piles for soldier piles should be installed in pre-drilled holes. The use of trench boxes and any system which does not provide continuous support to the excavation walls should be prohibited.

The temporary excavation support system should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539.

The design of internally braced soldier pile and lagging walls or other temporary support systems is the responsibility of the contractor. For design considerations, the system design should be based on trapezoid-shaped apparent earth pressure distributions using the design parameters given below. Where the support to the wall is provided by anchors or rakers, the wall design should be based on conventional active and passive earth pressure distributions using the design parameters given below. The internal bracing or raker/anchor supports must be designed to accommodate the loads applied from earth pressures and surcharge pressures from area, line or point loads as well as the effects of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using conventional passive earth pressure distribution acting over an equivalent width equal to three times the soldier pile socket diameter provided that the soldier piles are separated by more than three times the socket diameter.

For granular fill, the unfactored trapezoidal earth pressure distribution (p in kN/m^2 ; constant with depth), can be calculated as follows

$$p = 0.3\gamma H_T + q$$

where H_T = the total height (depth) of the excavation (in m)
 γ = soil unit weight (in kN/m^3)
 q = surcharge for traffic and other loading (in kN/m^2)

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly. The above pressure distributions are the minimum pressures for the ultimate stress condition. A stiffer design may be required than predicted by these distributions in order to maintain displacements within an acceptable range.

For cantilevered walls, the unfactored triangular earth pressure distribution (p in kN/m^2 ; increasing with depth), can be calculated as follows:

$$p = K_a (\gamma H - u + q)$$

where K_a = active coefficient of earth pressure
 γ = soil unit weight (in kN/m^3)
 H = the height of the excavation at any point (in m)
 u = porewater pressure (in kN/m^3)
 q = surcharge for traffic and other loading (in kN/m^2)

As noted above, the selection and design of the protection system is the responsibility of the Contractor. The support systems may be designed using the following parameters:

Soil Type	Coefficient of Lateral Earth Pressure			Internal Angle of Friction (Degrees)	Unit Weight (kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p		
Sand and Gravel (Fill)	0.31	0.47	3.3	32	21.0
Clayey Silt to Clayey Silt with Gravel	0.36	0.53	2.8	28	19.0
Sand and Gravel to Sandy Gravel	0.31	0.47	3.3	32	21.0
Sand	0.33	0.50	3.0	30	20.0
Silty Sand to Silty Sand and Gravel (Till)	0.28	0.44	3.5	34	22.0
Clayey Silty with Sand (Till)	0.28	0.44	3.5	34	22.0

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be calculated in accordance with Figure 3 in Section 2 of Chapter 3 in NAVFAC (1986). A stiffer design may be required than predicted by these distributions to maintain displacement within an acceptable range.

Following construction, the entry and exit shafts should be reinstated to existing conditions by backfilling with suitable earth fill placed and compacted in accordance with OPSS 902 (Excavating and Backfilling Structures).

6.7 Settlement Estimates and Monitoring

6.7.1 Settlement

Settlement above tunneled or trenchless installations are typically described as exhibiting the shape of an inverted normal probability distribution curve ("bell curve") with the maximum settlement at the centreline of the trenchless installation, tapering to near zero at some distance from the centreline. The ground surface settlement troughs above the culvert pipe are estimated to extend about 3 m to 3.2 m on each side of its alignment for the proposed 3.0 m diameter culvert. The estimated ground surface/pavement settlement directly above the replacement culvert may be as much as about 25 mm assuming that the settlement trough unit volume is equivalent to about 1% of the tunnel unit volume per running tunnel metre (i.e., "volume loss") provided that sufficient face pressures are applied by EPB or slurry TBMs and that groundwater is fully controlled, a hooded shield and fore-poling or spiling are used for manual or mechanically-assisted excavation systems. The estimated settlements assume that the tunnelling method and equipment are properly selected with good quality work carried out by an experienced contractor and exclude low-probability excessive settlement events from uncontrolled ground losses (if any). Settlement monitoring of the Highway 401 undercrossing should be carried out prior to, during and after the trenchless installation to assess any effects of the work on the highway. In addition, the undercrossing construction should be independently monitored by qualified geotechnical personnel.

6.7.2 Instrumentation

Consistent with MTO requirements, a monitoring program utilizing a combination of settlement monitors/points consisting of settlement points (SP) and surface settlement markers (SSM) is recommended. The standard MTO requirements have been slightly modified to minimize the requirements for lane closures during monitoring after the initial installation of the monitoring points. The SSM monitoring points could consist of hexagonal head bolts embedded in the pavement and marked with paint and/or flagging tape provided the bolts and flagging do not adversely affect traffic safety and the surveying can be carried out with sufficient precision (± 2 mm). The proposed locations of the settlement monitoring points and installation details are shown on Drawing 2.

A total of 24 surface settlement markers (SSMs) and 10 settlement points (SPs) should be installed to measure settlement. The SSMs should be positioned on the culvert centre line and offset 2.0 metres from the centreline in both directions and be arranged in 8 arrays oriented perpendicular to the undercrossing and be installed within the paved roadway, median and shoulder and along the lane marking lines. A total of 10 deep settlement points (SP) should be installed along the undercrossing centerline near the north and south ends of the bore path and an array of three (3) settlement points (SP) at each shoulder of Highway 401. The deep settlement monitors should consist of a steel rod anchored at a depth below the depth of frost penetration which for this site is 1.4 m, as interpreted from OPSD 3090.101 (Frost Penetration Depths for Southern Ontario) with the bottom 0.2 metres grouted in place. A riser pipe should be provided to separate the rod from the surrounding ground.

The monitoring point installations should be carried out under the direction of qualified geotechnical personnel and the subsequent survey monitoring should be carried out by the Contract Administrator or a licensed surveyor

retained by the Project Team with the results being promptly reviewed by a qualified geotechnical engineer on an ongoing basis. Consideration should be given to remote (off-road) monitoring of the settlement points, thereby reducing traffic impacts associated with lane closure requirements.

6.7.3 Establishment of Baseline Conditions

A condition survey to document the existing pavement condition should be carried out prior to the start of construction. The condition survey should include notes and locations about visible flaws such as cracks, distortions, and deviations, heaves, and depressions. A baseline survey of the monitoring array should be carried out at least three times, prior to construction, with the points referenced to two independent benchmarks. Anomalous readings should be rechecked and/or discarded, as necessary. Acceptance of the baseline survey by all parties should be acknowledged in writing.

6.7.4 Monitoring Frequency

Monitoring should be carried out at least once per shift or three times daily during the construction period (whichever is more frequent), and daily during temporary work stoppages, non-operational periods and weekends. More frequent readings will be required if anomalous conditions are encountered or alert levels are being approached or exceeded. Once construction has ceased, monitoring should continue weekly for the first month and monthly until such time at which all parties agree that further movement has stopped. The monitoring data should be evaluated to see if the magnitude of any movements detected during construction warrant continued monitoring beyond a period of two months following the crossing installation. Anomalous readings should be rechecked and/or discarded, as necessary.

The monitoring frequencies recommended above may be increased if review of alert or review levels are exceeded. If a total recorded movement of 10 millimetres (Review Level) relative to the baseline readings is achieved, this should trigger a review of the contractor's methods including construction rate, sequence and ground stabilization methods with a view to arresting excessive movements. An Alert Level of 15 millimetres relative to the baseline movements should require cessation of construction. At this point, implementation of pre-planned settlement mitigation measures is required to assure public safety and maintain traffic flow.

6.7.5 Data Collection and Transfer

Data collection and transfer procedures should be developed in consultation with the MTO. The contractor should be responsible for the supply, install and baseline of the settlement monitoring points/markers. Ongoing review of the data should also be the responsibility of the contractor and prime consultant and qualified geotechnical personnel (RAQs approved *High Complexity Consultant*) should be contacted for technical support with Review and Alert Levels are exceeded.

6.8 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete pipes and headwalls, steel pipes and reinforced steel and other concrete elements buried in the soil. The long-term performance and durability of the structures are directly related to their respective corrosion resistance. Generally, the corrosivity of a structure depends on the soil resistivity, hydrogen ion concentration, salts (chloride and sulphate) concentrations and redox potential. The analytical results for a single composite sample are presented in Section 4.4 and included in Appendix C.

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 sulphate concentration measured on the soil sample are less than 0.1 per cent, which is below the exposure class of S-3 (Moderate).

Therefore, based on the test results of the sample of existing fill and native soils, when the designer is selecting the exposure class for the structure, the effects of sulphates from within the existing fill and native deposits around the culvert may not need to be considered.

The soil has a pH of between 8.1 and 8.6 and a resistivity of between 2,330 ohm-cm and 11,100 ohm-cm. According to the MTO Gravity Pipe Design Guidelines (2014), pH values between 5.5 and 8.5 are not considered to be detrimental to the culvert durability. Three of the four samples tested had a pH below 8.5 and the sample taken at a depth of 1.5 m at the inlet (Borehole 18-2, SA 4) had a pH of 8.6. The resistivity range indicates that the soil corrosiveness ranges from moderate ($2,000 \text{ ohm-cm} < R < 4,500 \text{ ohm-cm}$) to very low ($6,000 \text{ ohm-cm}$ to $10,000 \text{ ohm-cm}$), as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014). Based on these results some level of pipe protection will be required depending on pipe material may be required.

Based on the results of the sample tested, and given that the culvert is located under the roadway shoulder and will be exposed to de-icing salt, consideration should be given by the designer to designing for a “C” type exposure class as defined by CSA A23.1 Table 1.

It is ultimately up to the designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

6.9 Construction Considerations

It is recommended that geotechnical review and consultation continue throughout the design and construction of the undercrossings due to the complexity of this project. A program of inspection and monitoring will be required during construction of the undercrossing to ascertain whether the intent of the design recommendations provided in this report are being met and that the various project criteria are being achieved.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Madison Kennedy, B.A.Sc., members of the geotechnical engineering group. The technical aspects were reviewed by Mr. Matthew Kelly, P.Eng. a geotechnical engineer and Dr. Storer J. Boone, Ph.D., P.Eng., a Principal of Golder and Designated MTO Tunneling Specialist. Mr. Kevin Bentley, P.Eng., an Associate and Designated MTO Foundations Contact for Golder conducted an independent quality control review of this report.

Signature Page

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Geotechnical Engineer



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MWK/KJB/rb

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<https://golderassociates.sharepoint.com/sites/19941g/deliverables/foundations/1-16-mile-creek-culvert/2-final/1789436-fidr-16-mile-creek-culvert-2018nov20.docx>

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Ontario Regulation 213/91 Construction Projects as amended by O. Reg. 443/09

Ontario Provincial Standard Drawing:

OPSD 3090.101 Foundation, Frost Penetration Depths for Southern Ontario

Ontario Provincial Standard Specification:

OPSS.PROV 539 Construction Specifications for Temporary Protection Systems

OPSS 902 Construction Specifications for Excavating and Backfilling – Structures

OPSS.PROV 1010 Material Specifications for Aggregates-Base, Subbase, Select Subgrade, and Backfill Material

OPSS 1860 Material Specification for Geotextiles

Ontario Water Resources Act:

Ontario Regulation 372/9Amendment to Ontario Regulation 903

ASTM Standards:

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

CSA Standards:

CAN/CSA-A23.1 Concrete Materials and Methods of Concrete Construction / Test Methods and Standard Practices for Concrete

TABLES

Table 1: Summary of Existing Culvert Details

Culvert Location	Approximate Height of Embankment / Cover Thickness ¹	Existing Culvert Details			Approximate Invert Elevation ²		Boreholes
		Type	Approximate Dimensions	Approximate Length	Upstream	Downstream	
Highway 401 – STA 17+940 (STA 17+884 to STA 17+966) Site No. 10-592/C	10.6 m to 10.9 m / 7.5 m to 10.2 m	Concrete Rigid Frame Open Footing	3.1 m wide x 1.6 m high	122 m	238.9 m	234.5 m	6 Boreholes (18-1 to 18-6)

Notes: 1. Embankment height is relative from the top of embankment to the existing ground surface level at the toe of the embankment adjacent to the culvert and the thickness of soil cover above the top of culvert is based on the GA drawings provided by AIA dated April 18, 2018.
 2. Culvert invert elevations are based on drawings provided by AIA dated April 18, 2018.

