



## Foundation Investigation and Design Report

*McIntyre Creek Culvert (Site No. 30-522/C) Retaining/Gabion Wall Replacement, Highway 26, County of Simcoe, Ontario, W.P. No.: 2444-15-00*

Submitted to:

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# PART A

FOUNDATION INVESTIGATION REPORT  
MCINTYRE CREEK CULVERT RETAINING/GABION WALL REPLACEMENT  
HIGHWAY 26, COUNTY OF SIMCOE, ONTARIO  
W.P. 2444-15-00



## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation investigation and engineering services for the proposed replacement of the existing gabion retaining wall at the south end of the McIntyre Creek culvert (MTO Structure Site No. 30-522/C), located on Highway 26 in the County of Simcoe, Ontario, under W.P. 2444-15-00.

The Terms of Reference for the foundation engineering services are outlined in MTO's Work Item Order No. 2016-E-0029-004, dated August 2017, which forms part of the Consultant's Assignment for the Central Region Large Value Retainer under Agreement No. 2016-E-0029-004.

## 2.0 SITE DESCRIPTION

The McIntyre Creek culvert is located along Highway 26 between Stayner and Sunnidale Corners, about 135 m east of Sideroad 3&4 Sunnidale, in the County of Simcoe, Ontario. The site is surrounded by farmland, with the ground generally flat-lying. The McIntyre Creek channel is at approximately Elevation 197.8 m at the culvert site, and the creek water level within the culvert was at approximately Elevation 197.9 m on November 21, 2017. The natural ground surface to the west and east of the creek channel rises to about Elevation 200 m to 202 m. The Highway 26 grade is at about Elevation 203.7 m at the culvert site.

The approximately 3 m to 6 m high embankment side slopes at this location are oriented slightly steeper than 2 horizontal to 1 vertical (2H:1V), with sheet pile wingwalls and gabion walls present on both sides of the south end of the culvert. The existing gabions walls are about 1 m high and run parallel to McIntyre Creek as shown in the following photographs. The southeast gabion wall was partially collapsed at the time of the site visit.



*Photograph 1: Southwest gabion wall, looking south along McIntyre Creek from Highway 26*





*Photograph 2: Southeast gabion wall, looking south along McIntyre Creek from Highway 26*

The McIntyre Creek culvert is a flat bottomed, corrugated steel pipe (CSP) arch culvert that was constructed in 1976. The culvert is approximately 43.3 m long, 5.1 m wide and 3.0 m high, with about 4 m of fill above the culvert. The creek water flow is from south to north under Highway 26.

### 3.0 INVESTIGATION PROCEDURES

The field work at the McIntyre Creek Culvert was carried out on November 20 and 21, 2017, during which time two boreholes (designated as Boreholes 17-1 and 17-2) were advanced at the site. The borehole locations shown on Drawing 1 and the borehole records are presented in Appendix A. Borehole 17-1 was advanced atop the embankment in the eastbound lane of Highway 26, and Borehole 17-2 was advanced at the toe of the embankment beside the southeast gabion wall.

Borehole 17-1 was drilled using 203 mm outer diameter hollow-stem augers with a D90 truck-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. Borehole 17-2 was advanced using 80 mm outer diameter casing with a portable tripod drill rig supplied and operated by OGS Inc. of Almonte, Ontario. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in Borehole 17-1 and driven by a manual hammer in Borehole 17-2 in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)<sup>1</sup>.

Borehole 17-1 was advanced through the road embankment to a depth of about 18.9 m below existing ground surface. Borehole 17-2 was advanced at the base of the embankment beside the southeast gabion wall and terminated upon casing refusal at a depth of about 8.2 m below existing ground surface.

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in Borehole 17-2 to permit monitoring of the water level. The

<sup>1</sup> ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

installed piezometer consist of a 20 mm diameter PVC pipe, with a 1.5 m slotted screen sealed within a filter sand pack with the bottom of the well about 8 m below ground surface within the borehole. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to the ground surface with bentonite pellets. Piezometer installation details and water level readings are described on the borehole record in Appendix A. Borehole 17-1 was backfilled to ground surface with bentonite and sealed at the surface with cold patch asphalt upon completion, in accordance with Ontario Regulation 903, Wells (as amended).

The field work was monitored on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, directed the sampling and in situ testing operations, logged the boreholes and examined the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further visual review and geotechnical laboratory testing on selected samples, consisting of natural moisture content, Atterberg limits and grain size distribution analyses conducted in accordance with MTO and / or ASTM Standards as applicable. The results of this testing program are shown on the borehole records as well as on the laboratory test figures contained in Appendix B.

The borehole locations were marked in the field by Golder personnel relative to the existing culvert, gabion walls and other site features. The locations given in the Record of Borehole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	MTM NAD83		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
17-1	4,920,915.2 (44.426970)	261,849.4 (-80.039439)	203.7	18.9
17-2	4,920,900.0 (44.426835)	261,865.9 (-80.039231)	198.5	8.2

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

This section of Highway 26 is located in the Stayner Clay Plain within the Simcoe Lowlands physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)<sup>2</sup>.

The Simcoe Lowlands physiographic region covers the central portion of the County of Simcoe. Following the retreat of the last glacial ice sheet, the lowland was flooded by the now extinct post-glacial Lake Algonquin. This past lacustrine environment is marked by deep sand, silt and clay beds overlying glacial ground moraine material. The Stayner Clay Plain is partly a bevelled till plain with pebbly till appearing at or near the surface. Some areas are floored with deeper beds of clay, while in other places the clay is covered with up to several feet of sand.

<sup>2</sup> Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.



## 4.2 General Overview of Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory tests are provided on the borehole records in Appendix A. The results of the in situ field tests (i.e., SPT “N”-values) as presented on the borehole records, on the stratigraphic profiles and in Section 4 are uncorrected.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profiles on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected; however, the factual data presented on the borehole records governs any interpretation of the site conditions.

In general, the native subsurface soils encountered near the existing McIntyre Creek Culvert consist of predominantly non-cohesive deposits of sand and silt separated by interlayers of clayey silt.

### 4.2.1 Fill

Borehole 17-1 was advanced through the Highway 26 road surface and encountered approximately 200 mm of asphalt at the surface, overlying an approximately 5.4 m thick layer of fill material, extending to Elevation 198.1 m.

Standard Penetration Tests (SPT) “N”-values measured within the non-cohesive portion of the fill were 18 blows and 34 blows per 0.3 m of penetration, indicating that the non-cohesive fill has a compact to dense relative density. SPT “N”-values measured within the cohesive portion of the fill range from 3 blows to 12 blows per 0.3 m of penetration, suggesting that the cohesive fill has a soft to stiff consistency.

The top 1.2 m of the fill layer consists of gravelly sand containing some silt and trace cobble fragments, overlying 4.2 m of clayey silt with sand fill containing trace gravel and trace organics. A grain size distribution test was carried out on one sample of the cohesive fill layer and the results are shown on Figure B1 in Appendix B. An Atterberg limits test was carried out on one sample of this cohesive fill layer and measured a liquid limit of about 28 per cent, a plastic limit of about 15 per cent, and a plasticity index of about 14 per cent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, indicate that the cohesive fill layer is clayey silt of low plasticity. A natural water content of 6 per cent was measured on one sample of the gravelly sand, while natural water contents of about 22 and 31 per cent were measured on two samples of the clayey silt with sand fill.

### 4.2.2 Surficial Clayey Silt

A surficial clayey silt deposit was encountered in both boreholes. In Borehole 17-1 the deposit was encountered directly beneath the cohesive fill material and is approximately 0.6 m thick, extending to Elevation 196.5 m. In Borehole 17-2, the deposit was encountered immediately below ground surface and is 1.5 m thick, extending to Elevation 197.1 m.

SPT “N”-values of 2 blows, 37 blows and 76 blows per 0.3 m of penetration were measured within the surficial clayey silt deposit. The SPT “N”-value of 2 blows per 0.3 m of penetration was measured immediately below surface in Borehole 17-2, and this portion of the deposit is considered to have a very soft to soft consistency. The remaining SPT “N”-values suggest a hard consistency.

The deposit consists of clayey silt containing trace to some sand, as well as trace to some rootlets in Borehole 17-2. A grain size distribution test was carried out on one sample of the clayey silt deposit and the results are shown in Figure B3 of Appendix B. An Atterberg limits test was carried out on one sample of the surficial clayey silt and measured a liquid limit of about 22 percent, a plastic limit of about 16 per cent and a plasticity index of

about 6 per cent. These results, which are plotted on a plasticity chart on Figure B4 in Appendix B, indicate that the surficial cohesive deposit is a clayey silt of low plasticity. The natural water content measured on two samples of this deposit are about 14 and 21 per cent.

### 4.2.3 Silt to Sand Deposit

A deposit consisting of interlayers of silt containing trace to some sand, and sand containing trace to some silt, was encountered underlying the surficial clayey silt deposit in both boreholes. Borehole 17-1 terminated within this deposit after penetrating it for approximately 11.7 m, to Elevation 184.8 m. Borehole 17-2 extended through 5.7 m of this deposit before encountering a lower clayey silt layer at Elevation 191.3 m.

SPT “N”-values ranging from 22 blows to greater than 100 blows per 0.3 m of penetration were recorded within the silt to sand deposit, indicating a compact to very dense relative density.

Grain size distribution tests were carried out on four samples of the silt portion of the deposit, and the results are shown on Figure B5 in Appendix B. Atterberg limits tests were carried out on three samples of the silt deposit; two of the results show that the silt deposit is non-plastic, while the third test measured a liquid limit of about 17 percent, plastic limit of about 14 per cent and a plasticity index of about 3 percent. These results, which are plotted on a plasticity chart on Figure B6 in Appendix B, indicate that this layer consists of a silt of slight plasticity. The natural water content measured on samples of the silt to sand range from about 15 to 23 per cent.

