



FOUNDATION INVESTIGATION AND DESIGN REPORT

Alona Bay Creek Culvert Rehabilitation, Site No. 38S-0353/C0
Highway 17, Slater Township, Algoma District
Ministry of Transportation, Ontario
GWP 5376-11-00; WP 5170-16-01

Submitted to:

AECOM Canada Ltd.

189 Wyld Street, Suite 103
North Bay, ON
P1B 1Z2

Submitted by:

Golder Associates Ltd.

33 Mackenzie Street, Suite 100, Sudbury, Ontario
P3C 4Y1, Canada
+1 705 524 6861

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PART A

FOUNDATION INVESTIGATION REPORT
ALONA BAY CREEK CULVERT REHABILITATION, SITE NO. 38S-0353/C0
HIGHWAY 17, SLATER TOWNSHIP, ALGOMA DISTRICT
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5376-11-00; WP 5170-16-01

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 Foundation Engineering services for the rehabilitation of the existing Alona Bay Creek Culvert (Site No. 38S-0353/C0), located on Highway 17 in Slater Township, Algoma District, Ontario (GWP 5376-11-00, WP 5170-16-01).

The purpose of this investigation is to obtain subsurface soil, bedrock, and groundwater information at the site, by means of a limited number of boreholes and geotechnical and analytical laboratory testing. The investigation also included a review of available geotechnical information from a previous foundation investigation completed by others at the structure site, as presented in the following report:

- **MTO GEOCREs No. 41N-001:** “Highway 17 at Mile 17 Creek Crossing, Proposed Culvert at approximately Station 3770+20, Approx. 71 Miles North of Sault Ste. Marie, Dist. 18, WP 911-60, 60-F-33”, dated June 22, 1960.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO’s Request for Proposal (Assignment 5017-E-0018, dated October 2017), and the subsequent clarifications/addenda. Golder’s proposal dated January 26, 2018, for Foundation Engineering services associated with the rehabilitation of this structure is contained in Section 17.8 of AECOM’s technical proposal for this assignment. The work has been carried out in accordance with Golder’s Supplementary Specialty Plan for Foundation Engineering services for this project, dated April 11, 2018.

2.0 SITE DESCRIPTION

The Alona Bay Creek Culvert is located on Highway 17 at Station 18+094, in Slater Township, Algoma District, Ontario, approximately 41.2 km north of Highway 563 and 115 km south of the Highway 101, as shown on the Key Plan on Drawing 1. It should be noted that the orientation (i.e., north, south, east, west) stated in the text of the report, is typically referenced to project north and therefore may differ from magnetic north shown on the drawing. For the purpose of this report, Highway 17 is oriented in a north-south direction along this section of highway, with the culvert positioned on a slight skew from perpendicular to the highway in a generally east-west orientation.

The existing culvert consists of a twin cell reinforced cast-in-place concrete box, which was constructed in 1960. Alona Bay Creek is located at the east end of the culvert (inlet) and Lake Superior is located at the west end of the culvert (outlet). Based on the General Arrangement (GA) drawing provided by AECOM, each cell has a span of approximately 4.6 m, an internal height of approximately 3.7 m, and a length of approximately 51.1 m. The 1960 GA indicates that the existing culvert (inlet and outlet) and wingwalls are founded at Elevations 189.6 m and 189.5 m, respectively. However, the current project survey provided by AECOM determined that the existing culvert (inlet and outlet) and wingwalls are founded at Elevations 189.0 m and 188.9 m, respectively. Similarly, the 1960 GA indicates that the existing culvert invert (at the inlet and outlet) is at Elevation 190.1 m; however, the current project survey provided by AECOM determined the existing culvert invert (at the inlet and outlet) is at Elevation 189.5 m. Based on discussions with AECOM, we understand that the site has been resurveyed and the current project survey information is considered to be correct.

The highway grade at the site is understood to be about Elevation 200.9 m. The existing highway embankment is approximately 11.9 m and 9.9 m high, relative to the ground surface at the toes of the embankment side slopes at

the east and west creek banks, respectively. The side slope to the top of the west wingwalls is inclined at about 1.6 Horizontal to 1 Vertical (1.6H:1V) to 2H:1V and the side slope to the top of the east wingwalls is inclined at about 2H:1V to 3H:1V.