Prepared By: MCK
 Checked By: MWK
 Reviewed By: KJB

Table 2: Comparison of Trenchless Techniques for Culvert Replacement

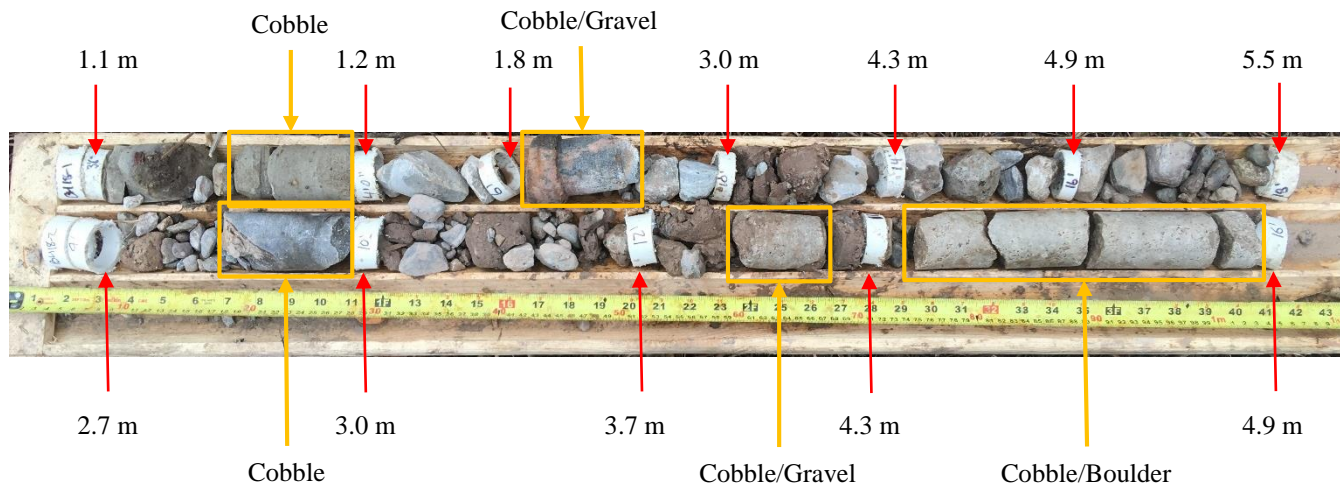
Method	Feasibility	Advantages	Disadvantages	Estimated Relative Cost Factor ²	Risk/Consequence
Jack-and-Bore	Not Feasible	Length and diameter preclude the use of jack and bore ¹			
Pipe Ramming	Not Feasible	Length and diameter preclude the use of jack and bore ¹			
Microtunnelling	Marginally Feasible	<ul style="list-style-type: none">Closed-face machine (slurry MTBM) equipped with rock cutters and crusher chamber required to deal with cobbles and boulders in the fill, sand and gravel and glacial till can provide adequate face supportHigh accuracy for line and gradeDewatering required at shafts onlyFastest rate of advance	<ul style="list-style-type: none">No person access to faceEven if MTBM equipped with rock cutters and crusher chambers, it will not be able to accommodate oversized boulders (larger than 1/3 the machine diameter) and operations will be haltedWoody debris if encountered will likely clog the machineA rescue shaft will be needed to free machine if cobble nests, oversized boulders (> 1/3 machine diameter) or woody debris encounteredMost expensive trenchless solutionRequires largest work area at entry shaft due to the large amount of topside equipmentWhere fills are much softer/looser than the underlying dense to very dense native materials, or where boulders are encountered, the machine may be deflected requiring correction in line or grade	1.4 to 2.0	<ul style="list-style-type: none">Encountering woody debris and oversized cobble nests or boulders - high to very high risk of not completing installationTunnel invert along interface between embankment fill and native materials - low to moderate risk of not achieving line and grade if machine deflected by boulders or hard/very dense soilsGroundwater at the tunnel invert - moderate to high risk of ground loss due to flowing of sands at the tunnel invert and interface with the underlying glacial till
Hand Mining or Mechanically-Assisted Excavation within hooded shield with jacked pipe and face plates (sometimes called breasting boards or plates) that can be closed and opened to control different parts of the tunnel face	Feasible and preferred	<ul style="list-style-type: none">Highly adaptable to variable conditions along alignmentFace access facilitates removal of cobbles, boulders and obstructions in the fill, sand and gravel and glacial tillGood accuracy for line and gradeMost economical solution with line and grade accuracy comparable to microtunnellingSmallest footprint required for entry shaftPotentially the most economical method of installing the culvert at low end of cost range. At high end of cost range, cost may be approaching conventional tunneling with TBM option.	<ul style="list-style-type: none">Labour intensive: The contractor's selected equipment and methods must provide effective control of the stability of the face (e.g., use of hooded shield, stiffeners, forepoling, retractable breast plates with doors etc.)Extensive delays may occur removing and breaking up oversized boulders with high strength, if encounteredUnlike microtunnelling and conventional tunneling, dewatering by horizontal drainage lances/pipes from start and end shafts may be required to control flow in saturated non-cohesive soils <p>Slowest rate of advance</p>	1.0	<ul style="list-style-type: none">Encountering woody debris and oversized cobble nests or boulders - low to moderate risk of not completing installationTunnel invert along interface between embankment fill and native materials - low to moderate risk of not achieving line and grade if machine deflected by boulders or hard/very dense soilsGroundwater at the tunnel invert - moderate risk to high of ground loss due to flowing of sands; risk can be minimized with proper horizontal drainage within tunnel and effective dewatering at shafts and use of and face plates (sometimes called breasting boards or plates) that can be closed and opened to control different parts of the tunnel face
Conventional Tunnelling with Tunnel Boring Machine (TBM)	Feasible	<ul style="list-style-type: none">Face access facilitates removal of cobbles, boulders and obstructions in the fill, sand and gravel and glacial tillClosed-face machine must be used to provide effective control of face stabilityHigh accuracy with line and gradeDewatering required at shafts only	<ul style="list-style-type: none">Older TBMs that do not include a secondary bulkhead and controlled muck discharge system (e.g., discharge gates controlled by load or pressure sensors) should be prohibitedMachines can become jammed or clogged with wood and/or cobbles and boulders; face opening size for any TBM should be restricted so that rock pieces do not pass through the cutterhead and block screw conveyor or slurry systemsRequires a work area at the entry shaft ranging from somewhat smaller than that needed for microtunnelling to comparable	1.0 – 1.5	<ul style="list-style-type: none">Encountering woody debris and oversized cobble nests or boulders - moderate to high risk of not completing installationTunnel invert along interface between embankment fill and native materials - low to moderate risk of not achieving line and grade if machine deflected by boulders or hard/very dense soilsGroundwater at the tunnel invert – moderate to high risk of ground loss due to flowing of sands if closed-face machine selected

Notes:

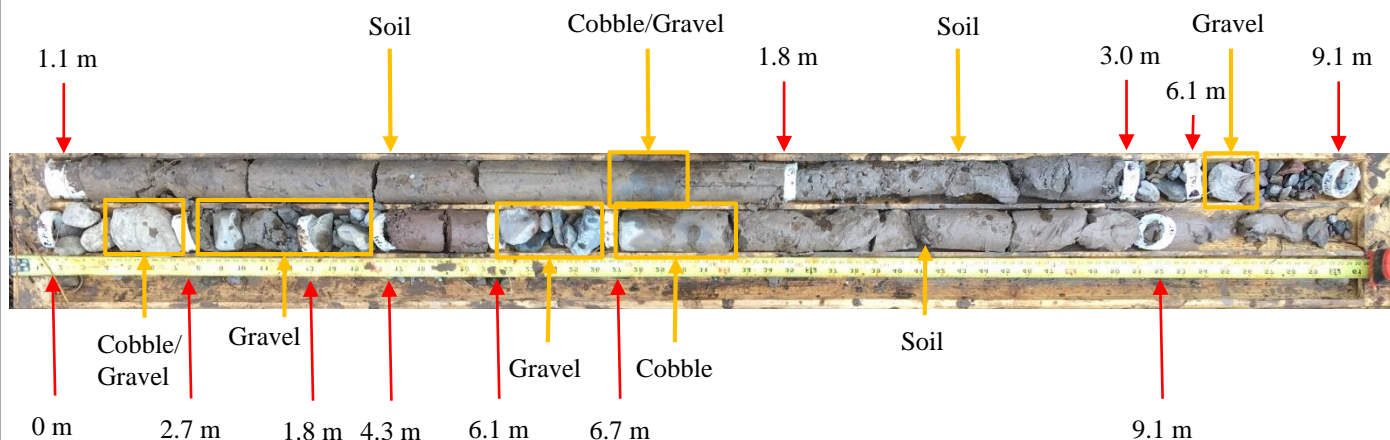
1. The availability of contractors in Ontario with large-scale jack and bore or pipe ramming equipment and experience installing pipes with diameters of 1.8 m or greater is rare or non-existent. The typical maximum casing sizes are 1.83 m for jack and bore and 2.44 m for pipe ramming.
2. The estimated relative cost factor represents an approximately simplified cost estimate for each option divided by the estimated cost for the least expensive option (e.g., a relative cost factor of 2 indicates that the trenchless technology option is twice as costly as the least expensive option).

FIGURES

Borehole 18-1 (Upper Row) and 18-2 (Lower Row)




Borehole 18-5 (Lower Row) and 18-6 (Upper Row)



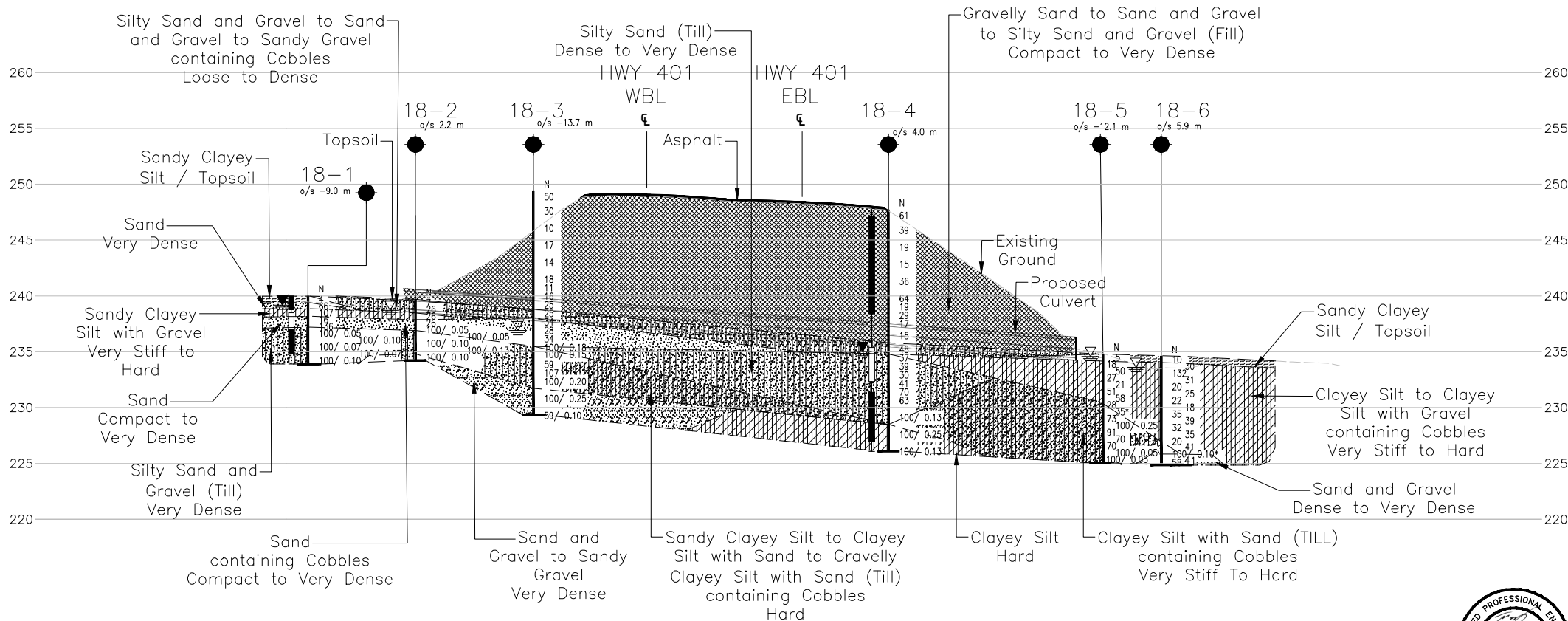
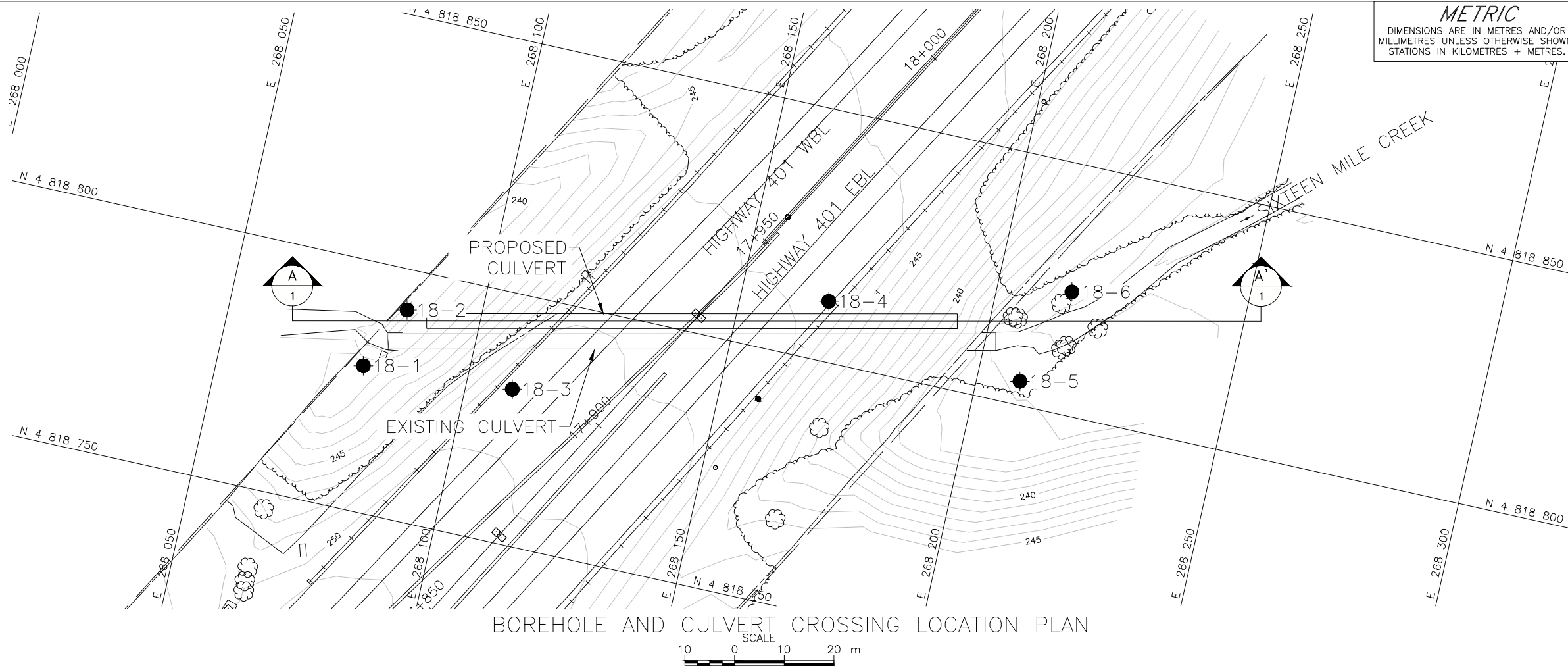
Note:

Borehole advancement was carried out with BW casing utilizing washboring techniques with AW casing. The photographs represent the material recovered from the AW casing during borehole advancement.

REVISION DATE: May 16, 2018 BY: MCK Project: 1789436

PROJECT SIXTEEN MILE CREEK CULVERT REPLACEMENT STRUCTURAL CULVERT REHABILITATION / REPLACEMENT, HIGHWAY 401 G.W.P 2219-14-00			
TITLE AW Recovery Photographs Boreholes 18-1, 18-2, 18-5 and 18-6			
	PROJECT No. 1789436		FILE No. ----
	DESIGN	MCK	SCALE
	CADD	-- --	REV.
	CHECK		
	REVIEW		
FIGURE 1			

DRAWINGS



CONT No.
GWP No. 2219-14-00

HIGHWAY 401
SIXTEEN MILE CREEK CULVERT
BOREHOLE LOCATIONS AND SOIL STRATA

KEY PLAN
SCALE
2 0 2 4 km

LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer
- WL in piezometer, measured on March 22, 2018
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD83, ZONE 10)

No.	ELEVATION	NORTHING	EASTING
18-1	240.0	4818779.1	268079.0
18-2	239.7	4818791.9	268085.2
18-3	249.4	4818781.0	268109.3
18-4	247.7	4818812.1	268167.6
18-5	234.8	4818804.8	268208.6
18-6	234.6	4818824.6	268214.9

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 plan and surface data provided in digital format by AIA, drawing file no. E1624014_GA_WithoutProfile.dwg, dated , 2018, received May 16, 2018.

Culvert crossing profile general arrangement provided in digital format by AIA, drawing file no. E1624014.dwg, dated March, 2018, received April 18, 2018.

Base Data MNRF 2018.

NO.	DATE	BY	REVISION

Geocres No. 30M12-425

HWY. 401	PROJECT NO. 1789436	DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: 10/18/2018
DRAWN: SMD	CHKD. MWK	APPD. KJB

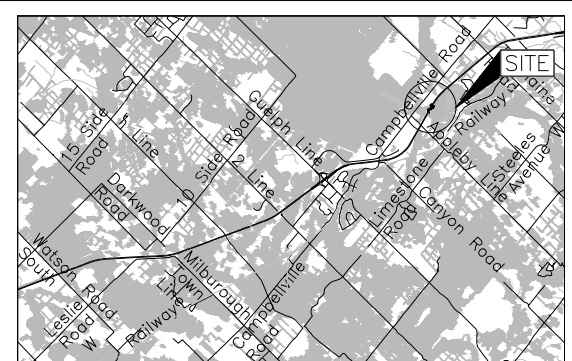
SITE: 10-592/C
DWG. 1



HIGHWAY 401

SIXTEEN MILE CREEK CULVERT

SETTLEMENT MONITORING INSTRUMENTATION LOCATIONS AND DETAILS





KEY PLAN
SCALE



2 0 2 4 km

LEGEND

- | | |
|---|---------------------------|
|  | Surface Settlement Marker |
|  | Settlement Point |

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.