### 4.2.4 Lower Clayey Silt

A lower deposit or layer of clayey silt was encountered below the silt to sand deposit in Borehole 17-2 at a depth of 7.2 m, corresponding to Elevation 191.3 m. Borehole 17-2 terminated within this deposit/layer, penetrating it for a thickness of 1.0 m. The deposit consist of clayey silt containing some sand.

An SPT “N”-value of 41 blows per 0.3 m was measured within the lower clayey silt deposit, suggesting a hard consistency.

The natural water content measured on one sample of this deposit is about 16 per cent.

### 4.2.5 Groundwater Conditions

The groundwater levels in the open boreholes were measured upon completion of drilling operations. A standpipe piezometer was installed in Borehole 17-2 to permit monitoring of the groundwater level at this site. Details of the piezometer installation and the measured groundwater levels are shown on the borehole records in Appendix A. The groundwater level recorded in the open boreholes and piezometer are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
17-1	203.7	7.9	195.8	Nov. 20, 2017	Open borehole (borehole caved to 7.9m)
17-2	198.5	1.5	197.0	Nov. 21, 2017	Open borehole
		4.0*	194.5	Jan. 26, 2018	Piezometer

\* Water was apparently frozen within the standpipe piezometer at a depth of 4.0 m below ground surface and may not be representative of the groundwater level.

The groundwater level observations at this site will be subject to seasonal fluctuations and precipitation events; the water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation.

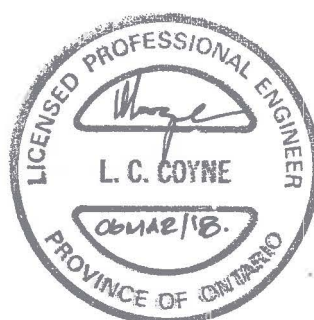
## 5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Darcy Hansen and reviewed by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact and Principal of Golder, conducted a quality control review of the report.

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# PART B

FOUNDATION DESIGN REPORT  
MCINTYRE CREEK CULVERT RETAINING/GABION WALL REPLACEMENT  
HIGHWAY 26, COUNTY OF SIMCOE, ONTARIO  
W.P. 2444-15-00



## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides discussion and foundation engineering recommendations for the proposed replacement of the existing wingwalls and southeast gabion wall at the south (inlet) end of the McIntyre Creek culvert (MTO Structure Site No. 30-522/C), along Highway 26 in the County of Simcoe, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the design engineers with sufficient information to assess the feasible alternatives and carry out the detail design of the wall replacement.

The discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and their designers for this project, and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. The contractor must make their own interpretation of the factual information provided in Part A (Foundation Investigation Report), as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

Based on the Structural Report prepared by AECOM (dated December 1, 2017) and the design drawings provided by AECOM, the existing wingwalls and southeast gabion wall will be replaced with new retaining walls oriented perpendicular to the culvert. Concrete cantilever walls are proposed, as retained soil system (RSS) walls have not been accepted adjacent to the inlet end of this culvert.

Additionally, based on AECOM's detailed design, it is understood that a second relief culvert will be required to accommodate the flows under the design storm conditions. Retaining walls (also proposed as concrete cantilever walls) are planned on either side of the south end of this relief culvert, oriented at a skewed angle to the relief culvert and parallel to Highway 26. Recommendations for the proposed relief culvert are not included in Golder's foundation engineering scope under this assignment, nor addressed in this report.

### 6.2 Retaining Wall and Foundation Options

This section of the report presents a comparison of alternative retaining systems and foundation types based on advantages, disadvantages, risks or opportunities, and relative costs, from a geotechnical/foundations perspective. It should be noted that the selection of the type of walls and foundation alternative will depend on many factors beyond geotechnical / foundation recommendations. The following retaining system/foundation types and geometries have been considered:

- **Reinforced Earth Slope:** A reinforced earth slope constructed at an inclination of 1 horizontal to 1 vertical (1H:1V), or even steeper, is geotechnically feasible at this site; however, it is understood that there is insufficient space for all but vertical retaining solutions, and hence reinforced slopes are not discussed further in this report.
- **Reinforced Soil System (RSS) Wall:** An RSS wall is geotechnically feasible given the competent nature of the shallow soil conditions; these wall types are often advantageous relative to concrete walls on shallow foundations as shallower excavation depths are required, with associated reduction in groundwater control, cofferdam and/or protection system requirements. However, an RSS wall within a floodplain or near flowing water, as is the case for this site, would require a site-specific design submission and review and approval by the MTO RSS Committee. It is understood from AECOM that this option will not be pursued for this site, as such RSS walls are not discussed further in this report.

- **Concrete Retaining Wall on Shallow Foundations:** A concrete retaining wall supported on shallow strip footing foundations is geotechnically feasible for this site. Temporary protection systems would be required parallel to Highway 26 to permit excavation through the existing embankment side slopes to permit removal of the existing walls and reach the footing founding level. The excavation is expected to extend near the groundwater level at the site, particularly during wet periods of the year, and greater groundwater control is expected to be required as compared with RSS walls.
- **Concrete Retaining Wall on Deep Foundations:** A concrete wall supported on deep foundations (driven piles or caissons) is not required at this site due to the competent materials present at shallow depth. Therefore, this option is not discussed further in this report.
- **Soldier Pile and Concrete Panel Wall:** A soldier pile and concrete panel system is considered feasible from a geotechnical/foundation perspective. However, it is anticipated that either a pile driving rig or caisson rig would have to access and work in the floodplain, and such access and work zone preparation may prove to be challenging, as experienced during the foundation field investigation program. If soldier piles are to be installed in pre-augered holes rather than driven, temporary liners would be required, in conjunction with the use of drilling fluids, to control the ground and groundwater for socket formation prior to the placement of soldier piles. For the permanent wall, special measures may be required to minimize the potential for loss of soils from behind the concrete panels, due to the location adjacent to the creek channel and floodplain. Based on these risks, geotechnical/foundation recommendations are not provided for this type of retaining wall, although they can be provided if other disciplines deem this wall type to be the preferred solution.

A comparison of the various retaining wall options based on advantages, disadvantages, relative costs and risks is presented in Table 1. Based on this comparison and given the subsurface conditions as encountered in the boreholes, the preferred retaining wall alternative from a geotechnical perspective is an RSS wall; however, given the challenges and risks associated with using such a wall type adjacent to the creek channel and floodplain, a concrete retaining wall founded on shallow foundations is the preferred option from a geotechnical perspective.

The following sections of this report present foundation recommendations for the strip footings, and the results of the assessment of settlement and global stability of the proposed retaining walls.

## 6.3 Site Classification

### 6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code* and its *Commentary* (CHBDC (2014), also referenced as CAN/CSA-S6-14), the proposed retaining/gabion wall replacement and its foundation system is considered to be classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed in comparison to the degree of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding. Accordingly, the appropriate corresponding ultimate limit states (ULS) and serviceability limit states (SLS) consequence factor,  $\Psi$ , from Table 6.1 and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$ , from Table 6.2 of CHBDC (2014) have been used for design.

### 6.3.2 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the field investigation and geotechnical laboratory testing. The SPT “N”-values measured in the soil layers and the interpreted shear wave velocity of soils were used to define the seismic site classification in accordance with

Table 4.1 of the CHBDC (2014). Based on this methodology it is considered that a Site Class C would be applicable for the design of the retaining wall structures.

### 6.3.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the CHBDC (2014), the peak ground acceleration (PGA) values and design spectral acceleration (Sa) values for Site Class C based on the National Resource Canada (NRC) website are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.025	0.037	0.058
PGV (m/s)	0.025	0.038	0.06
Sa (0.2) (g)	0.045	0.066	0.1
Sa (0.5) (g)	0.034	0.049	0.072
Sa (1.0) (g)	0.02	0.03	0.045
Sa (2.0) (g)	0.0097	0.015	0.023
Sa (5.0) (g)	0.0021	0.0036	0.0058
Sa (10.0) (g)	0.001	0.0015	0.0026

## 6.4 Concrete Cantilever Wall Founded on Shallow Foundations

### 6.4.1 Founding Elevations

Strip footing foundations are feasible for the support of the proposed retaining walls and should be founded below any fill and the very soft portion of the clayey silt deposit, which was encountered down to about Elevation 198 m in Borehole 17-2. All footings should be founded at a minimum depth of 1.5 m below the adjacent final grade to provide adequate protection against frost penetration, in accordance with OPSD 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario*).

Based on the design drawings provided, the lowest final grade in front of the retaining walls will be at approximately Elevation 197 m, as such, the strip footings should be founded at a maximum founding elevation of 195.5 m, on the compact to very dense sand deposit. Where the ground surface rises away from the creek channel, the retaining wall foundations may be stepped up, provided that they remain founded at a minimum depth of 1.5 m, and that they are founded below Elevation 198 m to extend below any soft surficial clayey silt. A continuous strip footing constructed of sections at different founding elevations must include a sloping base on native ground (or granular pad) between sections, inclined no steeper than 1H:1V (i.e., not vertical).

The above-noted founding levels will extend into or near the groundwater level at the site, and groundwater control will be required. MTO's Non-Standard Special Provision (NSSP – "FOUN0003") for dewatering structure excavations should be included in the Contract Documents, as discussed further in Section 6.6.2.

The sand and clayey silt subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. It is recommended that a 100 mm thick, 20 MPa concrete working slab be placed within four hours following inspection and approval of the subgrade, to protect the subgrade from softening. This

requirement should be illustrated on the Contract Drawings, and an NSSP should be included in the Contract Documents. An NSSP is included for this item in Appendix C.