Based on the Ontario Structure Inspection Manual (OSIM) report, dated August 21, 2015, the existing culvert, wingwalls, and embankments are generally in good condition; however, the southwest wingwall has separated from the culvert and rotated forward by approximately 6 degrees. Based on our site observations at the time of the field investigation and a review of the available site photographs/satellite images, the existing embankments in the culvert area generally appear to be performing satisfactorily with little to no evidence of soil movement, tilted vegetation, or tension cracks near the embankment crest that would be indicative of instability; with the exception of the southwest embankment side slope where surficial embankment sloughing was noted along with signs of erosion in the upper portion of the embankment (i.e., due to surface water run-off).

The highway platform at the culvert location and ground surface conditions at the culvert inlet and outlet area (east and west ends, respectively) are shown on Photographs 1 to 6. In general, the topography in the area of the culvert structure consists of undulating/hilly terrain, which is heavily forested beyond the highway Right-of-Way (ROW).

3.0 INVESTIGATION PROCEDURES

The field exploration was carried out on May 8 and 9, 2019, and August 13 and 15, 2019, during which time four boreholes (designated as Boreholes AB-1 to AB-4) were advanced to depths of 9.8 m below ground surface. The boreholes were advanced on land, in the vicinity of the culvert inlet and outlet, as shown on Drawing 1.

The boreholes were advanced using NW casing with mud rotary drilling techniques and a CME 55 LC track-mounted drill rig supplied and operated by George Downing Estate Drilling of Grenville-Sur-La-Rouge, Quebec. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic hammer, in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586)¹.

Due to the use of mud rotary drilling techniques (addition of drilling mud in the boreholes), the groundwater conditions in the open boreholes could not be observed during drilling. However, the depth to the water level (unstabilized) was measured inside the casing upon completion of drilling each borehole. The boreholes were backfilled upon completion, in accordance with Ontario Regulation 903 Wells (as amended).

Field work was monitored on a full-time basis by a member of Golder's technical staff who: located the boreholes in the field; arranged for the clearance of underground services; supervised the drilling and sampling operations; logged the boreholes; and examined the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's geotechnical laboratory in Sudbury for further examination and laboratory testing. Index and classification testing consisting of water content determinations and grain size distributions were carried out on selected soil samples. The geotechnical laboratory testing was completed according to ASTM and MTO LS standards, as applicable. In addition, a select soil sample was submitted to a

¹ Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soil, ASTM D1586

specialist analytical laboratory (Maxxam Analytics Inc.) under chain of custody procedures for testing for a suite of parameters including pH, resistivity, conductivity, sulphate, sulphide, and chloride.

The as-drilled borehole locations were measured by a member of our technical staff relative to the existing culvert structure and wingwalls using a measuring tape and converted into northing/easting coordinates on the plan drawing. Given the relatively short distances between the boreholes and the existing structures, the measurements are considered to be accurate to within 0.5 m horizontally. The ground surface elevation at the borehole locations were obtained using a survey level and rod and the borehole survey loop was closed to within 0.1 m vertically. The boreholes were surveyed relative to the centreline of the roadway, the elevation of which was obtained from the survey drawing (17SLA GWP 5376-11-00.dwg) provided by AECOM. The NAD 83 MTM Zone 13 northing and easting coordinates and World Geodetic System 1984 (WGS 84) geographical coordinates, ground surface elevations referenced to Geodetic datum, and borehole depths at each borehole location are presented on the Record of Borehole Sheets included in Appendix A and summarized below.

Borehole Number	Location (NAD 83, MTM Zone 13)		Location (WGS 84)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting	Latitude	Longitude		
AB-1	5225177.8	252402.9	47.163912	84.691119	192.3	9.8
AB-2	5225169.0	252394.9	47.163833	-84.691224	190.5	9.8
AB-3	5225192.1	252327.1	47.164035	-84.692121	189.3	9.8
AB-4	5225181.1	252326.7	47.163936	-84.692125	189.6	9.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain Study (NOEGTS)² mapping by the Ministry of Natural Resources, the Alona Bay Creek Culvert site is located within a raised (abandoned) beach form comprised of sand and gravel, bordered by jagged, rugged and cliffed bedrock knobs.

Based on geological mapping by the Ministry of Northern Development and Mines (MNDM)³, the overburden deposits are underlain by sandstone, shale, conglomerate, bordered by foliated tonalite suite bedrock.

² Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 41NSE

³ Ontario Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East-Central Sheet, Ontario Geological Survey – Map 2543

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. Geotechnical laboratory test results are provided in Appendix B and analytical laboratory test results are provide in Appendix C.