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Base Data. MNRF 2018.

NO.	DATE	BY	REVISION				
Geocres No. 30M12-425							
HWY. 401			PROJECT NO. 1789436				DIST. .
SUBM'D. MCK		CHKD. MCK		DATE: 10/18/2018		SITE: .	
DRAWN: SMD		CHKD. .		APPD. .		DWG. 2	



APPENDIX A

Record of Boreholes

LIST OF SYMBOLS

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

I. GENERAL

π	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

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	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
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_{α}	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
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

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

Notes: 1
2

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

PROJECT		1789436		RECORD OF BOREHOLE No 18-1		SHEET 1 OF 1		METRIC						
G.W.P.		2219-14-00		LOCATION		N 4818779.1; E 268079.0 MTM NAD 83 ZONE 10 (LAT. 43.508038; LONG. -79.954146)		ORIGINATED BY EG						
DIST		Central HWY 401		BOREHOLE TYPE		Portable Equipment (Manual Hammer), BW Casing, Washboring		COMPILED BY BR						
DATUM		Geodetic		DATE		March 12 to 13, 2018		CHECKED BY MCK						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS					
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p	W	W _L	γ	GR SA SI CL
240.0	0.0	GROUND SURFACE												
239.4	0.6	Sandy CLAYEY SILT, trace gravel, containing organics and cobbles Firm Dark brown Moist		1	SS	4								
238.9	1.1	SAND, some silt Very dense Brown Wet		2A	SS	66		239						
238.2	1.8	SANDY CLAYEY SILT with GRAVEL, containing cobbles (75 mm to 150 mm) Hard Grey to dark grey Moist to wet		2B	SS	107								
237.2	2.8	SAND, some silt, trace to some gravel, trace to some clay, containing cobbles (100 mm) Compact to very dense Red/Brown Wet		3	SS	107		238						5 76 14 5
				4	SS	16								
				5A	SS	136								
				5B	SS	100/0.06		237						
				6	SS	100/0.10								
				7	SS	100/0.10		236						32 41 22 5
				8	SS	100/0.07								
				9	SS	100/0.07		235						
				10	SS	100/0.10								
233.9	6.1	END OF BOREHOLE						234						
NOTES: 1. Groundwater level measurement in piezometer: Date: Depth (m) Elev. (m) 03/13/18 1.8 238.2 04/12/18 0.8 239.2 2. Point load index testing carried out on a sample of cobble collected in core barrel at a depth of 1.1 m (Elev. 238.9 m) Type Is(50) Axial 2.6 MPa Diametral 1.4 MPa														

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PROJECT		1789436		RECORD OF BOREHOLE No 18-2		SHEET 1 OF 1		METRIC									
G.W.P.		2219-14-00		LOCATION		N 4818791.9; E 268085.2 MTM NAD 83 ZONE 10 (LAT. 43.508154; LONG. -79.954070)		ORIGINATED BY EG									
DIST		Central HWY 401		BOREHOLE TYPE		Portable Equipment (Manual Hammer), BW Casing, Washboring		COMPILED BY BR									
DATUM		Geodetic		DATE		March 14, 2018		CHECKED BY MCK									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
239.7	GROUND SURFACE																
0.0	TOPSOIL																
	Silty SAND and GRAVEL, trace to some clay, containing organics and cobbles Loose to compact Dark brown to red/brown Wet		1	SS	7	▽	239										
			2	SS	26		239										
238.5	Sandy CLAYEY SILT with GRAVEL Very stiff Red/Brown Wet		3	SS	26		238										
237.9			4	SS	26		238										
1.8			5A	SS	100/0.05		237										
236.9	SAND, trace to some gravel, trace silt, containing cobbles Compact to very dense Red/brown Wet		5B	SS	100/0.05		237										
2.8	CLAYEY SILT with SAND, trace to some gravel, containing cobbles (75 mm to 250 mm) (TILL) Hard Red/Brown Wet		6	SS	100/0.06		236										
			7	SS	100/0.10		236										
			8	SS	100/0.13		236										
234.8	Sandy GRAVEL Very dense Red/brown Moist		9	SS	100/0.10	235											
4.9																	
234.2	END OF BOREHOLE																
5.5	NOTES: 1. Water level recorded in open borehole at a depth of 1.0 m below ground surface (Elev. 238.7 m) upon completion of drilling. 2. Point load index testing carried out on a sample of cobble collected from core barrel at a depth of 2.7 m (Elev. 237.0 m) Type Is(50) Axial 5.0 MPa Diametral 2.2 MPa																

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PROJECT 1789436		RECORD OF BOREHOLE No 18-3		SHEET 1 OF 2		METRIC	
G.W.P. 2219-14-00		LOCATION N 4818781.0; E 268109.3 MTM NAD 83 ZONE 10 (LAT. 43.508057; LONG. -79.953772)		ORIGINATED BY JL			
DIST Central HWY 401		BOREHOLE TYPE Diedrich D90; 108 mm I.D., 210 mm O.D. Hollow Stem Augers		COMPILED BY BR			
DATUM Geodetic		DATE March 11 to 13, 2018		CHECKED BY MCK			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							w _p w w _L		
249.4	GROUND SURFACE							20 40 60 80 100									
0.0	ASPHALT (225 mm)																
0.2	Gravelly sand, trace fines (FILL) Dense to very dense Brown and grey Dry		1	SS	50												
	- Containing clayey silt pockets below a depth of 1.5 m		2	SS	30												
246.8																	
2.6	Silty sand and gravel (FILL) Compact Red-brown and grey Moist - Auger grinding at a depth of 3.0 m to 4.6 m - Wet pocket at 3.4 m		3	SS	10												
			4	SS	17												
			5	SS	14												
	- Auger grinding at a depth of 7.0 m																
			6	SS	18												
		7	SS	11													
		8	SS	16													
	- Auger grinding from 10.4 m to 10.7 m																
		9	SS	25													
238.8																	
10.6	Sandy GRAVEL, trace to some silt, trace clay Compact Grey Wet		10	SS	25												
238.0																	
11.4	SAND, some gravel, some silt, containing clayey silt pockets Compact to dense Red-brown Wet		11A	SS	34												
			11B														
			12	SS	28												
			13	SS	34												
235.5			14	SS	100/ 0.18												
235.1	Sandy CLAYEY SILT, some gravel (TILL) Hard Brown to grey Wet		15	SS	100/ 0.15												
14.3																	
234.5																	

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

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PROJECT 1789436		RECORD OF BOREHOLE No 18-3				SHEET 2 OF 2		METRIC									
G.W.P. 2219-14-00		LOCATION N 4818781.0; E 268109.3 MTM NAD 83 ZONE 10 (LAT. 43.508057; LONG. -79.953772)				ORIGINATED BY JL											
DIST Central HWY 401		BOREHOLE TYPE Diedrich D90; 108 mm I.D., 210 mm O.D. Hollow Stem Augers				COMPILED BY BR											
DATUM Geodetic		DATE March 11 to 13, 2018				CHECKED BY MCK											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
14.9	Silty SAND, some gravel, trace to some clay (TILL) Very dense Brown to grey Wet Gravelly CLAYEY SILT with SAND (TILL) Hard Brown to grey Wet - Auger grinding between depths of about 16.2 m to 16.8 m and 17.1 m to 17.7 m		16	SS	59		234										
			17	SS	107		233										
			18	SS	100/0.20												
231.7	SAND and GRAVEL, trace to some silt, trace clay Very dense Brown and grey Wet - Auger grinding at a depth of about 17.7 m to 18.0 m and 18.6 m to 19.8 m						232										
17.7			19	SS	100/0.25		231										
							230										
229.3	END OF BOREHOLE:		20	SS	59/0.10												
20.1	NOTE: 1. Water level recorded in open borehole at a depth of about 12.5 m below ground surface (Elev. 236.9 m) on March 13, 2018.																

PROJECT 1789436		RECORD OF BOREHOLE No 18-4		SHEET 1 OF 2		METRIC	
G.W.P. 2219-14-00		LOCATION N 4818812.1; E 268167.6 MTM NAD 83 ZONE 10 (LAT. 43.508339; LONG. -79.953053)		ORIGINATED BY JL			
DIST Central HWY 401		BOREHOLE TYPE Diedrich D90; 108 mm I.D., 210 mm O.D. Hollow Stem Augers		COMPILED BY BR			
DATUM Geodetic		DATE March 14 to 16 and 18, 2018		CHECKED BY MCK			


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _P W W _L	WATER CONTENT (%)			
247.7	GROUND SURFACE													
0.0	ASPHALT (200 mm)													
0.4	Sand and gravel, containing asphalt fragments, trace fines (FILL) Very dense Grey to black Dry		1	SS	61									
246.5														
1.2	Gravelly sand, trace fines (FILL) Very dense Grey and brown Dry		2	SS	39									
	Sand and gravel, some silt, trace clay (FILL) Compact to very dense Brown, grey, red/brown Dry to moist - Auger grinding between a depth of about 1.8 m to 2.4 m													
			3	SS	19									
	- Wet pocket at 4.6 m													
	- Auger grinding from a depth of about 4.9 m to 5.8 m		4	SS	15									
	- Auger grinding from a depth of about 6.0 m to 8.8 m		5	SS	36									
			6	SS	64									
			7	SS	19									
	- Auger grinding from a depth of about 9.1 m to 9.8 m		8	SS	29									
			9	SS	17									
	- Containing clayey silt seams below a depth of 11.2 m		10A	SS	15									
			10B											
235.8														
11.9	SAND and GRAVEL, some silt, trace clay Dense Brown Wet - Auger grinding from a depth of about 12.2 m to 12.8 m		11	SS	48									
234.8														
12.9	Silty SAND, some gravel, trace to some clay (TILL) Dense to very dense Red/brown and grey Wet - Auger grinding from a depth of about 13.7 m to 14.3 m		12	SS	37									
			13	SS	39									
			14	SS	30									

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+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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PROJECT 1789436		RECORD OF BOREHOLE No 18-4		SHEET 2 OF 2		METRIC						
G.W.P. 2219-14-00		LOCATION N 4818812.1; E 268167.6 MTM NAD 83 ZONE 10 (LAT. 43.508339; LONG. -79.953053)		ORIGINATED BY JL								
DIST Central HWY 401		BOREHOLE TYPE Diedrich D90; 108 mm I.D., 210 mm O.D. Hollow Stem Augers		COMPILED BY BR								
DATUM Geodetic		DATE March 14 to 16 and 18, 2018		CHECKED BY MCK								
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES					
	--- CONTINUED FROM PREVIOUS PAGE ---					20 40 60 80 100	20 40 60 80 100	10 20 30				
228.5	Silty SAND, some gravel, trace to some clay (TILL) Dense to very dense Red/brown and grey Wet		15	SS	41							
			16	SS	70							
			17	SS	63							
			18	SS	100/ 0.13							
19.2	CLAYEY SILT, trace to some sand Hard Grey Wet - Auger grinding at a depth about 19.2 m - Auger Grinding at 20.1 m		19	SS	100/ 0.25							0 7 65 28
226.1			20	SS	100/ 0.13							
21.6	END OF BOREHOLE											
NOTES: 1. Water level recorded in borehole at a depth of 12.5 m below ground surface (Elev. 235.2 m) on March 18, 2018. 2. Water level measured in standpipe Piezometer: Date Depth (m) Elev. (m) 03/22/18 12.5 235.2												

PROJECT		1784436		RECORD OF BOREHOLE No 18-5		SHEET 1 OF 1		METRIC									
G.W.P.		2219-14-00		LOCATION		N 4818804.8; E 268208.6 MTM NAD 83 ZONE 10 (LAT. 43.508276; LONG. -79.952545)		ORIGINATED BY EG									
DIST		Central HWY 401		BOREHOLE TYPE		Portable Equipment (Manual Hammer), BW Casing, Washboring		COMPILED BY BR									
DATUM		Geodetic		DATE		March 29 and April 4, 2018		CHECKED BY MCK									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa									WATER CONTENT (%)	
234.8	GROUND SURFACE																
0.0	Sandy CLAYEY SILT with organics, some to trace gravel		1	SS	5												
234.2	Firm Brown to Red/brown Wet		2A	SS	18												
0.6	CLAYEY SILT, some sand, trace gravel, containing silt seams and cobbles (100 mm)		2B	SS	18												
	Very stiff to hard Grey Wet		3	SS	50												
			4	SS	27												
231.8			5	SS	21												
3.1	CLAYEY SILT with GRAVEL, some sand		6	SS	51												
	Hard Brown Wet		7	SS	58												
230.5			8	SS	28												
4.3	CLAYEY SILT with SAND, some gravel, containing cobbles (75 mm to 130 mm) (TILL)		9	SS	35*												
	Very stiff to hard Red/brown to grey Wet		10A	SS	73												
			10B	SS	73												
			11	SS	100/0.25												
			12	SS	91												
			13	SS	70												
			14	SS	70												
			15	SS	100/0.05												
			16	SS	100/0.05												
225.1	- Grey below 9.1 m																
9.8	END OF BOREHOLE																
* - No Recovery																	
NOTES:																	
1. Water level recorded in open borehole at a depth of 0.1 m below ground surface (Elev. 234.7 m) upon completion of drilling.																	
2. Point load index testing carried out on a sample of cobble collected from core barrel at a depth of 6.7 m (Elev. 228.1 m)																	
Type Is(50)																	
Axial 12.2 MPa																	
Diametral 11.1 MPa																	

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PROJECT		1789436		RECORD OF BOREHOLE No 18-6		SHEET 1 OF 1		METRIC						
G.W.P.		2219-14-00		LOCATION		N 4818824.6; E 268214.9 MTM NAD 83 ZONE 10 (LAT. 43.508454; LONG. -79.952469)		ORIGINATED BY EG						
DIST		Central HWY 401		BOREHOLE TYPE		Portable Equipment (Manual Hammer), BW Casing, Washboring		COMPILED BY BR						
DATUM		Geodetic		DATE		March 26 to 28, 2018		CHECKED BY MCK						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
234.6	GROUND SURFACE													
0.0	TOPSOIL		1	SS	10									
234.0	CLAYEY SILT, some sand, trace to some gravel, containing cobbles (75 mm) Very stiff to hard Dark brown to grey Wet - Containing silt and clay seams below a depth of 4.9 m		2	SS	30									
0.6			3	SS	132									
			4	SS	31									
			5	SS	20									
			6	SS	25									
			7	SS	22									
			8	SS	18									
			9A	SS	35									
			9B											
			10	SS	39									
			11	SS	32									
			12	SS	35									
			13	SS	20									
			14	SS	41									
225.8		SAND and GRAVEL, some silt, trace clay Dense to very dense Red/brown Wet		15	SS	100/0.10*								
8.8			16	SS	41									
			17	SS	58									
224.9														
9.8	END OF BOREHOLE													
	* - No Recovery - Split-Spoon Sampler Refusal NOTES: 1. Water level recorded in borehole at a depth of 1.0 m below ground surface (Elev. 233.6 m) upon completion of drilling. 2. Continuous sampling was carried out to a depth of 1.8 m. An adjacent borehole was advanced using BW casing to carry out continuous sampling below a depth of 1.8 m.													