The footing subgrade should be inspected by a qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) and SP109S12 (*Amendment to OPSS 902*) to check that all existing fill and/or other unsuitable material have been removed. Where subexcavation of fill or unsuitable material is required along the east side of the retaining wall (where it is stepped), the sub-excavated area could be backfilled with granular material meeting OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II that is placed and compacted in accordance with OPSS.PROV 501 (*Compacting*), or the thickness of the footing increased to the full excavation depth. If replacement of unsuitable materials with engineered fill is being considered, the area to be subexcavated should be defined by a line extending from the top of the engineering fill pad outward and downward at 1H:1V. The top of the granular engineered fill should extend at least 1 m beyond the plan limits of the footing, and constructed in accordance with OPSS.PROV 501 (*Compacting*). Temporary protection systems will be required, and this is discussed further in Section 6.6.1.

### 6.4.2 Factored Geotechnical Resistance

Strip footings founded on the properly prepared subgrade, at or below the design elevations given in Section 6.4.1, should be designed based on the factored ultimate geotechnical resistances and the factored serviceability geotechnical resistance (for 25 mm of settlement) given below.

Highest Founding Elevation (m)	Founding Stratum	Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
195.5	Compact to very dense sand	3.0	425	350
		4.5	525	225
198.0	Hard clayey silt	3.0	400	375
		4.5	450	250

The geotechnical resistances and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.10.4 of the CHBDC (2014).

### 6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of the compact to dense sand, hard clayey silt or on Granular A or Granular B Type II engineered fill, the coefficient of friction,  $\tan \delta$  or  $\tan \phi'$ , can be taken as follows:

- Cast-in-place footing to concrete working slab:  $\tan \delta = 0.7$



- Cast-in-place concrete working slab to sand deposit:  $\tan \phi' = 0.62$
- Cast-in-place concrete working slab to clayey silt deposit:  $\tan \phi' = 0.62$

#### 6.4.4 Global Stability

Slope stability analyses have been performed for the proposed retaining walls using the commercially available program SLIDE V7 produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS; in general, circular slip surfaces were analyzed. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factored FoS of 1.5 is adopted for the design of retaining walls under static conditions at the end of construction as per the CHBDC (2014). This FoS is considered adequate for the retaining walls at this site considering the design requirements and the field data available.

The following parameters have been used in the analysis for the long-term (drained, effective stress) condition, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Effective Friction Angle
Compact to dense granular fill	19	32°
Generally stiff clayey silt with sand fill	19	32°
Hard clayey silt	20	32°
Compact to dense silt to sand	20	33°
Very dense silt to sand	21	34°

A maximum retained wall height of 4.7 m was assumed for the retaining walls. The groundwater level was inferred from the highest water levels encountered during drilling, as shown on the borehole records.

The stability analysis result indicates that the proposed concrete retaining wall founded on shallow foundations will have a factor of safety greater than 1.5 against global instability. An example of the static global stability results is provided on Figure 1. The proposed 2H:1V slope above the retaining wall may experience some surficial erosion and shallow sloughing, and in accordance with MTO's standard practice for earth slopes, vegetation cover should be established on all fill slope faces to protect against surficial erosion, as per OPSS.PROV 804(*Seeding and Cover*).

#### 6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- For unrestrained walls, where sufficient space is available behind the wall, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the walls (Figure C6.20(b) of the *Commentary* to the CHBDC 2014).

### 6.5.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures are based on a flat ground surface above the wall; the lateral earth pressure coefficients must be adjusted to account for the sloping ground above the wall.

- For unrestrained walls, granular fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing in accordance with Figure C6.20(b) of the *Commentary* to the CHBDC (2014). The following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II	Existing Native Materials
Soil Unit Weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>	20 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:			
Active, $K_a$	0.27	0.27	0.33
At rest, $K_o$	0.43	0.43	0.50

- If the retaining wall structures do not allow lateral yielding, at-rest earth pressures should be assumed for the foundation design. If the retaining wall structure allows for lateral yielding, active earth pressures should be used in the foundation design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the *Commentary* to the CHBDC (2014).

### 6.5.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be taken into account in the design of retaining walls in accordance with Section 4.6.5 of the CHBDC (2014). In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC (2014) and its Commentary, for structures which allow lateral yielding, the horizontal seismic coefficient,  $k_h$ , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding,  $k_h$  is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient  $k_v$  is taken as zero.
- The following seismic active pressure coefficients ( $K_{AE}$ ) may be used in design; these coefficients reflect the maximum  $K_{AE}$  obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

	Design Earthquake	Site PGA	Seismic Active Pressure Coefficients, $K_{AE}$		
			Granular A	Granular B Type II	SSM
Yielding Wall	475-Yr	0.025	0.25	0.25	0.31
	975-Yr	0.037	0.26	0.26	0.31
	2,475 Yr	0.058	0.26	0.26	0.32
Non-Yielding Wall	475-Yr	0.025	0.26	0.26	0.32
	975-Yr	0.037	0.27	0.27	0.32
	2,475 Yr	0.058	0.28	0.28	0.34

- The  $K_{AE}$  value for a yielding wall is applicable provided that the wall can move up to  $250k_h$  mm, where  $k_h$  is the site specific PGA as given in the table above. This corresponds to displacements of 6, 9, and 15 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ yielding walls}$$

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ non-yielding walls}$$

where:  $\sigma_h(d)$  is the (static plus seismic) lateral earth pressure at depth,  $d$ , (kPa);  
 $K_a$  is the static active earth pressure coefficient;  
 $K_o$  is the static at-rest earth pressure coefficient;  
 $K_{AE}$  is the seismic active earth pressure coefficient;  
 $\gamma$  is the unit weight of the backfill soil (kN/m<sup>3</sup>), as given in Section 6.5.1;

- d is the depth below the top of the wall (m); and,  
H is the total height of the wall (m).

## 6.6 Construction Considerations

The following subsections identify construction-related issues that may impact the design, and for which Non-Standard Special Provisions should be included in the Contract Documents.

### 6.6.1 Excavation and Temporary Protection Systems

The foundation excavations for strip footings will extend through the fill material and native clayey silt, into the silt and sand deposit, near or below the groundwater level. Open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) and Regulation 213 for Construction Activities. The existing fill materials are classified as Type 3 soil and the native soils are classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e., those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V through the Type 2 and 3 soils and to within 1.2 m of the bottom of the excavation in Type 2 soils only, provided that appropriate groundwater control is in place.

Temporary protection systems will be required to facilitate removal of the existing wingwalls and gabion retaining wall and the construction of the new retaining wall footings, in order to safely maintain traffic on Highway 26 and protect the construction zone. The temporary excavation support systems 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.

It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the inferred subsurface soil conditions and groundwater conditions. An interlocking sheetpile system would contribute to both ground and groundwater control; however, there is some risk associated with driving the sheetpiles to sufficient depth within the relatively hard/dense native soils that are present at relatively shallow depth below the natural ground surface, and this must be considered by the contractor's temporary works designer. For a soldier pile and lagging system, it is anticipated that soldier piles could be either driven or installed in pre-augered holes. Where pre-augered holes are used, they would need to be advanced with temporary liners and drilling fluids to avoid disturbance of the ground, and to control seepage or include measures to mitigate loss of soil particles through the lagging boards near the base of the excavations and adjacent to the creek channel. Lateral support to the sheetpiles or soldier piles could be provided in the form of rakers, temporary anchors or cross-bracing. The selection and design of the protection system will be the responsibility of the Contractor.

### 6.6.2 Surface Water and Groundwater Control

Control of the surface water and groundwater will be necessary to allow excavation and foundation construction to be carried out in dry conditions. MTO's "FOUN0003" NSSP shall be included in the Contract Documents to address the dewatering requirements for retaining wall construction adjacent to the culvert.

Precipitation runoff in the construction area should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the sand and clayey silt subgrade or granular backfill/bedding material.

Based on the water level measurements in the piezometer, the groundwater level appeared to be frozen or blocked at Elevation 194.5 m, but based on observations during drilling and the proximity to the creek channel, it is anticipated that the water level could be higher, on the order of Elevation 195 m to 196 m, particularly during wet periods of the year. Due to the proximity of the proposed retaining walls to the edge of the McIntyre Creek,



it is recommended that a groundwater cut-off (cofferdam or similar measure) be incorporated to minimize dewatering requirements and potential environmental impacts for excavation of the strip footings. A cut-off/cofferdam could consist of interlocking steel sheetpiles driven below the proposed base of excavation, although as noted above, there may be some risk associated with driving sheetpiles to sufficient depth given the relatively hard/dense nature of the soils at the site. Alternatively, an inflatable bladder system may be adopted to separate the foundation excavation from the creek, or alternatively the work may be able to be conducted within a shored, flooded excavation.

### 6.6.3 Subgrade Protection

The native soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic, groundwater infiltration and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP. An NSSP is included in Appendix C.

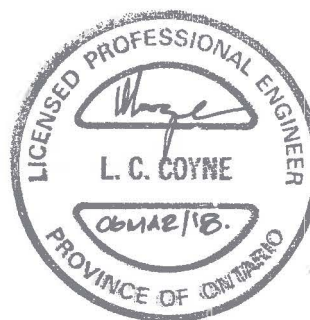
## 7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Darcy Hansen and reviewed by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact and Principal of Golder, conducted an independent technical and quality control review of the report.

**Golder Associates Ltd.**



Nikol Kochmanová, Ph.D., P.Eng., PMP  
*Geotechnical Engineer*



Lisa Coyne, P.Eng.  
*Designated MTO Foundations Contact, Principal*

DH/NK/LCC/rb

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<https://golderassociates.sharepoint.com/sites/15994g/6. deliverables/wo 004 - mcintyre culvert/3. final/1671430 wo4 fidr 2018mar06 mcintyre creek culvert gabion wall replacement.docx>

## REFERENCES

Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.