The results of the in-situ field tests (i.e., SPT 'N' values), as presented on the Record of Borehole sheets and in Section 4, are uncorrected. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic cross-sections on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

The soil conditions encountered during the current investigation are generally consistent with those encountered in the previous 1960 investigation. In general, the native subsurface conditions encountered in the 2019 boreholes consist of a surficial loose to compact gravel deposit underlain by a compact to dense silty sand to sand deposit. A detailed description of the soil deposits and groundwater conditions encountered in the boreholes is provided in the following sections.

4.2.1 GRAVEL (GW-GM)

A 0.7 m to 1.5 m thick deposit of gravel and sand was encountered in all boreholes at ground surface and extends to depths ranging from 0.7 m to 1.5 m below ground surface (Elevations 191.6 m to 188.1 m). The gravel deposit is brown, moist to wet, and contains trace silt, trace organics, and cobbles (in Borehole AB-2). An approximately 100 mm size cobble was also encountered in Borehole AB-4.

The SPT 'N'-values measured within the gravel deposit range from 4 blows to 27 blows per 0.3 m of penetration, indicating a loose to compact compactness condition.

The in-situ moisture content measured on one sample of the deposit is about 11 per cent.

The result of a grain size distribution test completed on one sample of the gravel deposit is shown on Figure B-1 in Appendix B.

4.2.2 SILTY SAND (SM) to SAND (SP)

A 8.3 m to 9.1 m thick deposit of silty sand to sand was encountered underlying the gravel deposit in all boreholes. The deposit was encountered at depths ranging from 0.7 m to 1.5 m below ground surface (Elevations 191.6 m to 188.1 m) and extends to the borehole termination depths of 9.8 m below ground surface (Elevations 182.6 m to 179.5 m). A 2.1 m thick interlayer of silt was encountered within the silty sand to sand deposit in Borehole AB-2, at a depth of 3.5 m below ground surface (Elevation 187.0 m), as further described in Section 4.2.3. The silty sand to sand deposit ranges in gradation from silty sand to sand, some silt, to sand and contains trace gravel and trace clay. Cobbles and boulders should be anticipated in this deposit, as inferred from the grinding of the casing during drilling within this deposit in Boreholes AS-1 and AS-3. Heaving/flowing conditions were encountered during drilling through this deposit, as noted on the borehole records.

The SPT 'N'-values measured within the silty sand to sand deposit range from 16 blows to 54 blows per 0.3 m of penetration, indicating a compact to very dense compactness condition. It is noted that the measured

SPT 'N'-values below depths of about 6 m to 9 below ground surface may have been taken in a disturbed stratum, due to heaving/flowing conditions under artesian groundwater pressure, as indicated on the borehole records.

The in-situ moisture content measured on fourteen samples of the silty sand to sand deposit ranges from about 20 to 25 per cent.

The results of grain size distribution tests completed on fourteen samples of the silty sand to sand deposit are shown on Figure B-2 in Appendix B.

4.2.3 SILT (ML) Interlayer

A 2.1 m thick interlayer of silt and sand was encountered within the silty sand to sand deposit in Borehole AB-2 at a depth of 3.5 m (Elevation 187.0 m).

The SPT 'N'-values measured within the silt interlayer are 19 blow and 20 blows per 0.3 m of penetration, indicating a compact compactness condition.

The in-situ moisture content measured on one sample of the silt interlayer is about 20 per cent.

The result of a grain size distribution test completed on one sample of the silt interlayer is shown on Figure B-3 in Appendix B.

4.2.4 Groundwater/Surface Water Conditions

The unstabilized water level measured in the boreholes (within the casing) upon completion of mud rotary drilling ranges from 0.6 m above ground surface to 1.2 m below ground surface, ranging from Elevation 191.1 m to 189.1 m. It is noted that the unstabilized water level is likely influenced by the addition of water and mud for mud rotary drilling techniques and may not be representative of stabilized groundwater levels at the site. During drilling of Boreholes AB-2 to AB-4, water was observed flowing upward to above ground surface, between the borehole wall and the auger casing, suggesting an upward groundwater gradient (i.e., artesian conditions).

The water level within the culvert was at Elevation 190.8 m in June 2018, as surveyed by others. However, based on our understanding of site conditions, it is understood that this water level corresponds to about the creek water level at the culvert inlet. The creek water level at the culvert inlet, east end of the culvert, was at Elevation 190.6 m on May 9, 2019, as measured by Golder. Based on visual observations, and as shown on Photographs 3 and 4, the creek water level at the culvert outlet is estimated to be about 0.8 m lower than the culvert invert (i.e., 0.5 m lower than Elevation 189.5 m) and is estimated to be about Elevation 189.0 m in May 2019.