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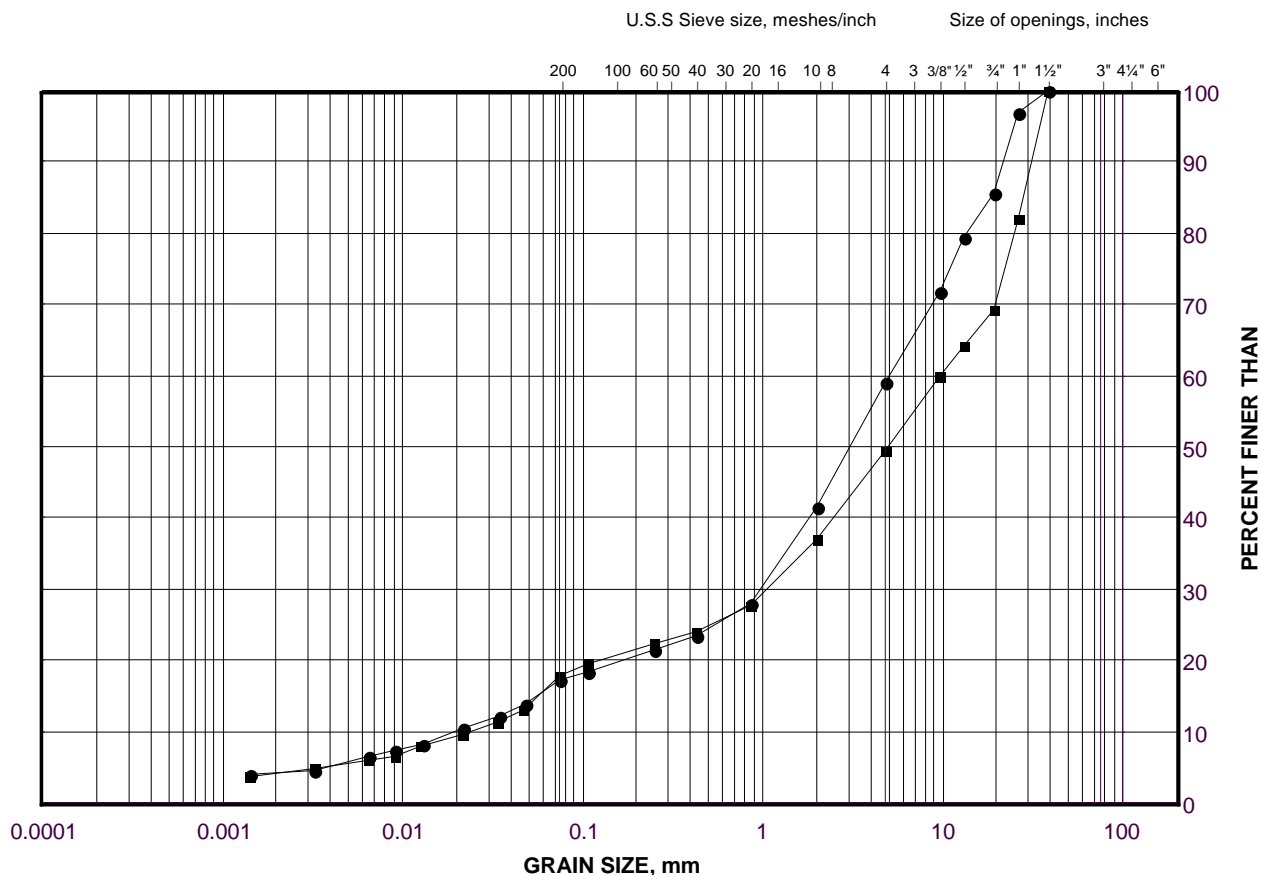
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Silty Sand and Gravel to Sand and Gravel (Fill)

FIGURE B1



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

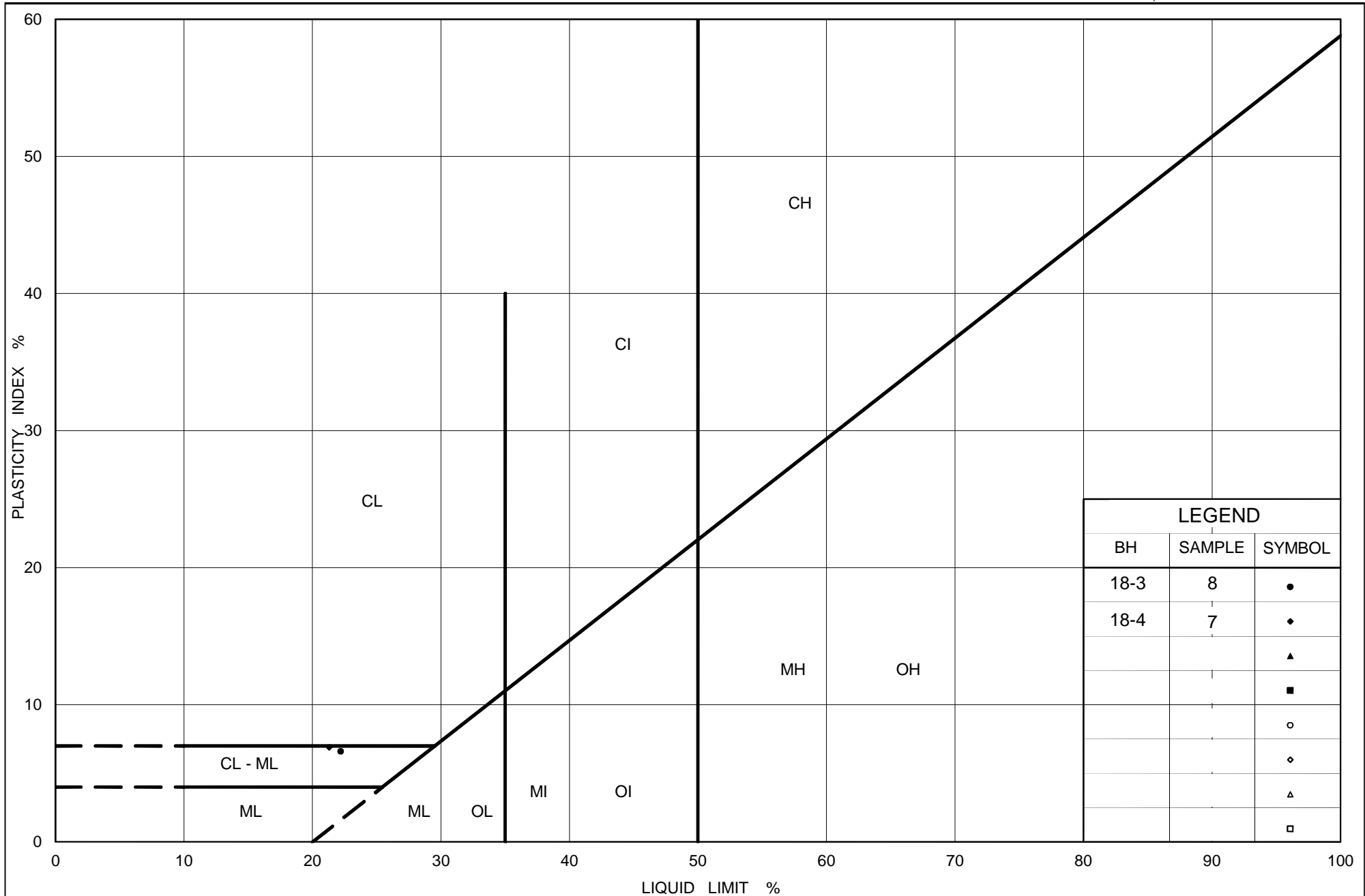
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	18-4	7	239.0
■	18-3	8	240.0

Project Number: 1789436

Checked By: MCK

Golder Associates

Date: 10-May-18



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Sand and Gravel to Sand and Gravel (Fill) (Fines Component)

Figure No. B2

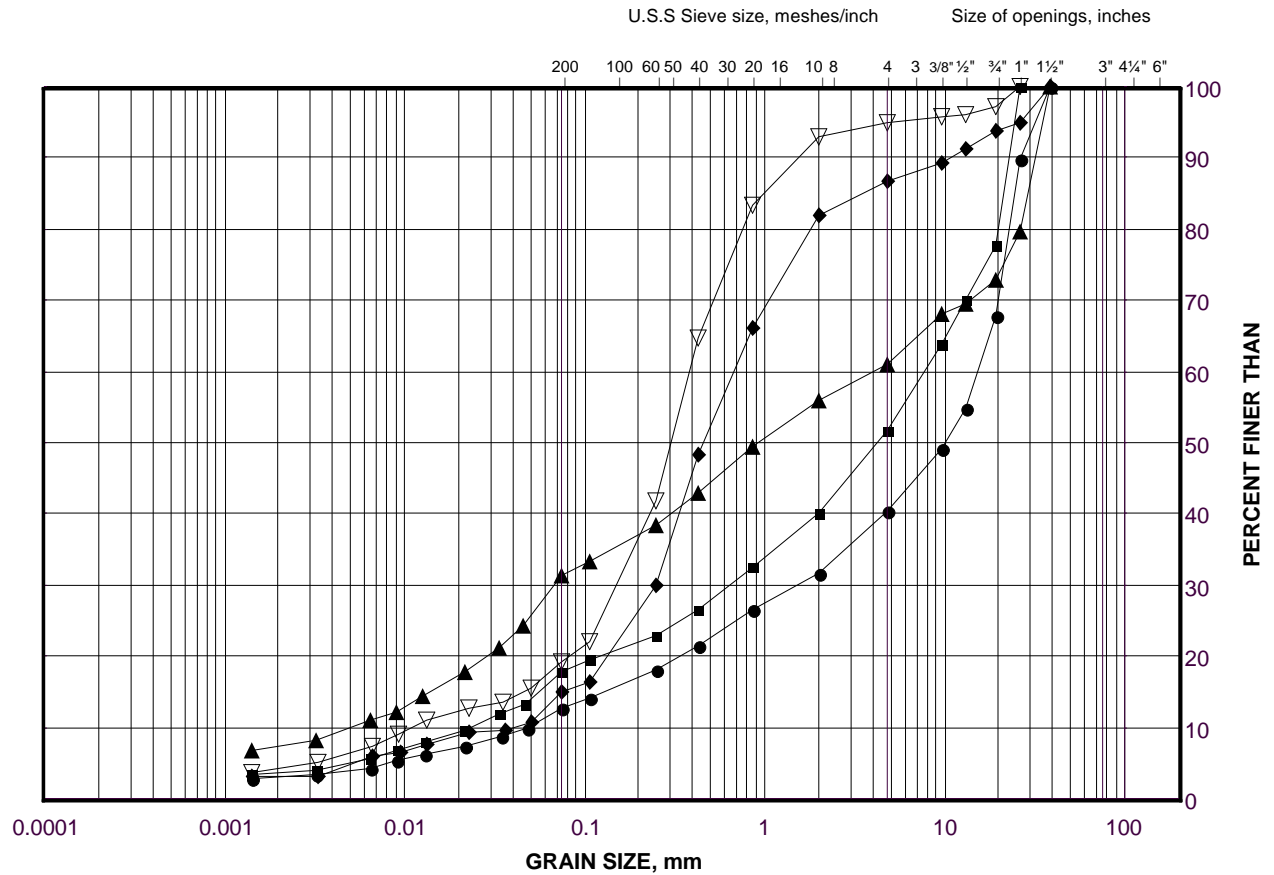
Project No. 1789436

Checked By: MCK

GRAIN SIZE DISTRIBUTION

Sand to Sand and Gravel to Sandy Gravel (Upper Granular Deposit)

FIGURE B3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	18-3	10	238.5
■	18-4	11	235.2
◆	18-3	13	236.2
▲	18-2	2	238.8
▽	18-1	4	237.8

Project Number: 1789436

Checked By: MCK

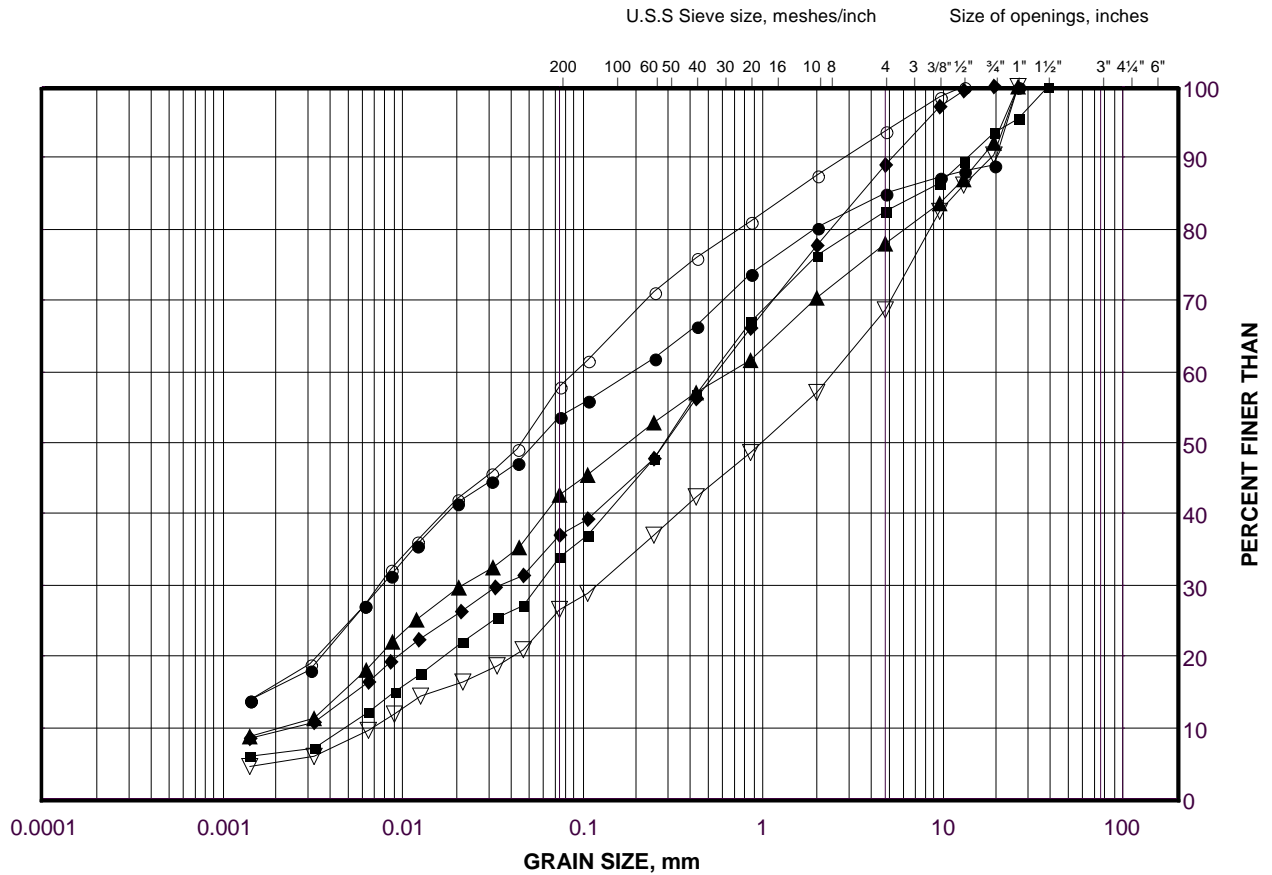
Golder Associates

Date: 10-May-18

GRAIN SIZE DISTRIBUTION

Silty Sand to Silty Sand and Gravel (Till) / Sandy Clayey Silt to Gravelly Clayey Silt with Sand (Till)