*Canadian Highway Bridge Design Code (CHBDC) 2014 and Commentary on CAN/CSA-S6-14*. Canadian Standard Association (CSA) Group.

Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

Kulhawy, F.H. and Mayne, P.W. 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

National Resources Canada, 2017. *Earthquake Hazard*. [http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index\\_2015-en.php](http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php). Accessed on Feb. 11, 2018.

### Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

### Special Provisions

SP109S12	Amendment to OPSS902
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### Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
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### ASTM International

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

### Ontario Water Resources Act

Ontario Regulation 903Wells (as amended)

### Ontario Occupational Health and Safety Act

Ontario Regulation 213Construction Projects (as amended)

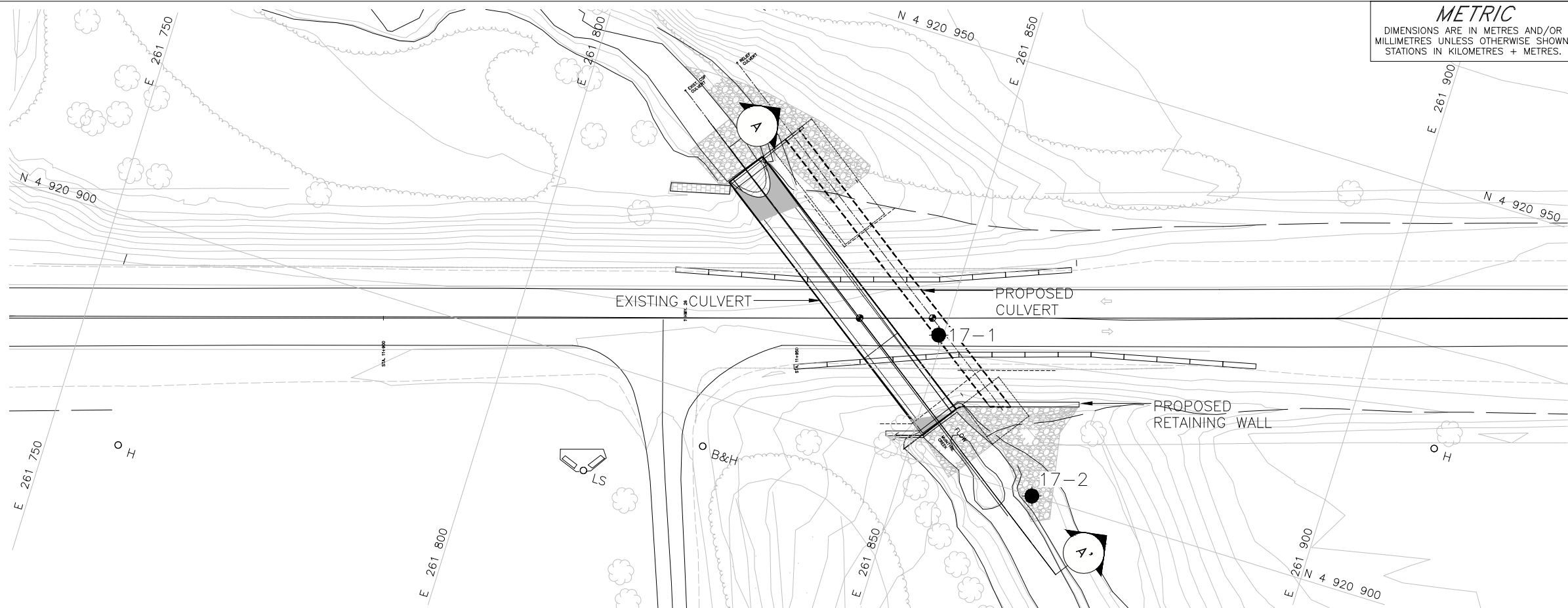
**TABLE 1 – COMPARISON OF RETAINING WALL AND FOUNDATION ALTERNATIVES  
RETAINING WALL NOS. 24-887/W AND 24-888/W**

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Reinforced earth embankment	Not feasible due to space restrictions at this site	<ul style="list-style-type: none"> <li>■ Relative ease of construction but proprietary product required</li> <li>■ Vegetated surfaces could be used to improve aesthetics</li> </ul>	<ul style="list-style-type: none"> <li>■ Special treatment of reinforced earth slopes required to allow vegetation to grow and minimize erosion; there can be challenges in establishing vegetation, although south-facing exposure for the McIntyre Creek site would lessen this risk</li> <li>■ Likely similar concerns over risk of constructing such systems in proximity to floodplain and flowing water</li> </ul>	<ul style="list-style-type: none"> <li>■ Lower cost than all vertical wall options</li> </ul>	<ul style="list-style-type: none"> <li>■ Requires wider footprint</li> <li>■ Very tolerant of settlement (although this is not an issue at this site)</li> <li>■ Risk of susceptibility to erosion in flood conditions</li> </ul>
RSS wall	Not feasible as the RSS wall would be within the floodplain and near flowing water.	<ul style="list-style-type: none"> <li>■ More tolerable to post-construction settlements, although this is not a significant issue at this site</li> <li>■ Lowest cost vertical wall alternative where feasible</li> </ul>	<ul style="list-style-type: none"> <li>■ Potentially larger excavation required to install reinforcing strips, but likely similar to footing width for a concrete wall; temporary protection systems required</li> <li>■ Potential for loss of soil particles in flood conditions, although special measures can be adopted to mitigate</li> </ul>	<ul style="list-style-type: none"> <li>■ Lower cost than concrete retaining wall or walls supported on deep foundations</li> </ul>	<ul style="list-style-type: none"> <li>■ Risk around loss of soil particles in flood conditions; special site-specific design and approval through MTO RSS Committee required</li> </ul>

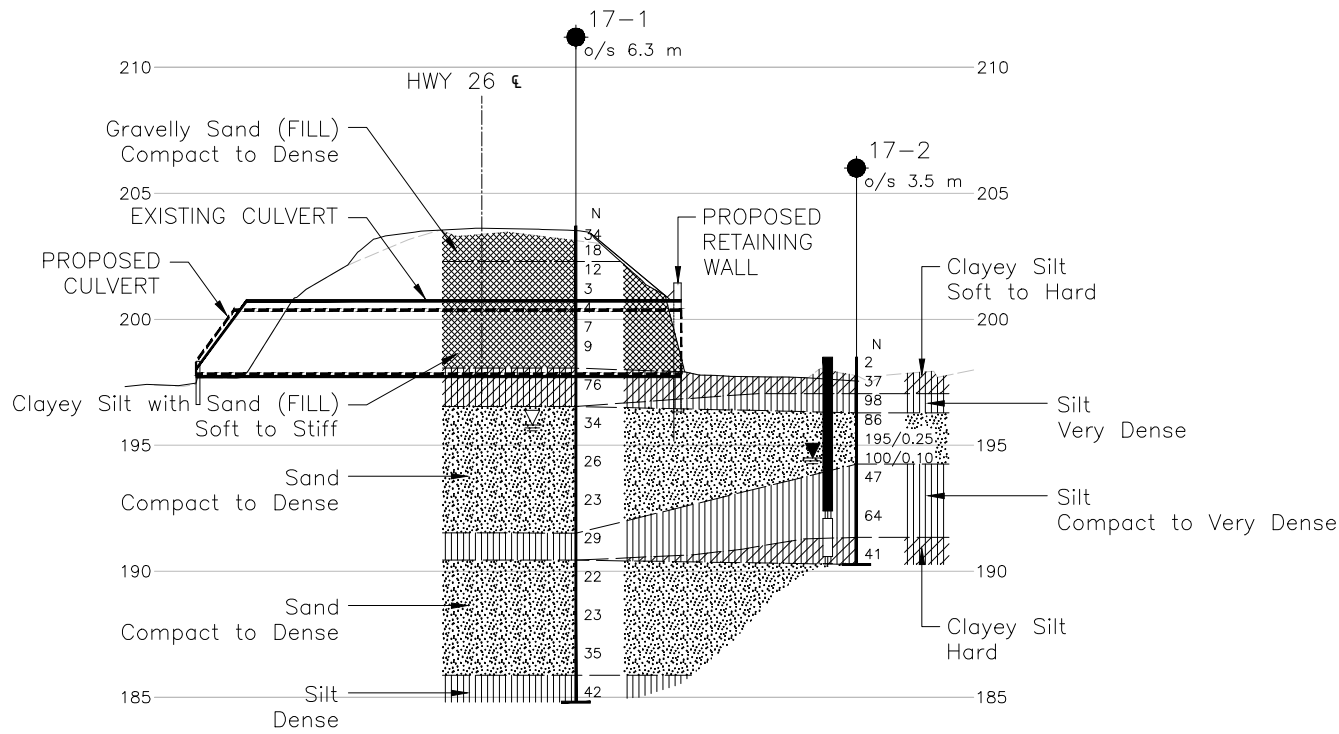
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Concrete retaining wall on shallow foundations	Feasible provided sufficient space is available during construction and/or for temporary shoring if used.	<ul style="list-style-type: none"> <li>■ Conventional excavation and construction techniques</li> <li>■ Suitable founding stratum below depth of frost penetration at this site</li> </ul>	<ul style="list-style-type: none"> <li>■ Less tolerable to post construction settlements</li> <li>■ Temporary protection systems will be required</li> <li>■ Footings must be founded below depth of frost penetration</li> </ul>	<ul style="list-style-type: none"> <li>■ Higher cost relative to RSS wall</li> </ul>	<ul style="list-style-type: none"> <li>■ Deeper excavation as compared with RSS walls, adjacent to existing culvert and creek channel</li> <li>■ Deeper protection systems and greater potential for groundwater control</li> </ul>
Concrete retaining wall on deep foundations	Not required at this site due to competent soil present at shallow depth	<ul style="list-style-type: none"> <li>■ Potentially reduced excavation, protection system and backfill requirements compared to RSS wall</li> </ul>	<ul style="list-style-type: none"> <li>■ Temporary/permanent liners would be required to allow for construction of caissons.</li> <li>■ If refusal (100-blow) stratum or obstructions are encountered, can get piles to hang-up, requiring pre-drilling</li> </ul>	<ul style="list-style-type: none"> <li>■ Higher cost relative to RSS wall</li> </ul>	<ul style="list-style-type: none"> <li>■ Least demanding on right-of-way space if tie-backs not required</li> </ul>

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Soldier pile and concrete panel wall	Feasible from a geotechnical perspective; however, greater challenges with constructability in terms of heavy caisson rig accessing and working in floodplain, and risks associated with ground loss	<ul style="list-style-type: none"> <li>■ Most advantageous in “top-down” construction applications, minimizes excavation and requirement for temporary excavation support</li> </ul>	<ul style="list-style-type: none"> <li>■ Likely requires heavy caisson rig to access and work in the floodplain</li> <li>■ Potential risk of loss of soil particles at gaps between panels/soldier piles</li> <li>■ Requires use of liners and fluid control to minimize disturbance and ground loss during formation of soldier pile holes</li> <li>■ Potential need for tie-back/anchor installation, with associated testing requirements</li> </ul>	<ul style="list-style-type: none"> <li>■ Anticipated to be comparable costs to concrete retaining wall, but higher than RSS wall</li> <li>■ Cost of temporary protection system combined with RSS wall is comparable</li> </ul>	<ul style="list-style-type: none"> <li>■ Lesser excavation than other options, and may eliminate or reduce requirements for temporary protection system</li> <li>■ Risks associated with heavier rig equipment accessing and working in floodplain</li> <li>■ Risks associated with loss of soil particles at gaps in panels during flood conditions (similar to, albeit not as high a risk as, RSS walls)</li> </ul>





PLAN



PROFILE A - A'



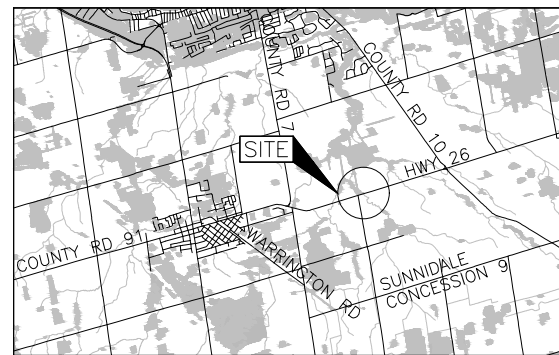
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No. 2018-2006  
WP No. 2444-15-00

MCINTYRE CREEK CULVERT  
REHABILITATION  
BOREHOLE LOCATION AND SOIL  
STRATA



SHEET  
19



KEY PLAN

SCALE  
0 2 4 km

LEGEND

- Borehole - Current Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ▽ WL upon completion of drilling
- ▽ WL in piezometer on January 26, 2018

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
17-1	203.7	4920915.2	261849.4
17-2	198.5	4920900.0	261865.9

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 plans provided in digital format by AECOM, drawing file nos. SITE 30-522C FOR INROADS.DWG, SITE 30-522C.DWG, received November 28, 2017, 4-60555407\_McINTYRE CREEK\_SITE\_30-522\_CSP\_LINER\_GA.dwg, received February 16, 2018 and 4-60555407\_McINTYRE CREEK\_SITE\_30-522\_CSP\_LINER\_GA.dwg, received March 5, 2018..

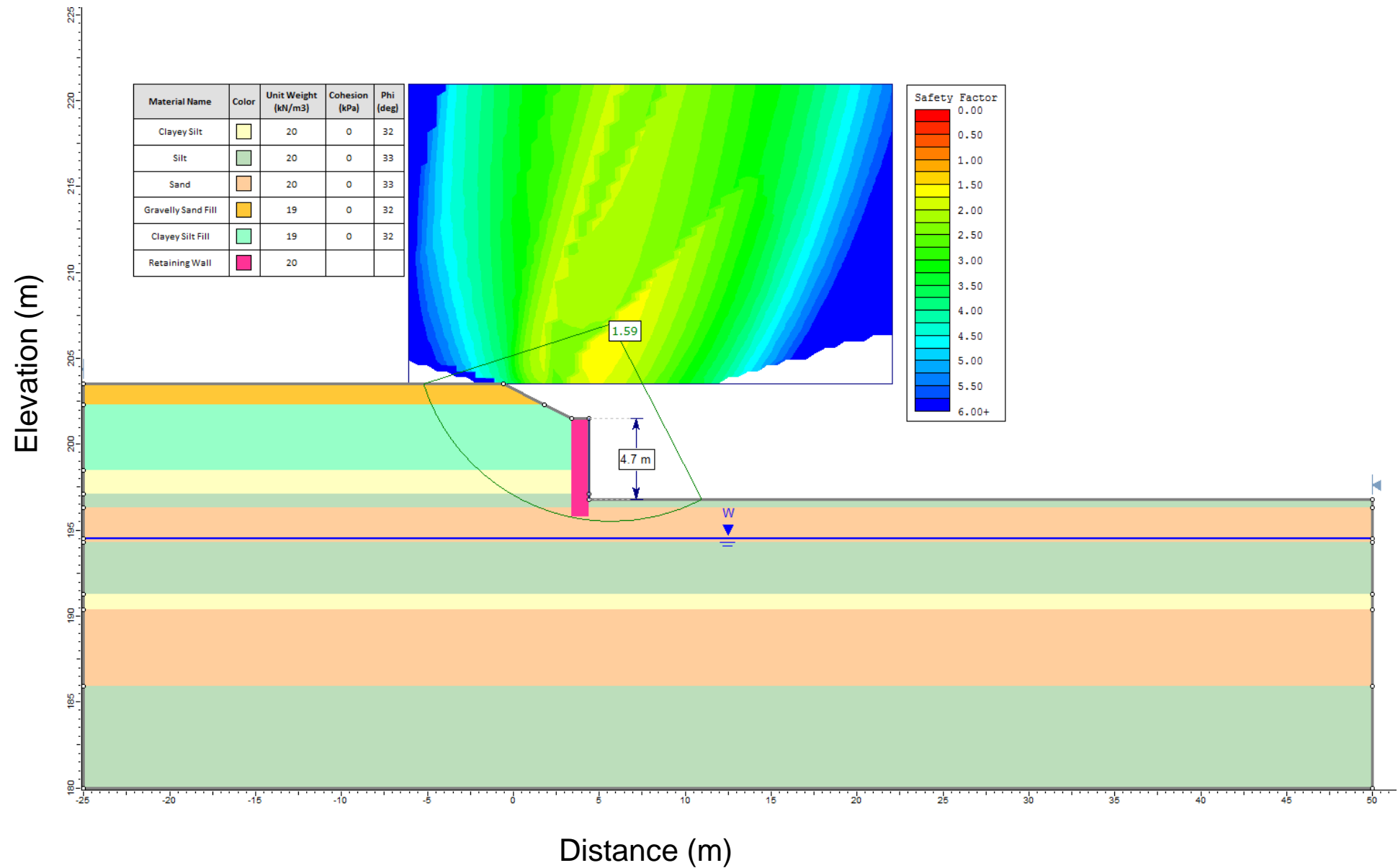


NO.	DATE	BY	REVISION
Geocres No. 41A-246			
HWY. 26	PROJECT NO. 1671430		DIST. 30-552/O
SUBM'D. DH	CHKD. NK	DATE: 3/6/2018	SITE: .
DRAWN: DD	CHKD. NK	APPD. LCC	DWG. 1



STATIC GLOBAL STABILITY  
CONCRETE CANTILEVER RETAINING WALL ON SHALLOW FOUNDATIONS

Figure 1



APPENDIX A

# Borehole Records

## LIST OF SYMBOLS

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

### I. GENERAL

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

### II. STRESS AND STRAIN

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

### III. SOIL PROPERTIES

#### (a) Index Properties

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

#### (a) Index Properties (continued)

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

#### (b) Hydraulic Properties

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

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

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

#### (d) Shear Strength

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

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

Notes: 1  
2

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

## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### II. PENETRATION RESISTANCE

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

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

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

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

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

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

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

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

### III. SOIL DESCRIPTION

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

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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
$SO_4$	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

### V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	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





## METRIC



CHECKED BY NK

Continued Next Page

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

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PROJECT <u>1671430</u>	<b>RECORD OF BOREHOLE No 17-1</b>	SHEET 2 OF 2	<b>METRIC</b>
G.W.P. <u>2016-E-0029</u>	LOCATION <u>N 4920915.2; E 261849.4 MTM NAD 83 ZONE 10 (LAT. 44.426970; LONG. -80.039439)</u>	ORIGINATED BY <u>DMF</u>	
DIST <u>Central</u> HWY <u>26</u>	BOREHOLE TYPE <u>203 mm O.D Hollow Stem Augers-Truck-mounted Drill Rig</u>	COMPILED BY <u>DH</u>	
DATUM <u>Geodetic</u>	DATE <u>November 20, 2017</u>	CHECKED BY <u>NK</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			LIQUID LIMIT	UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					GR	SA	SI	CL	
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>							
	--- CONTINUED FROM PREVIOUS PAGE ---																					
	SAND, trace to some silt Compact to dense Grey Moist to wet		14	SS	23																	
			15	SS	35																	
185.9																						
17.8	SILT, trace to some clay, trace sand Dense Grey Moist																					
			16	SS	42																	
184.8																						
18.9	END OF BOREHOLE																					
	NOTES:  1. Borehole caved to 7.9 m below ground surface upon completion of drilling.  2. Water level in open borehole observed at a depth of 7.9 m (Elev. 195.8 m) below ground surface upon completion of drilling.																					