FIGURE B4



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

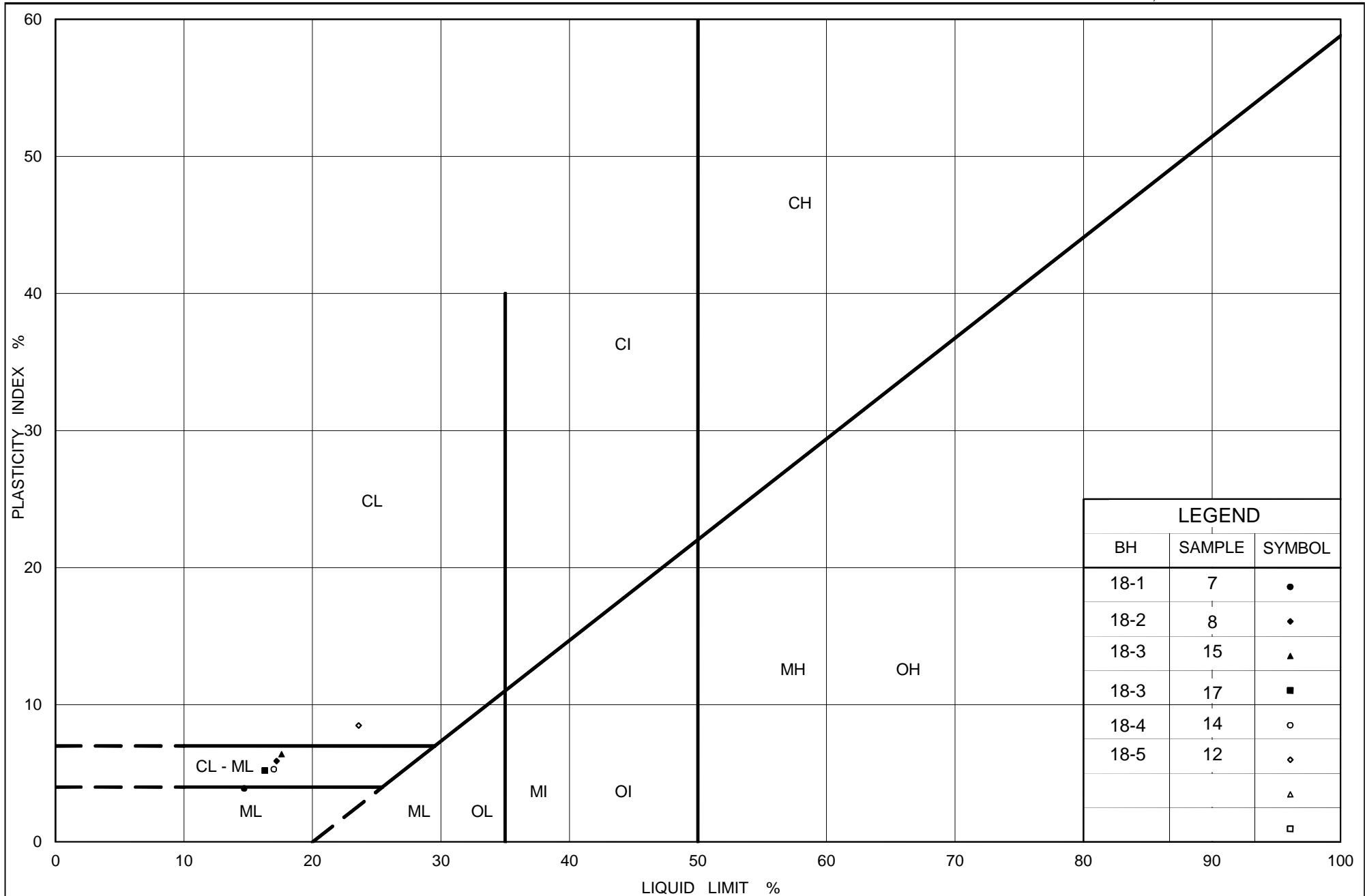
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	18-5	12	227.8
■	18-4	14	233.0
◆	18-3	15	235.1
▲	18-3	17	233.0
▽	18-1	7	236.2
○	18-2	8	235.3

Project Number: 1789436

Checked By: MCK

Golder Associates

Date: 10-May-18



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Ontario

PLASTICITY CHART

Silty Sand to Silty Sand and Gravel (Till) / Sandy Clayey Silt to Gravelly Clayey Silt with Sand (Till) (Fines Component)

Figure No. B5

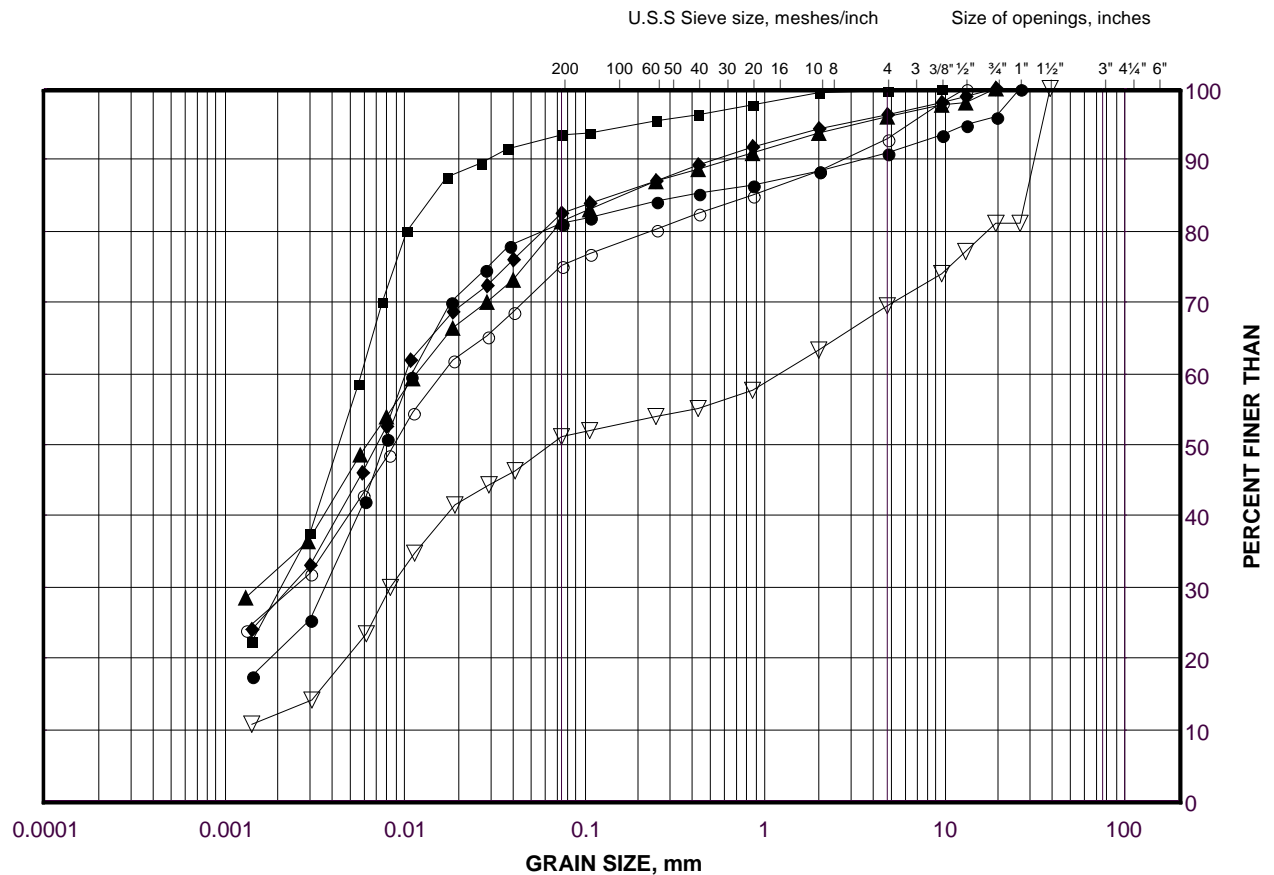
Project No. 1789436

Checked By: MCK

GRAIN SIZE DISTRIBUTION

Clayey Silt to Clayey Silt with Gravel

FIGURE B6



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

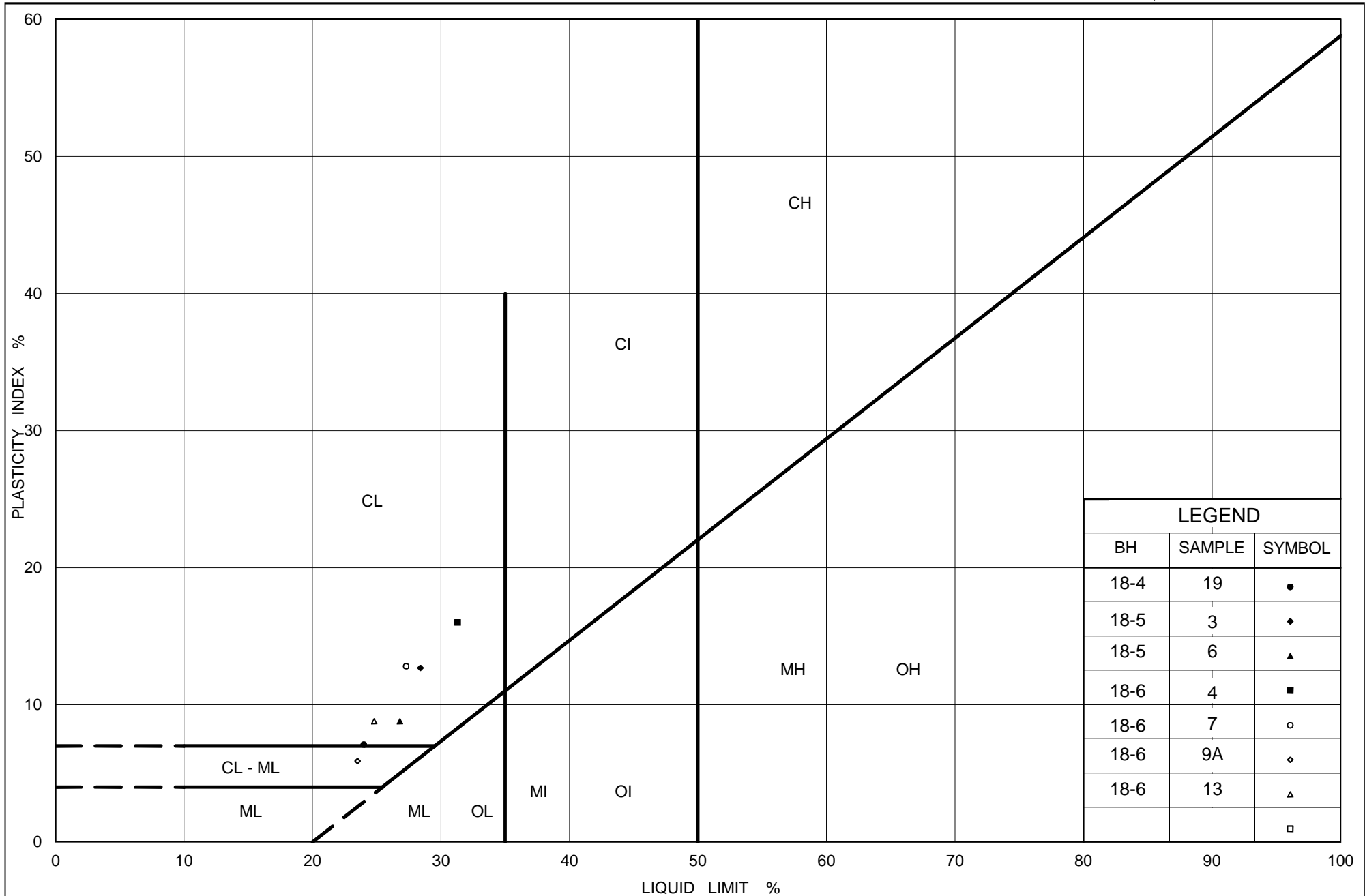
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	18-6	13	227.0
■	18-4	19	227.8
◆	18-5	3	233.3
▲	18-6	4	232.5
▽	18-5	6	231.5
○	18-6	7	230.6

Project Number: 1789436

Checked By: MCK

Golder Associates

Date: 10-May-18



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Ontario

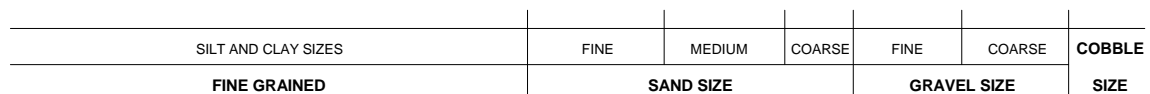
PLASTICITY CHART Clayey Silt to Clayey Silt with Gravel

Figure No. B7

Project No. 1789436

Checked By: MCK

FIGURE B8



SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	18-6	17	225.8
■	18-3	19	231.0

Date: 10-May-18

APPENDIX C

Analytical Test Results

Client: Golder Associates Ltd. (Mississauga)
6925 Century Avenue
Mississauga, ON
L5N 7K2
Attention: Ms. Madison Kennedy
PO#:
Invoice to: Golder Associates Ltd. (Mississauga)

Report Number: 1805486
Date Submitted: 2018-04-13
Date Reported: 2018-04-20
Project: 1789436
COC #: 197339

Page 1 of 3

Dear Madison Kennedy:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: _____

Addrine Thomas, Inorganics Supervisor

All analysis is completed in Ottawa, Ontario (unless otherwise indicated).

Eurofins Ottawa is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on our CALA scope of accreditation. It can be found at <http://www.cala.ca/scopes/2602.pdf>.

Eurofins(Ottawa) is certified and accredited for specific parameters by OMAFRA, Ontario Ministry of Agriculture, Food and Rural Affairs (for farm soils). Licensed by Ontario MOE for specific tests in drinking water.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required.

Certificate of Analysis

Client: Golder Associates Ltd. (Mississauga)
 6925 Century Avenue
 Mississauga, ON
 L5N 7K2
 Attention: Ms. Madison Kennedy
 PO#:
 Invoice to: Golder Associates Ltd. (Mississauga)

Report Number: 1805486
 Date Submitted: 2018-04-13
 Date Reported: 2018-04-20
 Project: 1789436
 COC #: 197339

					Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1353885 Soil 2018-03-14 BH 18-2 SA4	1353886 Soil 2018-03-12 BH 18-3 SA16	1353887 Soil 2018-03-14 BH 18-4 SA8	1353888 Soil 2018-03-27 BH 18-6 SA13
Group	Analyte	MRL	Units	Guideline					
Agri. - Soil	pH	2.00				8.58	8.43	8.33	8.13
	SO4	0.01	%			<0.01	0.02	<0.01	0.02
General Chemistry	Cl	0.002	%			0.002	0.003	0.026	0.004
	Electrical Conductivity	0.05	mS/cm			0.09	0.16	0.43	0.28
	Resistivity	1	ohm-cm			11100	6250	2330	3570

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
 Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Client: Golder Associates Ltd. (Mississauga)
 6925 Century Avenue
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 Attention: Ms. Madison Kennedy
 PO#:
 Invoice to: Golder Associates Ltd. (Mississauga)

Report Number: 1805486
 Date Submitted: 2018-04-13
 Date Reported: 2018-04-20
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QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 343852 Analysis/Extraction Date 2018-04-19 Analyst C_F			
Method Ag Soil			
pH	4.74	100	90-110
Method Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	99	85-115
Method Resistivity - soil			
Resistivity			
Run No 343932 Analysis/Extraction Date 2018-04-20 Analyst C_F			
Method C CSA A23.2-4B			
Chloride		101	90-110
Run No 343933 Analysis/Extraction Date 2018-04-20 Analyst C_F			
Method AG SOIL			
SO4	<0.01 %	84	70-130

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
 Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX D

Non-Standard Special Provisions

PIPE INSTALLATION BY TRENCHLESS METHOD – Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR THE INSTALLATION OF PIPES BY TRENCHLESS METHODS

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1. SCOPE

This specification covers the general requirements for the installation of pipes by trenchless methods, including Jack & Bore, Pipe Ramming, Directional Drilling, and Tunnelling. The Contractor shall determine the most appropriate method of installation for each of the crossing locations.

This specification shall supersede OPSS 415 (Construction Specification for Pipeline Installation by Tunneling), OPSS 416 (Construction Specification for Pipeline and Utility Installation by Jacking and Boring) and OPSS 450 (Construction Specification for Pipeline and Utility Installation in Soil by Horizontal Directional Drilling).

2. REFERENCES

This specification refers to the following standards, specifications, or publications:

Ontario Provincial Standard Specifications, General

OPSS 180 Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Construction

OPSS 401	Trenching, Backfilling, and Compacting
OPSS 404	Support Systems
OPSS 491	Preservation, Protection, and Reconstruction of Existing Facilities
OPSS 492	Site Restoration Following Installation of Pipelines, Utilities and Associated Structures
OPSS 517	Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS.PROV 539	Temporary Protection Systems

Ontario Provincial Standard Specifications, Material

OPSS.PROV 1004	Aggregates - Miscellaneous
OPSS.PROV 1350	Concrete - Materials and Production
OPSS.PROV 1440	Steel Reinforcement for Concrete
OPSS 1802	Smooth Walled Steel Pipe
OPSS.PROV 1820	Circular and Elliptical Concrete Pipe
OPSS 1840	Non-Pressure Polyethylene (PE) Plastic Pipe Products

American Society for Testing and Materials (ASTM) International Standards

ASTM A252-93	Welding and Seamless Steel Pipe Piles
ASTM D2657-03	Standard Practice for Heat Fusion Joining of Polyelofin Pipe and Fittings
ASTM D3350	Standard Specification for Polyethylene Plastics Pipe and Fittings Materials
ASTM F894	Polyethylene Large Diameter Profile Wall Sewer and Drain Pipe

Canadian Standards Association Standards:

CSA B182.6	Profile Polyethylene Sewer Pipe and Fittings.
CAN/CSA A5-93	Portland Cement
CSA W59	Welded Steel Construction (Metal Arc Welding)

3. DEFINITIONS

For the purpose of this specification, the following definitions apply:

Auger Jack & Bore: a method of forming a horizontal bore in the subsurface by essentially simultaneously jacking ahead and rotating a cutter head, followed by removal of material from inside the bore by using an auger.

Backreamer: a cutting head suitably designed for the subsurface conditions that is attached to the end of a drill string to enlarge the pilot bore during a pullback operation.

Bore Path: a drilled path according to the grade and alignment tolerances specified in the Contract Documents.

Design Engineer: means the Engineer retained by the Contractor who produces the original design and working drawings. The design engineer shall be licensed to practice in the Province of Ontario.

Design Checking Engineer: means the Engineer retained by the Contractor who checks the original design and working drawings. The design checking engineer shall be licensed to practice in the Province of Ontario.

Digger Shield/Hand Mining: a method of forming a horizontal bore in the subsurface by essentially simultaneously jacking ahead while tunnelling advances using hand-mining (man-entry operation or “Jack

and Mine) or a “digger” type shield with a hydraulic excavator arm to remove materials from inside the liner pipe.

Drilling Fluids: a mixture of water and additives, such as bentonite, polymers, surfactants, and soda ash, designed to block the pore space on a bore wall, reduce friction in the bore, and to suspend and carry cuttings to the surface.

Drilling Fluid Fracture or Frac Out: a condition where the drilling fluid’s pressure in the bore is sufficient to overcome the in situ confining stress, thereby fracturing the soil and/or rock materials and allowing the drilling fluids to migrate to the surface at an unplanned location.

Engineer: a Professional Engineer licensed by the Professional Engineers of Ontario to practice in the Province of Ontario.

Excavation: includes all materials encountered regardless of type and extent. Excavation shall include removal of natural soil, large boulders, cobbles, wood and fill regardless of means necessary to break consolidated materials for removal.

Environmentally Sensitive Area (ESA): areas adjacent to construction that are off limits to the Contractor as specified elsewhere in the Contract.

Fill: man-made mixture of previously placed/handled materials such as sand, clay, silt, gravel, broken rock, sometimes containing organic and/or deleterious materials, placed in an excavation or other area to raise the surface elevation.

Grouting: injection of grout into voids.

Guidance System: an electronic system capable of locating the position, depth and orientation of the drill head during the directional drilling process.

Directional Drilling (DD): directional boring or guided boring.

HDPE: high density polyethylene.

Inadvertent Returns: the flow of unexpected fluids, saturated materials (or running soil) towards the drilling rig that typically originated from an artesian aquifer encountered during the drilling process.

Loss of Circulation: the discontinuation of the flow of drilling fluid in the bore back to the entry or exit point or other planned recovery points.

Pilot Bore: the initial bore to set directional controlled horizontal and vertical alignment between the connecting points.

Pipe Jacking: a method for installing steel casing or concrete pipe in the subsurface utilizing hydraulically operated jacks of adequate number and capacity to ensure smooth and uniform advancement without overstressing the liner/pipe.

Pipe Ramming: a method for installing steel casings utilizing the energy from a percussion hammer to advance a steel casing with a cutting shoe attached at the front end of the casing.

Primary Liner (Support): system installed prior to or concurrent with excavation, to maintain stability of an

excavation and to support earth or rock and any structure utilities or other facilities in or on the supported earth or rock mass, until the excavation is completed.

Product: pipe culverts, pipe sewers, watermain pipe and sanitary pipe.

Pullback: that part of the DD method in which the drill string is pulled back through the bore path to the entry point.

Quality Verification Engineer (QVE): an Engineer who has a minimum of five (5) years experience in the field of pipe installation using trenchless methods or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and to issue Certificate(s) of Conformance.

Reaming: a process for pulling a tool attached to the end of the drill string through the bore path to enlarge the bore and mix the cuttings with the drilling fluid. This typically includes multiple passes.

Rock: natural beds or massive fragments, or the hard, stable, cemented part of the earth's crust, igneous, metamorphic, or sedimentary in origin, which may or may not be weathered and includes boulders having a size equivalent to 0.3 m in diameter or greater.

Secondary Liner: concrete pipe, HDPE pipe or un-reinforced cast-in-place concrete, installed subsequent to tunnel excavation.

Shaft: vertically sided excavation used as entry and/or exit points from which the trenchless method is initiated or directed for the installation of product.

Strike Alert: a system that is intended to alert and protect the operator in the case of inadvertent drilling into an electrical utility cable. The strike alert system consists of a sensor and an alarm connected to the drill rig and a grounding stake. The alarm may be audio or visual or both.

Slurry: a mixture of soil and/or rock cuttings, and drilling fluid.

Soil: all materials except those defined as rock, and excludes stone masonry, concrete, and other manufactured materials; includes rock fragments having an equivalent size less than 0.3 m in diameter.

Trenchless Installation: an underground method of constructing a passage open at both ends that involves installing a pipe. For the purpose of this specification, the pipe may be installed by any of the various methods defined herein such as Auger Jack & Boring, Pipe Jacking, Pipe Ramming, Directional Drilling, or using a tunnelling machine or hand mining methods.

Tunnelling: An underground method of constructing a passage using a tunnel boring machine (TBM), a microtunnel boring machine (MTBM) or hand mining using a shield to support the opening.

DESIGN AND SUBMISSION REQUIREMENTS

4.01 General

The Contractor's documentation, submission requirements and installation methods shall specifically consider and address the subsurface conditions at each pipe crossing as identified in the Foundation Investigation Report or elsewhere in the Contract Documents.

4.02 Working Drawings

Three copies of stamped working drawings for portal or shaft construction, primary liner, excavation, secondary lining, dewatering and groundwater control and grouting shall be submitted to the Contract Administrator (CA) at least one week prior to the commencement of the work for information purposes. All submissions shall bear the seal and signature of the Design Engineer and Design Checking Engineer. The Contractor shall have a copy of the stamped working drawings at the site during construction.

As a minimum, working drawings/details pertaining to the tunnel design and construction shall include the following (as appropriate):

a) Plans, Elevations and Details:

- A work plan outlining the materials, procedures, methods and schedule to be used to execute the work;
- A list of personnel, including backup personnel, and their qualifications and experience;
- A safety plan including the company safety manual and emergency procedures;
- The work area layout;
- An erosion and sediment control plan that includes a contingency plan in the event the erosion and sediment control measures fail;
- A drilling fluid management plan, if applicable, that addresses control of frac-out pressures, any potential environmental impacts and includes a contingency plan detailing emergency procedures in the event that the fluid management plan fails;
- Lighting, ventilation and fire safety details as may be required by applicable occupational health and safety regulations; and
- Excavated materials disposal plan.

b) Design Criteria:

- Primary liner design details, if applicable;
- Design assumption and material data when materials other than those specified are proposed for use; and
- Drill path design, details of alignment and alignment control, maximum curvature and reaming stages.

c) Materials:

- Certification from the manufacturer that the product furnished on the contract meets the specifications cited in the manufacturer's product specification and that the materials supplied are suitable for the application; and
- Material mixture for filling voids and installation procedures.

d) Upstream/Downstream Portal Installation Procedure:

- The access shaft or entry/exit pit details designed and stamped/signed by the Design Engineer, as applicable; and
- Face support and other temporary support details, if applicable.

e) Primary Liner/Secondary Liner Installation and Grouting Procedure:

- Excavation and pipe installation procedures, including methods to handle obstructions and prevent soil cave-in; and
- Details of tunnelling equipment/methods to be used for the works.

f) Excavation and Dewatering:

- Ground control/dewatering details, as applicable, describing the proposed method for control, handling, treatment, and disposal of water.

g) Monitoring Method:

The methods to be employed to monitor and maintain the alignment of the installation.

4.03 Site Survey

Prior to commencing the work, the Contractor shall, at each pipe location, lay-out the alignment and install settlement monitoring points.

4.04 Certificate of Conformance

The Contractor shall submit details of the sequence and method of construction to the Quality Verification Engineer for review, prepared and stamped by the Design Engineer. The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the construction procedures are in conformance with the requirements and specifications of the contract documents.

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations and prior to commencement of each subsequent operation for each pipe installation:

Site Surveying (as noted in Section 4.02)s
Excavation for pits including dewatering of excavations
Jacking/Ramming/Directional Drilling of Casing/Liner
Installation of the Product
Grouting Operations

Each Certificate of Conformance shall state that the work has been carried out in general conformance with the contract documents, specifications and/or stamped working drawings.

In addition, upon completion of the installation of the pipe at each location, the Contractor shall submit to the Contract Administrator a final Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate shall state that the pipe has been installed in general conformance with the Contractor's Submission and Design Requirements, stamped working drawings and contract documents.

The Design Engineer will not be permitted to carry out the work of the Quality Verification Engineer.

5. MATERIALS

5.01 Product

The product shall be concrete pipe or high density polyethylene pipe as specified.

5.02 Concrete

Concrete shall be according to OPSS.PROV 1350. The concrete strength shall be as specified in the Contractor's design submission.

5.03 Concrete Reinforcement

Steel reinforcing for concrete work shall be according to OPSS.PROV 1440.

5.04 Timber

Timber shall be sound, straight, and free from cracks, shakes and large or loose knots.

5.05 Grout

The Contractor shall submit the proposed grout mix design for grouts to be used for lubricating jacking pipe and for filling of voids and annular spaces. Purging grout shall consist of a mixture of one part Portland cement conforming to the requirements of CAN/CSA A5-93 and two parts mortar sand conforming to OPSS.PROV 1004 wetted with only sufficient water to make the mixture plastic.

5.06 Auger Jack & Bore Materials

5.06.01 Pipe Materials

Steel pipe shall conform with ASTM A252-93 welded joints suitable for jacking operations. The Contractor shall select pipe class for pipe jacking.

Concrete pipe as per OPSS.PROV 1820.

Fittings shall be suitable for and compatible with the class and type of pipe with which they will be used.

5.07 Pipe Ramming Materials

5.07.01 Pipe Materials

Steel pipe shall conform with ASTM A 252-93 welded joints.

New steel casing when specified shall be smooth wall carbon steel pipe according to ASTM A252-93 Grade 2.

Used steel casing can be used provided that the steel casing can resist the applicable static and dynamic loadings.

Pipe wall thickness shall be determined by the Contractor based on static and dynamic loads from traffic loading and anticipated ramming forces for selected pipe and driven pipe lengths. The wall thickness shall be increased as required to ensure the casing is not damaged during handling and installation. The pipe minimum wall thickness shall be as per Table 1 of OPSS 1802.

Pipe segments shall be determined by the Contractor.

Steel pipe joints shall be pressure fit type or welded.

All steel casing pipe shall be square cut.

Steel casing pipe shall have roundness such that the difference between the major and minor outside diameters shall not exceed 1% of the specified nominal outside diameter or 6 mm, whichever is less.

Steel casing pipe shall have a minimum allowable straightness of 1.5 mm maximum per metre of length.

5.07.02 Mill Certificates

For permanent casing, the Contractor shall submit to the Contract Administrator at the time of delivery one copy of the mill certificate, indicating that the steel meets the requirements for the appropriate standards for casings.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificate verified by testing by a Canadian laboratory. The laboratory shall be accredited by a Canadian National Accreditation Body to comply with the requirements of ISO/IEC Guide 25 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date and the signature of an authorized officer of the Canadian testing laboratory.

5.08 Directional Drilling Materials

5.08.01 Drilling Fluids

The drilling fluids shall be mixed according to the manufacturer's recommendations and be appropriate for the anticipated subsurface conditions.

5.08.02 Pipe Materials

High Density Polyethylene (HDPE) pipe as per OPSS 1840 shall be used in accordance with ASTM D3350.

The requirements for fittings shall be suitable for and compatible with the class and type of pipe with which they will be used and in according to CAN/CSA-B182.6 or ASTM F894.

The Contractor shall determine the required dimensional ratio (DR) of the HDPE pipe to support all subsurface conditions and hydrostatic pressures, and to withstand the grouting pressure and installation forces. The Contractor shall identify these forces in his submission requirements.

The Contractor's submission shall demonstrate, in conjunction with the manufacturer's specifications, that the heat resistance of the pipe material is sufficient to tolerate without damage the heat of hydration generated by grout curing.

Fittings shall be suitable for and compatible with the class and type of pipe with which they will be used.

Jointing of HDPE piping shall be completed by thermal butt fusion in accordance with manufacturer's recommended procedures and as outlined in the latest revision of ASTM D2657. All manufacturer's recommendations and procedures shall be followed during the jointing process.

Jointing of HDPE piping to other piping materials or appurtenances shall be completed using flanged connections.

5.09 Tunnelling Materials

5.09.01 Primary Liner

Tunnelling methods will require installation of a primary liner. The primary liner shall be designed by the

Contractor and the design/drawings shall be stamped/signed by the Design Engineer. The design shall be submitted to the Contract Administrator as specified herein.