PROJECT 1671430		RECORD OF BOREHOLE No 17-2		SHEET 1 OF 1		METRIC															
G.W.P. 2016-E-0029		LOCATION N 4920900.0; E 261865.9 MTM NAD 83 ZONE 10 (LAT. 44.426835; LONG. -80.039231)		ORIGINATED BY DMF/DH																	
DIST Central HWY 26		BOREHOLE TYPE 76 mm O.D Casing Portable Tripod Drill Rig		COMPILED BY DH																	
DATUM Geodetic		DATE November 20 to 21, 2017		CHECKED BY NK																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
198.5	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30											
0.0	CLAYEY SILT, some sand, trace gravel, trace to some rootlets, oxidation staining Very soft to hard Brown to grey Moist to wet		1	SS	2		198														1 19 67 13
197.1			2	SS	37																
1.5	SILT, trace to some sand, trace to some clay, oxidation staining Very dense Grey/brown Moist		3	SS	98		197														0 7 92 2
196.3			4	SS	86		196														
2.2	SAND, trace to some silt Very dense Grey/brown Moist		5	SS	195/0.24		195														
			6	SS	100/0.14																
194.3			7	SS	47		194														0 18 80 2
4.3	SILT, some sand, trace clay Dense to very dense Grey/brown Moist		8	SS	64		192														
191.3			9	SS	41		191														
7.2	CLAYEY SILT, some sand Hard Grey Moist																				
190.3	END OF BOREHOLE																				
8.2	NOTES:  1. Water level measured at a depth of 1.5 m (Elev. 197.0 m) below ground surface upon completion of drilling.  2. Water in monitoring well frozen at a depth of 4.0 m (Elev. 194.5 m) below ground surface on January 26, 2017.																				

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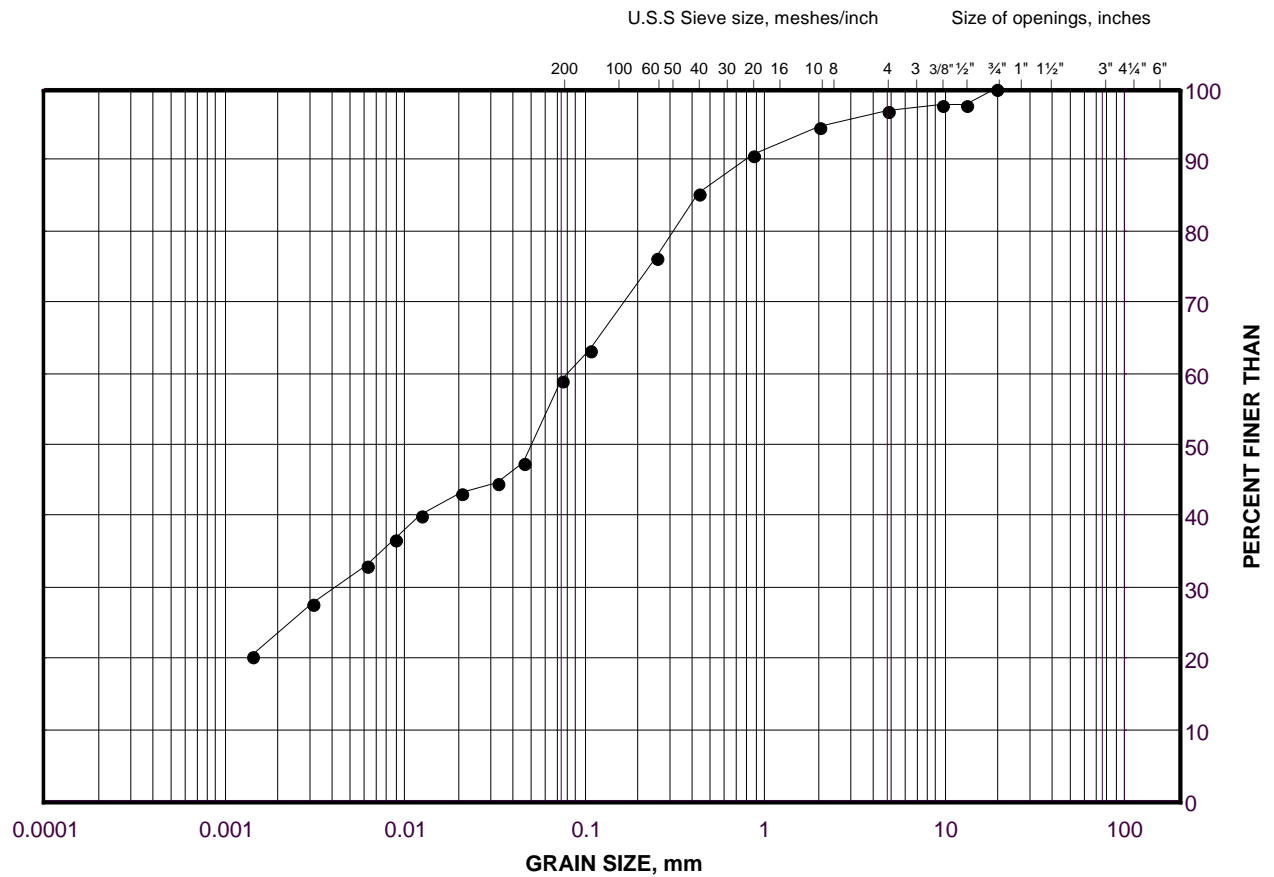
**APPENDIX B**

# Laboratory Test Results

# GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (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)
•	17-1	4	201.1