5.09.02 Secondary Liner

Concrete or High Density Polyethylene Pipe shall be used according to the following requirements.

5.09.02.01 Concrete Pipe

Concrete pipe as per OPSS.PROV 1820 shall be used. The Contractor shall select the pipe class to withstand grouting pressure and installation forces. The Contractor shall identify these forces in his submission requirements.

Fittings shall be suitable for and compatible with the class and type of pipe with which they will be used.

5.09.02.02 High Density Polyethylene (HDPE)

High Density Polyethylene (HDPE) pipe as per OPSS 1840 shall be used in accordance with ASTM D3350.

The requirements for fittings shall be according to CAN/CSA-B182.6 or ASTM F894.

The Contractor shall determine the required dimensional ratio (DR) to withstand the grouting pressure and installation forces. The Contractor shall identify these forces in his submission requirements.

Fittings shall be suitable for and compatible with the class and type of pipe with which they will be used.

Jointing of HDPE piping shall be completed by thermal butt fusion in accordance with manufacturer's recommended procedures and as outlined in the latest revision of ASTM D2657. All manufacturer's recommendations and procedures shall be followed during the jointing process.

Jointing of HDPE piping to other piping materials shall be completed using flanged connections.

6. EQUIPMENT

6.01 Auger Jack & Bore Equipment

Pipe auger jack & bore equipment shall be determined by the Contractor and shall be identified in the submission requirements specified herein.

Specific details of the manner in which rock or boulders will be broken and removed from the face and the face will be protected to prevent soil loss into the liner shall be submitted to the Contract Administrator for information purposes prior to proceeding with the works.

6.02 Pipe Ramming Equipment

Pipe ramming equipment shall be determined by the Contractor and shall be identified in the submission requirements specified herein.

The pipe ramming hammer(s) shall be capable of driving the pipe casing from the drive pit through the existing subsurface conditions at the site.

Specific details of the manner in which rock or boulders will be broken and removed from the face and the face will be protected to prevent soil loss into the pipe shall be submitted to the Contract Administrator for information purposes prior to proceeding with the works.

6.03 Directional Drilling Equipment

6.03.01 General

The directional drilling equipment shall consist of a directional drilling rig and a drilling fluid mixing and delivery system of sufficient capacity to successfully complete the product installation without exceeding the maximum tensile strength of the product being installed.

6.03.02 Drilling Rig

The directional drilling rig shall:

- consist of a leak free hydraulically powered boring system to rotate, push, and pull hollow drill pipe into the ground at a variable angle while delivering a pressurized fluid mixture to a guidable drill head;
- contain a guidance system to accurately guide boring operations;
- be anchored to the ground to withstand the rotating, pushing, and pulling forces required to complete the product installation; and
- be grounded during all operations unless otherwise specified by the drilling rig manufacturer.

6.03.03 Drill Head

The drill head shall be steerable by changing its rotation, be equipped with the necessary cutting surfaces and drilling fluid jets, and be of the type for the anticipated subsurface conditions,

6.03.04 Guidance System

The guidance system shall be setup, installed, and operated by trained and experienced personnel. The operator shall be aware of any magnetic or electromagnetic anomalies and shall consider such influences in the operation of the guidance system when a magnetic or electromagnetic system is used.

6.03.05 Drilling Fluid Mixing System

The drilling fluid mixing system shall be of sufficient size to thoroughly and uniformly mix the required drilling fluid.

6.03.06 Drilling Fluid Delivery System

The delivery system shall have a means of measuring and controlling fluid pressures and be of sufficient flow capacity to ensure that all slurry volumes are adequate for the length and diameter of the final bore and the anticipated subsurface conditions. Connections between the delivery pump and drill pipe shall be leak-free.

6.04 Tunnelling Equipment

Tunnelling equipment shall be determined by the Contractor and shall be identified in the submission requirements specified herein.

Specific details of the manner in which rock or boulders will be broken and removed from the tunnel face

shall be submitted to the Contract Administrator information purposes. Use of rock fracturing chemicals shall only be considered subject to a field demonstration satisfactory to the Ministry prior to its use. Use of explosives is prohibited.

7. CONSTRUCTION

7.01 General

The Contractor shall notify the Contract Administrator at least 48 hours in advance of starting work. The proposed method of pipe installation to be used by the Contractor shall be submitted to the Contract Administrator for information purposes prior to commencing the work and shall be subject to the limitations presented in the following subsections.

7.01.01 Layout, Alignment and Depth Control

The location of the installation shall be established from the lines, elevations and tolerances specified in the Contract Documents. The pipe installation shall be to the horizontal and vertical alignments specified in the Contract Drawings. Deviations from location, alignment, grades and/or invert levels shall be corrected by the Contractor at no cost to the Ministry.

All reference points necessary to construct the pipe installation and appurtenances shall be laid out.

The Contractor shall calibrate tracking and locating equipment at the beginning of each work day, and shall monitor and record the alignment and depth readings provided by the tracking system at every 5 m in normal conditions and every 2 m where precise alignment control is necessary;

The Contract Administrator shall be provided with the assistance and access necessary to check the layout of the pipe installation and associated appurtenances.

All excavations shall be carried out in accordance with the Occupational Health and Safety Act (OHSA) of Ontario.

For directional drilling, the contractor shall ensure that during pilot hole drilling the maximum degree of deviation or “dog-leg” shall be 2.5 degrees per 9m drill pipe length. Any deviation exceeding 2.5 degrees will necessitate a pull-back and straightening of the alignment at the Contractor’s sole expense. The pilot hole exit location shall be within 0.5m of the target location.

7.01.02 Construction Shafts

Construction shafts shall be specified in the Contractor's submission. The boundaries and protection of these shall be as required to contain all disturbances to areas outside of the ESA limits.

Shafts shall be maintained in a drained condition.

A minimum 2.4 m high secure fence shall be installed around the perimeter of the construction shaft area with gates and truck entrances. The fence shall be removed on completion of the work.

7.01.03 Protection Systems

The construction of all protection systems shall be according to OPSS.PROV 539. Where the stability, safety, or function of an existing roadway, watercourse, other works, proposed works or ESA’s may be impaired due

to the method of operation, protection shall be provided. Protection may include sheathing, shoring, and piles where necessary to prevent damage to such works or proposed works.

7.01.04 Settlement or Heave

Any disturbance to the ground surface (settlement or heave) as a result of the pipe installation shall be immediately corrected by the Contract, at no additional cost to the Ministry.

7.01.05 Stability of Excavation

The construction methods, plant, procedures, and precautions employed shall ensure that excavations are stable, free from disturbance, and maintained in a drained condition.

The construction methods, plant, and materials employed shall prevent the migration of soil and/or rock material into the excavation from adjacent ground.

7.01.06 Preservation and Protection of Existing Facilities

Preservation and protection of existing facilities shall be according to OPSS 491.

Minimum horizontal and vertical clearances to existing facilities as specified in the Contract Documents shall be maintained. Clearances shall be measured from the nearest edge of the largest cut diameter required to the nearest edge of the facility being paralleled or crossed.

Existing underground facilities shall be exposed to verify its horizontal and vertical locations when the outlet pipe path comes within 1.0 m horizontally or vertically of the existing facility. Existing facilities shall be exposed by non-destructive methods. The number of exposures required to monitor work progress shall be as specified in the Contract Documents.

7.01.07 Transporting, Unloading, Storing and Handling Materials

Manufacturer's handling and storage recommendations shall be followed.

7.01.08 Trenching, Backfilling and Compacting

Trenching, backfilling, and compacting for entry and exit points or other locations along the pipe path shall be according to OPSS 401.

7.01.09 Support Systems

Support systems shall be according to OPSS 404.

If any open excavation will encroach into the highway embankment the protection system shall satisfy the requirements for Performance Level 2 as specified in OPSS.PROV 539.

7.01.10 Dewatering

The work of this Section includes control, handling, treatment, and disposal of groundwater. The Contractor shall review the foundation investigation report for reference to soil and groundwater conditions on the project site and plan a dewatering scheme accordingly.

The Contractor shall control groundwater inflows to excavations to maintain stability of surrounding ground, to prevent erosion of soil, to prevent softening of ground exposed in the excavation, and to avoid interfering with execution of the work.

The Contractor shall maintain excavations free of standing water at all times during excavation, including while concrete is curing.

Should water enter the excavation in amounts that could adversely affect the performance of the work or could cause loss of ground, the Contractor shall take immediate steps to control the inflow.

The Contractor is alerted that seepage zones of perched water within the fill materials should be expected, particularly where granular materials are excavated.

Dewatering shall be according to OPSS 517.

7.01.11 Removal of Boulders

The Contractor is alerted that cobbles and boulders should be anticipated in the soil deposits at the site. Accordingly, the Contractor shall address the removal of cobbles and boulders in the proposed method of construction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered.

7.01.12 Record Keeping

Verification record requirements of the alignment and depth of the installation shall be as specified in the Contract Documents. A copy of the verification records shall be given to the Contract Administrator at the completion of the installation.

7.01.13 Testing

Testing of the product installation shall consist of verifying the specified grade between the two ends of the pipe and passing of water from the inlet end of the pipe to the outlet end to confirm gravity flow conditions.

7.01.14 Management and Disposal of Excess Material

Management and disposal of excess material shall be according to OPSS 180. Satisfactory re-usable excavated material required for backfill shall be separated from unsuitable excavated material.

7.01.15 Site Restoration

Site restoration shall be according to OPSS 492.

7.01.16 Supervision

A qualified individual, who is experienced in the pipe installation by trenchless methods shall supervise the work at all times.

7.02 Auger Jack & Bore Installation

7.02.01 Method of Installation Procedure

The installation procedure to be used shall be subject to the following limitations:

Hydraulically operated jacks of adequate number and capacity shall be provided to ensure smooth and uniform advancement without over-stressing of the pipe.

A suitably padded jacking head or collar shall be provided to transfer and distribute jacking pressure uniformly over the entire end bearing area of the pipe.

The jacking pipe shall be fully supported in the jacking pit at the specified line and grade.

Selection of the excavation method and jacking equipment shall take into consideration the conditions at each pipe crossing.

7.02.02 Pipe Installation

Concrete pipe joints shall be water tight and according to OPSS.PROV 1820 and must withstand jacking forces, determined by the Contractor.

During the jacking of the liner the space between the liner and the wall of the excavation shall be kept filled with bentonite slurry. Upon completion of jacking, the space between the liner and the wall of the excavation shall be filled with grout.

The annular space between the liner and the product shall be fully grouted with a water tight, expandable and stable grout.

7.03 Pipe Ramming Installation

For pipe ramming installation the following requirements apply:

Only smooth walled steel pipe shall be used. But welding of pipe joints shall conform to CAS W59.

Ramming equipment of adequate capacity shall be provided to ensure smooth and uniform advancement without overstressing of the pipe. Delays shall be avoided between ramming operations.

A ramming head shall be provided to transfer and distribute jacking pressure uniformly over the entire end bearing area of the pipe.

Two or more lubricated guide rails or sills shall be provided of sufficient length to fully support the pipe at the specified line and grade in the ramming pit. Pipe shall be installed to the line and grade specified.

Following installation of the liner pipe, all material shall be removed from the pipe to the satisfaction of the Contract Administrator. Any voids remaining between the pipe and the excavation wall shall be grouted as soon as the pipe is rammed. The annular space between the liner pipe and the product shall be fully grouted with a water tight, expandable and stable grout.

7.04 Directional Drilling Installation

7.04.01 General

When strike alerts are provided on a drilling rig, they shall be activated during drilling and maintained at all times.

7.04.02 Site Preparation

The work site shall be graded or filled to provide a level working area for the drilling rig. No alterations beyond what is required for DD operations are to be made. All activities shall be confined to designated work areas.

7.04.03 Pilot Bore

The pilot bore shall be drilled along the bore path in accordance with the grade, alignment, and tolerances as indicated on the Contractor's submitted drilling plan to ensure that the product is installed to the line and grade shown on the Contract Drawings. The Contractor's methods shall take into consideration the conditions at each crossing within the pipe alignment and shall be suitable to advance through such obstructions such as cobbles and boulders and address the potential for deflection off these obstruction and/or soil conditions.

In the event the pilot bore deviates from the submitted path, the Contract Administrator shall be notified. The Contract Administrator may require the Contractor to pullback and re-drill from the location along the bore path before the deviation.

In the event that a drilling fluid fracture, inadvertent returns, or loss of circulation occurs during pilot bore drilling operations, the Contract Administrator shall be advised of the event and action shall be taken in accordance with the Contractor's submitted contingency plan.

At the entry and exit points, there is potential for ravelling of the existing soil, fill and or weathered rock areas along the alignment. This is conventionally addressed by the use of drilling fluid. However, casing may be required. The Contractor's methods shall take into consideration the potential need to install sections of casing to manage ravelling at or near ground surface.

If a drill hole beneath the highway must be abandoned, the hole shall be backfilled with grout or bentonite to prevent future subsidence.

The Contractor shall maintain drilling fluid pressure and circulation throughout the DD process, including during the initial pilot bore and during the reaming process.

The Contractor shall at all times and for the entire length of the installation alignment be able to demonstrate the horizontal and vertical position of the alignment, the fluid volume used, return rates and pressures.

7.04.04 Drilling Fluid Fracture (Frac-Out)

In order to reduce the potential for hydraulic fracturing of the hole during directional drilling, a minimum depth of cover of 5m is normally maintained between the pipe and the ground surface. Sections of the pipe close to the exit pit with less than 5m cover shall be cased. The Contractor shall ensure that drilling fluid pressures are properly set and controlled to prevent frac-out, for the depth of cover available between the bottom of the pavement structure (bottom of the subbase material) and the top of the bore.

Since fluid loss normally occurs in fault zones, fracture zones, or seams of coarse material, fluid migration does not always gravitate to the surface, thus making detection difficult. Once a fluid loss is detected, the Contractor shall halt operations immediately and conduct a detailed examination of the drill path and implement measures to mitigate fluid loss. If no surface migration is evident, resume operation while paying particular attention to fluid monitoring.

In the event of a fluid migration to the surface occurring, the Contractor shall halt all operations immediately, isolate the migration site, and recover fluids. Once the fracture is controlled, continue drilling operations with

the operator paying particular attention to the fracture points

7.04.05 Reaming

The bore shall be reamed using the appropriate tools to a diameter at least 50% greater than the outside diameter of the product.

7.04.06 Product Installation

7.04.06.0 General

The product shall be jointed according to manufacturer's recommendations. The length of the product to be pulled shall be jointed as one length before commencement of the continuous pulling operation.

The product shall be protected from damage during the pullback operation.

The minimum allowable bending radius for the product shall not be exceeded.

Product shall be allowed to recover before connections to new or existing facility are made. Product recovery time shall be according to manufacturer's recommendations.

7.04.06.02 Pullback and Grouting

After successfully reaming the bore to the required diameter, the product shall be pulled through the bore path. Once the pullback operation has commenced, it shall continue without interruption until the product is completely pulled into bore unless otherwise approved by the Contract Administrator.

A swivel shall be used between the reamer and the product being installed to prevent rotational forces from being transferred to the product. When specified in the Contract Documents, a weak link or breakaway connector shall be used to prevent excess pulling force from damaging the product.

The product shall be inspected for damage where visible at excavation pits and where it exits the bore. Any damage noted shall be rectified to the satisfaction of the Contract Administrator,

The pull back and reaming operations shall not exceed the fluid circulation rate capabilities. Reaming and back pulling operations shall be planned to insure that, once started, all reaming and back pulling operations are completed without stopping and within the permitted work hours.