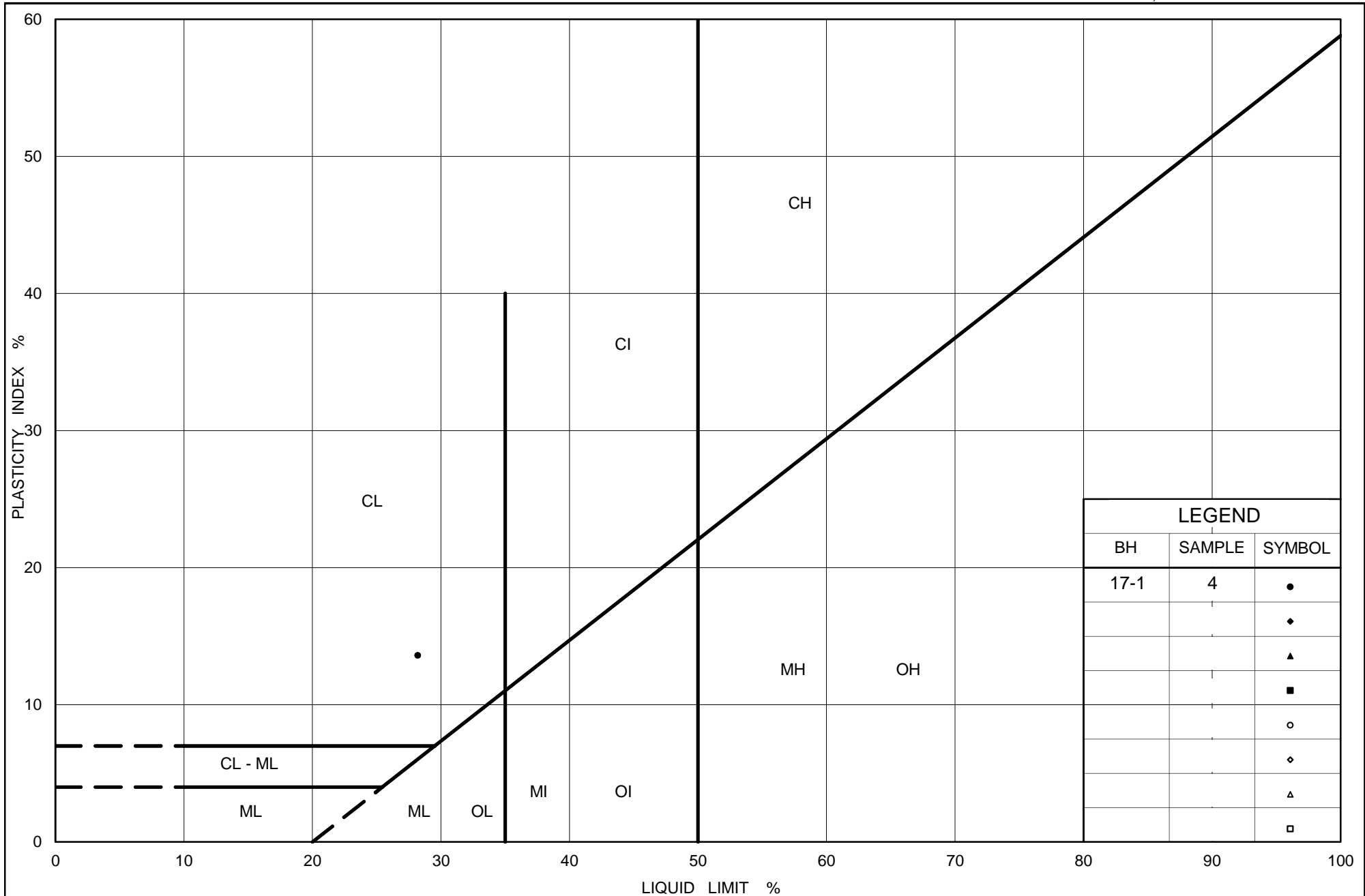
Project Number: 1671430/WO004

Checked By: NK

**Golder Associates**

Date: 05-Feb-18





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## PLASTICITY CHART

### Clayey Silt (FILL)

Figure No. B2

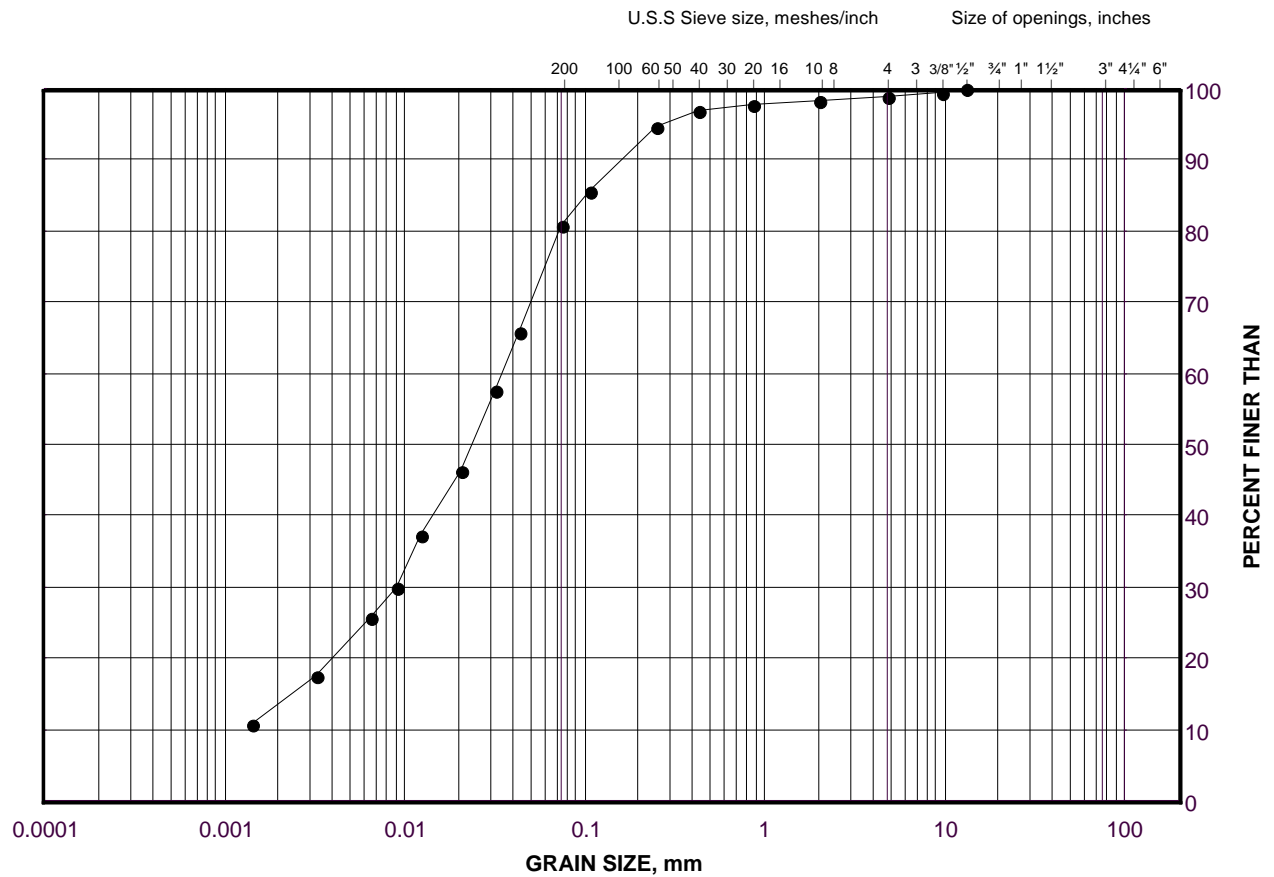
Project No. 1671430/WO004

Checked By: NK

# GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B3



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

## LEGEND

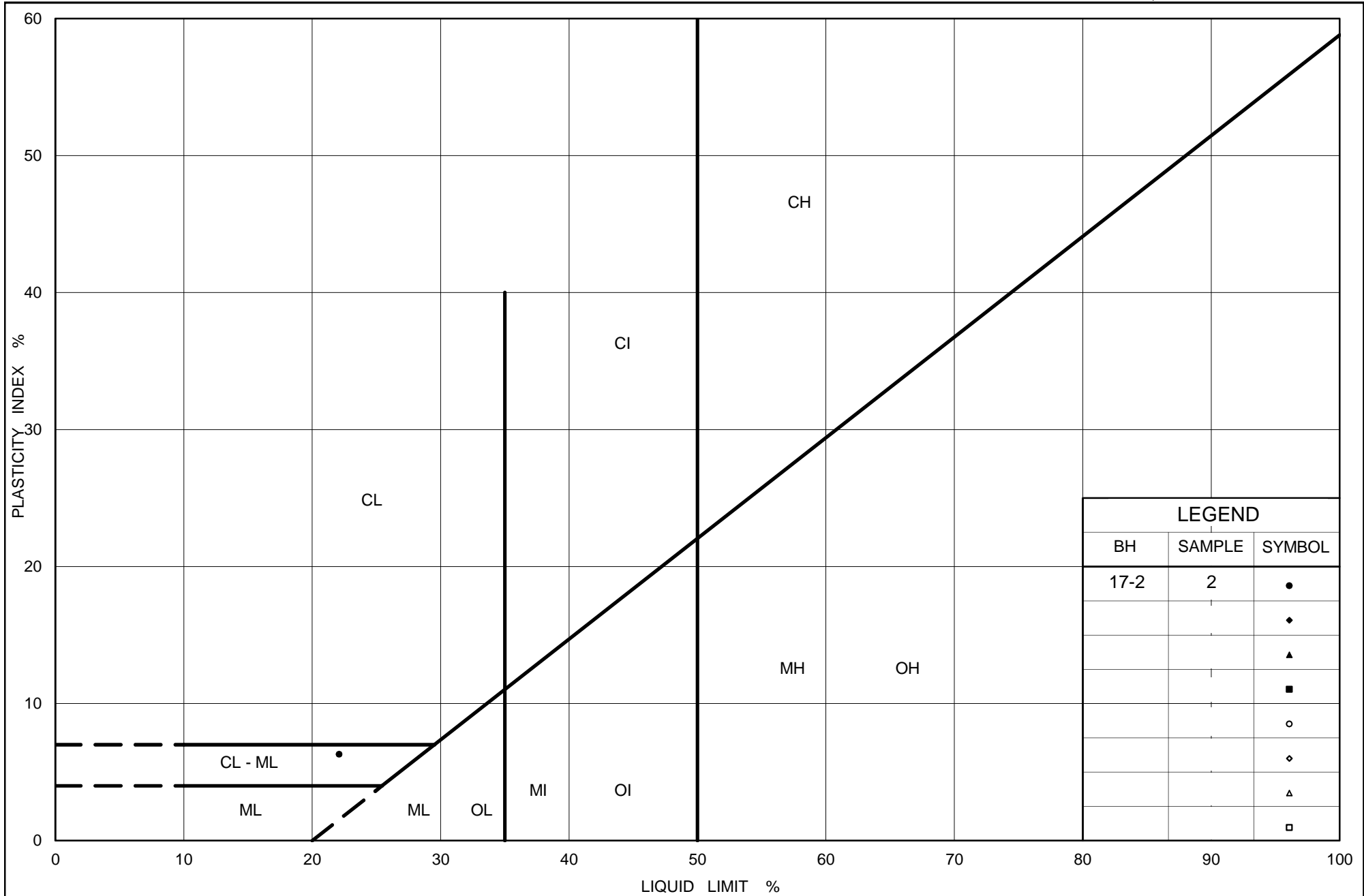
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	17-2	1	197.4

Project Number: 1671430/WO004

Checked By: NK

**Golder Associates**

Date: 05-Feb-18



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# PLASTICITY CHART Clayey Silt