The space between the pipe and the excavation walls shall be filled with grout.

7.05 Tunnelling Installation

7.05.01 General

The method of tunnelling shall be selected by the Contractor and shall be submitted to the Contract Administrator prior to commencement of the work for information purposes.

Excavation of native soil and fill shall be done in a manner to control groundwater inflow to the excavation and to prevent loss of ground into the excavation.

Methods of excavating the tunnel shall be capable of fully supporting the face and shall accommodate the

removal of boulders and other oversize objects from the face. Continuous ground support shall be maintained during excavation.

As the excavation progresses, the Contractor shall continuously monitor (every 2 m) indications of support distress, such as cracking, deflection or failure of support system and subsidence of ground near the excavation.

The Contractor shall advance the ventilation system as a regular part of the normal excavation cycle.

The Contractor shall provide lighting in accordance with OSHA requirements for the entire length of the tunnel.

The tunnel is to be kept sufficiently dry at all times to permit work to be performed in a safe and satisfactory manner.

The Contractor shall maintain clean working conditions at all times in tunnels.

In the event that excavation threatens to endanger personnel, the Work, or adjacent property, the Contractor shall cease excavation. The Contractor shall then evaluate methods of construction and revise as necessary to ensure the safe continuation of the work.

The Contractor shall maintain tunnel excavation line and grade to provide for construction of final lining within specified tolerances.

7.05.01 Tunnelling Method

The tunnelling method shall be suitable to provide face support in changing ground conditions that may be encountered during the progress of the work. The selection of the tunnelling method should consider the soil conditions at each pipe crossing and the presence of obstructions, such as cobbles and boulders, with respect to the tunnel alignment.

7.05.02 Primary Liner (Support System)

Primary support systems shall prevent deterioration, loosening, or unravelling of ground surfaces exposed by excavation.

The primary liner support system shall be designed and installed to achieve the intended performance requirements.

Primary liner support system shall maintain the safety of personnel, minimize ground movement into the excavation, ensure stability and maintain strength of ground surrounding the excavation.

The primary liner shall be designed to support all subsurface conditions and hydrostatic pressures and to withstand any additional loads caused by installation and grouting, and shall ensure that no ground loading or other loading will be placed on the new work until after design strength has been reached.

The primary liner shall be installed so that the exterior is as tight as possible to the excavated surface of the tunnel and allows the placement of the full design thickness of the secondary lining.

Primary support systems shall be compatible with the encountered ground conditions, with the method of excavation, with methods for control of water, and with placement of the permanent lining.

All voids between the primary lining and the surface of the excavation shall be filled with cement grout. If an unexpanded liner is used, the space outside the liner plates shall be grouted at least daily.

7.05.03 Secondary Liner

7.05.03.01 Placing of Grout

The void outside the finished secondary liner shall be filled with cement grout according to the Contractor's submission.

Grout shall not be placed until the lining has achieved 85% of its specified strength or 30 MPa. Grouting shall be limited to such sequences and programs as are necessary to avoid damaging any part of the works or any other structure or property.

7.06 Instrumentation and Monitoring

Settlement Monitoring

The work specified in this section includes furnishing and installing instruments for monitoring of settlement and ground stability.

Surface settlement markers (SSMs) and settlement points (SPs) for monitoring ground stability shall be installed at the pavement/ground surface level as detailed in the settlement monitoring plan. Two Benchmarks (BM) shall be installed by the contractor as described in this section.

The equipment and procedures used for settlement monitoring during construction must be capable of surveying the settlement point elevations to within 2 mm of the actual elevation.

BENCHMARKS

Number and Locations of Benchmarks The minimum number of Benchmarks for this crossing is two, and should be located in the field such that:

- Direct sighting is possible from all instruments to at least one Benchmark.
- Benchmarks are located in an area that will not experience ground movement as a result of the boring operations.
- Benchmarks are located in such a way to minimize interference with and damage by construction activities.
- The rod anchor elevation should be determined based on the subsurface conditions at each location and shall extend approximately 6 m into soil, or into soils having Standard Penetration Test 'N' value of greater than 50 blows per 0.3 m of penetration, whichever is less.

-

Materials for Benchmarks

The contractor shall supply all materials and equipment required for the installation of the Benchmarks, as follows:

Rod: The contractor shall supply a steel pipe, Schedule 40, with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

Sand: The contractor shall supply clean, washed sand for backfilling around the friction reducing sleeve.

Rod Anchor Grout: The contractor shall supply cement-bentonite grout suitable for anchoring the rod at the bottom of the borehole

Friction-Reducing Sleeve: The contractor shall supply a friction-reducing sleeve consisting of Schedule 40 — 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

Installation of Benchmarks

The Contractor shall install Benchmarks in accordance with the following:

Borehole: The borehole shall be advanced to the required depth using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction-reducing sleeve and rod anchor. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

Rod: The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

Rod Anchor: The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 300 mm of the borehole to form a concrete/soil anchor. Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 25 mm to 50 mm length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on. The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

Friction-Reducing Sleeve: The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

SURFACE SETTLEMENT MARKERS (SSM)

The Surface Settlement Markers shall be installed on/within the pavement in a way such that they can be surveyed without the need for traffic protection.

The contractor shall install SSMs at the approximate locations shown on the Settlement Monitoring Instrumentation Locations and Details drawing. In general, the Surface Settlement Markers shall be located on the pavement of the highway and shall be installed to be in line with the pavement marking lines where installed within the travelled lanes in order to reduce visual distraction to drivers.

Materials for Surface Settlement Markers (SSM)

The contractor shall supply all materials and equipment required for the installation of the Surface Settlement Markers, as follows:

Hex-Head Bolt: The contractor shall supply hardened steel hex-head bolt treated/coated to resist corrosion, with an exposed face of approximately 6 mm height. The face of the hex-head bolt shall be such that a single survey point can be clearly identified and returned to including the use of a scored line or paint marking, to eliminate the need for traffic protection, as may be required depending on the survey equipment used.

Washer: An appropriately sized washer shall be placed between the pavement surface and the head of the hex-head bolt, to minimize the potential for the head to be pushed into the pavement due to traffic travelling over the bolt.

Epoxy: The contractor shall use an appropriate high-strength, rapid-curing epoxy to secure the hex-head bolts into the asphalt or concrete pavement. The contractor shall ensure that the epoxy selected is non-expansive to ensure that the bolt is not pushed up from the hole.

Installation of Surface Settlement Markers (SSM)

The contractor shall install Surface Settlement Markers at the approximate locations shown on the Settlement Monitoring Instrumentation Locations and Details drawing following the typical installation details as shown in addition to what is stated below:

The SSMs should be installed on the painted divider lines separating the driving lanes (at the approximate spacing shown on the drawings). The hex-head bolts shall be rigidly affixed so as not to move relative to the surface to which it is attached. The hex-head bolts shall be installed in a pre-drilled hole, with a washer placed on the surface of the asphalt pavement below the head of the bolt, and the assembly affixed into place using an appropriate high-strength, rapid-curing, non-expansive epoxy.

SETTLEMENT POINTS (SP)

Materials for Settlement Points (SP)

The Geotechnical Monitoring Consultant shall supply all materials and equipment required for the installation of the Settlement Points, as follows:

Rod: The Geotechnical Monitoring Consultant shall supply a steel rod as shown on the Settlement Monitoring Instrumentation Locations and Details drawing, supplied in lengths as required to complete the installation. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to, where required to eliminate the need for traffic protection, and as may be required depending on the survey equipment used.

Sand: The contractor shall supply clean, washed sand suitable for backfilling around the friction reducing sleeve

Rod Anchor Concrete/Grout: The contractor shall supply concrete or cement-bentonite grout suitable for anchoring the rod.

Friction-Reducing Sleeve: The contractor shall supply a friction-reducing sleeve consistent with the detail shown on the Settlement Monitoring Instrumentation Locations and Details Drawing.

Installation of Settlement Points (SP)

The contractor shall install Settlement Points at the approximate locations shown on the instrumentation and monitoring plan following the typical installation details as shown in addition to what is stated below:

A borehole shall be advanced to the required depth using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction-reducing sleeve and rod anchor. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 200 mm of the borehole to form a concrete/soil anchor. Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 25 mm to 50 mm length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on. The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

General Conditions

The Contractor shall install all instruments a minimum of one week prior to the start of works. The contractor shall determine if an elevated platform is required to be able to have direct siting to the benchmark(s) and the roadway/railway instruments and construct an appropriate platform for use by the surveyor.

All survey work shall be performed by a licensed OLS (Ontario Land Surveyor) and/or OLIP (Ontario Land Information Professional) registered with the Association of Ontario Land Surveyors.

The contractor shall protect and avoid damaging instrumentation during construction. Instrumentation that is damaged as a result of the Contractor's operation shall be repaired or replaced by the Contractor within one business day. The costs for replacement/repair shall be borne by the Contractor.

At the completion of the job, the contractor shall decommission all instruments installed during the course of the work. For Benchmarks and Settlement Points, the steel rod shall be removed and the hole backfilled with fine sand up to the original ground surface. For Surface Settlement Markers, the hex head bolt shall be removed and the hole filled with a Crack Sealant, Rubberized Asphalt, Hot Poured, from an approved source

on the designated source materials list.

Reading and Reporting Requirements

The Contractor shall submit to the Contract Administrator a site plan showing the locations of the monitoring points, a geodetic survey of the settlement monitoring points including station, offset and elevation recorded at the following time intervals:

- Three consecutive readings at least one week prior to commencement of the work (Baseline Reading)
- Once per shift during tunnelling operations period
- Weekly after completion of the work for one month, or until such time at which all parties agree that further movement has stopped.

All readings shall be submitted to the Contract Administrator for information purposes within 24 hours of the reading being taken. Each report shall include all survey data collected in tabular and graphical format as plots of time versus settlement in comparison to survey data collected prior to commencement of the work.

The contractor shall submit biweekly reports to the MTO Pavements and Foundations office during the monitoring period for information purposes. The biweekly report shall consist of a site plan showing the locations of the monitoring points, a geodetic survey of the settlement monitoring points including station, offset and elevation. The report shall also include all survey data collected in tabular and graphical format as plots of time versus settlement in comparison to survey data collected prior to commencement of the work. The biweekly reports shall be submitted in digital PDF format by e-mail to the MTO Foundations representative; Darren Berwick, at darren.berwick@ontario.ca

7.07 Criteria for Assessment of Roadway Subsidence/Heave

Based on the monitoring of ground movement as specified, the following represents trigger levels that define magnitude of movement and corresponding action:

- Review Level: If a maximum value of 10 mm relative to the baseline readings is reached, the Contractor shall review or modify the method, rate or sequence of construction or ground stabilization measures to mitigate further ground displacement. If this Review Level is exceeded, the Contractor shall immediately notify the CA and review and discuss response actions. The Contractor shall submit a plan of action to prevent Alert Levels from being reached. All construction work shall be continued such that the Alert Level is not reached.

- Alert Level: If a maximum value of 15 mm relative to the baseline readings is reached, the Contractor shall cease construction operations, inform the Contract Administrator and execute pre-planned measures to secure the site, to mitigate further movements and to assure safety of public and maintain traffic. No construction shall take place until all of the following conditions are satisfied:

- The cause of the settlement has been identified.
- The Contractor submits a corrective/preventive plan.
- Any corrective and/or preventive measure deemed necessary by the Contractor is implemented.
- The CA deems it is safe to proceed.

9. MEASUREMENT FOR PAYMENT

Measurement shall be by Plan Quantity Payment as may be revised by Adjusted Plan Quantity Payment in metres, following along the centre line of the pipes from centre to centre of maintenance holes or chambers (catch basins) or from/to the end of the pipe where no maintenance hole or chamber is installed, of the actual length of pipe installed by trenchless methods.

10. BASIS OF PAYMENT

Payment at the contract price shall be full compensation for all labour, equipment and materials required for excavation (regardless of material encountered), dewatering, sheathing and shoring, supply and installation of pipe liners, settlement instrumentation and monitoring, site restoration, and all other work necessary to complete the installation as specified.

Payment for the rigid or flexible pipe conduits installed inside the pipe liners shall be paid separately under the appropriate tender items.

Where a protection system is made necessary because of the Contractor's operations (e.g. choice of trenchless installation method), the cost shall be included in this item and shall be full compensation for all labour, equipment and materials required to carry out the work including subsequently removing the temporary protection system and performing any necessary restoration work.

Payment for connecting intercepted drains and service connections shall be made on the following basis:

- a) Where such drains and service connections are shown on the contract drawings the cost of connections shall be included in the contract price for pipe installation.
- b) Where such drains and service connections are not shown on the contract drawings, the cost of connections will be considered an allowable extra to the contract.

Payment for removal of boulders/obstructions greater than an equivalent 0.3 m in diameter shall be on a time and materials basis. The Contractor shall inform the Contract Administrator when boulders/obstructions are encountered and prior to removal to allow for proper and accurate tracking of time and material charges.

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

Cobbles and Boulders should be anticipated to be encountered within the embankment fill, native sand and gravel deposits and the native glacial till soils at the site, as inferred present in Boreholes 18-3 and 18-4 and as cored in Boreholes 18-1, 18-2, 18-5 and 18-6. Although not encountered in the boreholes, debris consisting of abandoned temporary works associated with the original construction; logs, stumps and brush from the clearing and grubbing operations; and cobbles and boulders buried in the fill may also be present within the existing fill materials and near the fill / native soil interface. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for trenchless installation of the new culvert.

Basis of Payment

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

END OF SECTION

DEWATERING AND CONTROL OF GROUNDWATER - Item No.

Non-Standard Special Provision

The contractor shall be alerted that the groundwater table was encountered between about Elevation 239.2 m and 234.7 m at the proposed new culvert location. It is estimated that the base of temporary excavations for the entry and exit shafts may be below the groundwater level as measured in piezometers installed in Boreholes 18-1 and 18-4. Dewatering within the shaft excavations and along the tunnel alignment (depending on the construction method chosen) may be required and the excavation shall be kept stable during the work. It is considered that groundwater control using sumps may not be adequate in the saturated granular fill and native sand and gravel deposits present at the site. It may be necessary to use well points, eductors or the like. The groundwater level should be lowered to at least 0.5 m below the base of the excavations. The excavations should be protected from ingress of surface water. The Contractor is to design and install appropriate measures to control the groundwater during construction.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials required to do the work.

END OF SECTION

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

Special Provision No. 517F01

July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

Section 517.01 of OPSS 517 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the design, operation, and removal of a dewatering or temporary flow passage system or both to control water during construction, and the control of the water prior to discharge to the natural environment and sewer systems.

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

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

A dewatering or temporary flow passage system or both shall be designed to control water at the locations specified in the Contract Documents and at any other location where a system is necessary to complete the work. The design of the system shall be sufficient to permit the work at each location to be carried out as specified in the Contract Documents.

Subsection 517.04.01 of OPSS 517 is further 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.

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

Table A

IDF Curve Location	Latitude: *		Longitude: *			
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m ³ /s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
**	***	****	****	****	****	*****
Dewatering Systems						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)				Design Engineer Requirements (Note 1)	
Sixteen Mile Creek	N/A				Yes	
<p>Note:</p> <p>1. "Yes" means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. "No" means a minimum experience level is not required for the design Engineer and design-checking Engineer.</p> <p>2. "N/A" indicates a preconstruction survey is not required.</p>						



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