Figure No. B4

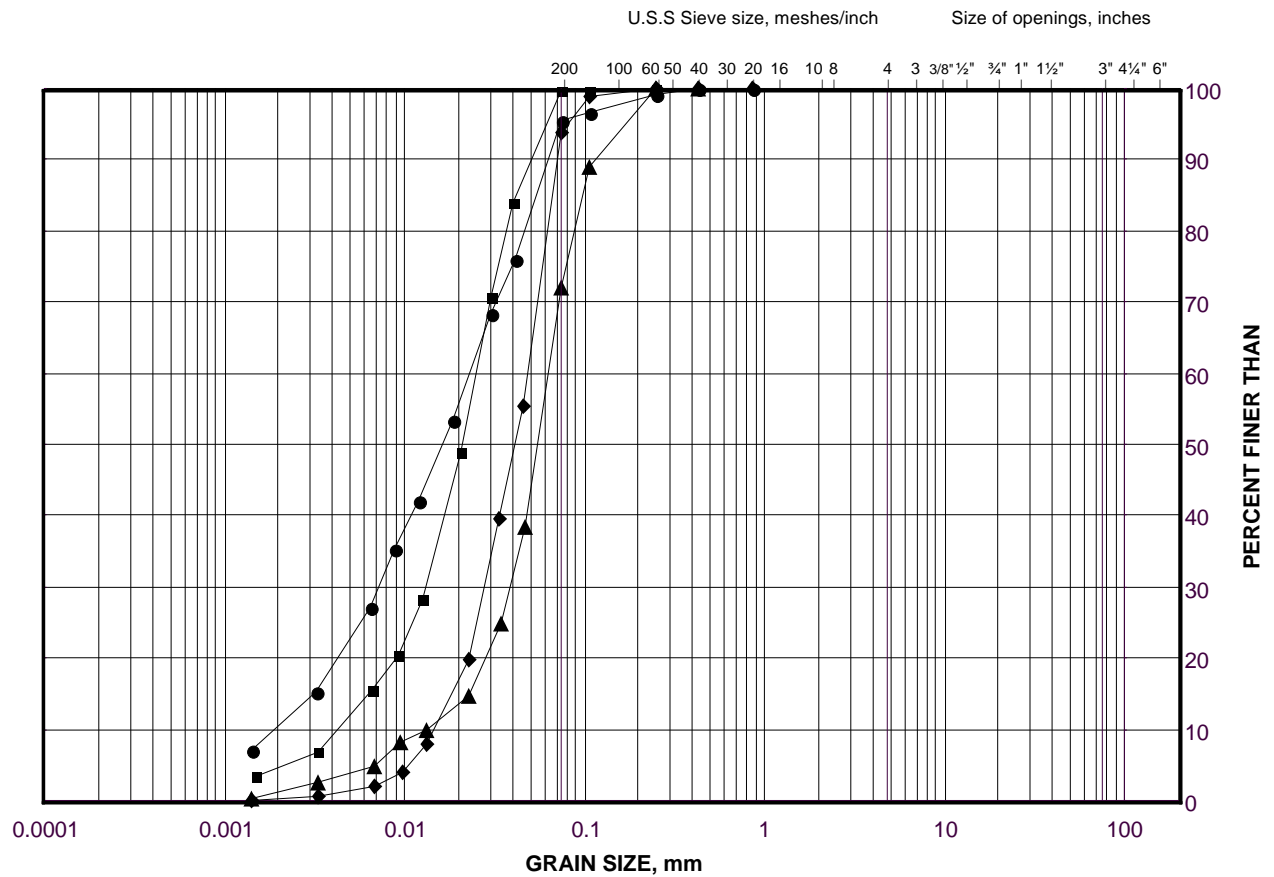
Project No. 1671430/WO004

Checked By: NK

# GRAIN SIZE DISTRIBUTION

Silt

FIGURE B5



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

## LEGEND

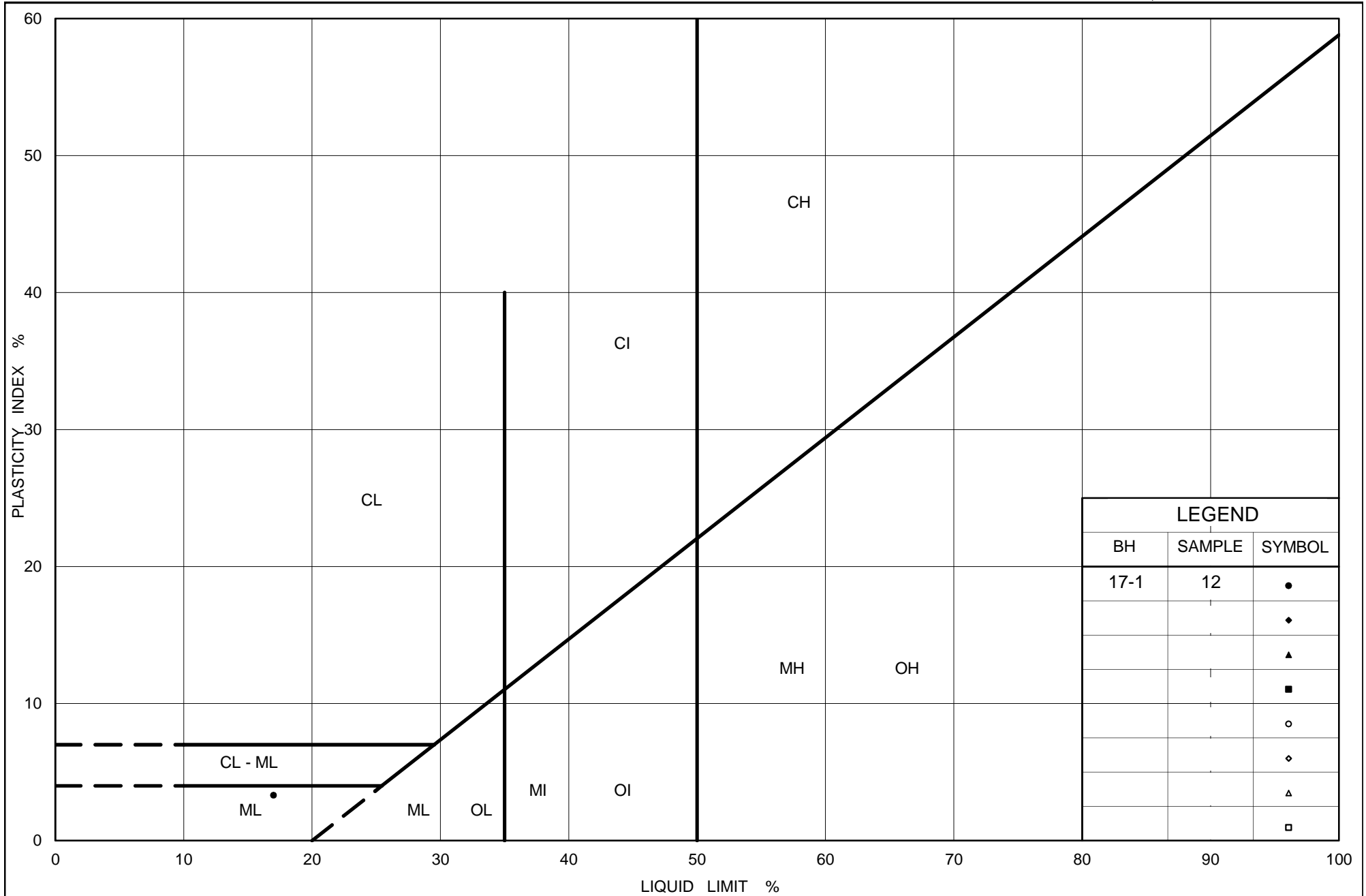
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-1	12	191.2
■	17-1	16	185.1
◆	17-2	3	196.7
□	17-2	7	193.6

Project Number: 1671430/WO004

Checked By: NK

**Golder Associates**

Date: 05-Feb-18



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# PLASTICITY CHART

## Silt

Figure No. B6

Project No. 1671430/WO004

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**APPENDIX C**

# Non-Standard Special Provisions

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**WORKING SLAB - Item No.**

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Non-Standard Special Provision

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**1.0 Scope**

This Special Provision covers the requirements for the supply and placement of a concrete working slab under foundations for the QEW retaining wall replacement structures.

**1.1 References**

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction  
OPSS 902 Excavating and Backfilling - Structures

**2.0 Definitions - Not Used****3.0 Design and Submission Requirements - Not Used****4.0 Materials**

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

**5.0 EQUIPMENT - Not Used****7.0 CONSTRUCTION****7.01 Excavation**

Excavation for the working slab shall be according to OPSS 902.

**7.02 Protection of Founding Soil**

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

**7.04 Dewatering**

Dewatering shall be carried out according to OPSS 902.

**6.0 Quality Assurance - Not Used****9.0 Measurement for Payment - Not Used****10.0 Basis of Payment****10.01 Working Slab - Item**

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

**END OF SECTION**



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**DEWATERING STRUCTURE EXCAVATIONS - Item No.**

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Non-Standard Special Provision

---

**Amendment to OPSS 902, November 2010****902.02 REFERENCES**

Section 902.02 of OPSS 902 is amended by the addition of the following:

**Ontario Provincial Standard Specifications, Construction**

OPSS 517	Dewatering
OPSS 805	Temporary Erosion and Sediment Control Measures

**902.03 DEFINITIONS**

Section 903.03 of OPSS 902 is amended by the addition of the following:

**Automatic Transfer Switch** means as defined in OPSS 517.

**Cofferdam** means as defined in OPSS 539.

**Cut-Off Wall** means as defined in OPSS 517.

**Design Storm Return Period** means as defined in OPSS 517.

**Dewatering System** means as defined in OPSS 517.

**Groundwater Control System** means as defined in OPSS 517.

**Plug** means as defined in OPSS 517.

**Sediment** means as defined in OPSS 517.

**Sediment Control Measure** means as defined in OPSS 517.

**Temporary Flow Passage System** means as defined in OPSS 517.

**Unwatering** means as defined in OPSS 517.

**Vegetated Discharge Area** means as defined in OPSS 517.

**Waterbody** means as defined in OPSS 517.

**Watercourse** means as defined in OPSS 517.

**902.04 DESIGN AND SUBMISSION REQUIREMENTS****902.04.01 Design Requirements****902.04.01.01 Dewatering**

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a two-year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

#### **902.04.02 Submission Requirements**

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

##### **902.04.02.01 Working Drawings**

Working Drawings for the dewatering system shall be according to OPSS 517.

##### **902.04.02.02 Preconstruction Survey**

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, utilities, and structures, within a distance of 300 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

##### **902.04.02.03 Milestone Inspections**

The Quality Verification Engineer shall witness the following Interim Inspections of the work:

- a) Dewatering of excavation for structure.
- b) Completion of excavation for foundation.
- c) Excavation for backfill and frost tapers.
- d) Backfilling.

A copy of the written permission to proceed shall be submitted to the Contract Administrator prior to commencement of the successive operation.

#### **902.07 CONSTRUCTION**

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

##### **902.07.04 Dewatering Structure Excavation**

###### **902.07.04.01 General**

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

#### **902.07.04.02 Discharge of Water**

The discharge of water shall be according to OPSS 517.

#### **902.07.04.03 Monitoring**

Monitoring shall be according to OPSS 517.

#### **902.07.04.04 System Amendments**

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

#### **902.07.04.05 Removal**

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.



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