Further, it should be noted that the groundwater level at site and the water level of Alona Bay Creek and Lake Superior are subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year. Based on site observations in January 2018, as shown on Photographs 5 and 6, it is anticipated that due to the fast-flowing conditions of the creek at this site, the water at the east and west ends of the culvert (i.e., within the Alona Bay Creek and Lake Superior) does not freeze over during the winter season.

4.2.5 Analytical Test Results of Soil Samples

One soil sample was submitted for analytical testing to assess the potential for the soil to cause deterioration to buried concrete and corrosion to steel. The results of the analytical testing are provided in Appendix C and are summarized below.

Parameter	Units	Borehole AB-1, Sample 3
Resistivity	ohm-cm	24,000
Conductivity	µmho/cm	42
pH	pH	5.34
Sulphate	µg/g	<20 ¹
Chloride	µg/g	<20 ¹
Sulphide	µg/g	<0.30 ¹

Note: 1. Concentration is lower than respective Reportable Detection Limit (RDL).

5.0 CLOSURE

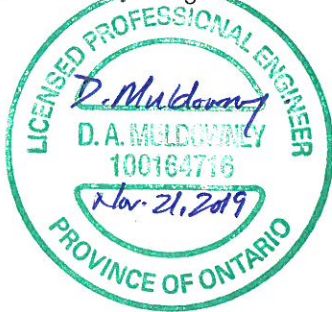
The field drilling program was carried out under the supervision of Mr. Shane Albert, under the overall direction of Mr. David Muldowney, P.Eng. This Foundation Investigation Report was prepared by Ms. Anastasia Poliacik, P.Eng., and Mr. David Muldowney, P.Eng., provided a technical review of the report. Mr. Jorge Costa, P.Eng., an MTO Foundations Designated Contact and Senior Consultant for Golder, conducted an independent quality control review of this report.

Signature Page

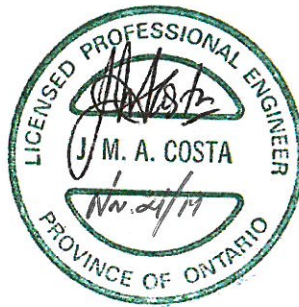
Golder Associates Ltd.



Anastasia Potiacik, P.Eng.
Geotechnical Engineer



David Muldowney, P.Eng.
Senior Geotechnical Engineer



Jorge M. A. Costa, P.Eng.
MTO Foundations Designated Contact, Senior Consultant

KJ/AMP/DAM/JMAC/ljv/sm/ca

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PART B

FOUNDATION DESIGN REPORT
ALONA BAY CREEK CULVERT REHABILITATION, SITE NO. 38S-0353/C0
HIGHWAY 17, SLATER TOWNSHIP, ALGOMA DISTRICT
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5376-11-00; WP 5170-16-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering recommendations for the proposed rehabilitation of the Alona Bay Creek Culvert (Site No. 38S-0353/C0) located on Highway 17 in Slater Township, Algoma District, Ontario (GWP 5376-11-00, WP 5170-16-01). The recommendations presented are based on an interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives, and to carry out conceptual level designs for the temporary dewatering systems. The foundation investigation report and the discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO), 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 contractors must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction, must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.1 General

The culvert rehabilitation works will consist of concrete repairs to soffit, walls, and base of the culvert and replacement of the northwest and southwest wingwalls at the west (outlet) end. The existing wingwalls are supported on footings founded at shallow depth, below the creek bed/banks and are approximately 2.7 m wide, adjacent to the culvert decreasing linearly to about 1.8 m wide at the ends, by 5.1 m long, and vary in height from 4.3 m (nearest the culvert) to 2.4 m (furthest from the culvert). A concrete shear key is present below the toe of the footing of each wingwall and extends about 0.8 m below the wingwall footing. The 1960 GA indicates that the existing culvert (inlet and outlet) and wingwalls are founded at Elevations 189.6 m and 189.5 m, respectively. However, the current project survey determined that the existing culvert (inlet and outlet) and wingwalls are founded at Elevation 189.0 m and 188.9 m, respectively. Similarly, the 1960 GA indicates that the existing culvert invert (at the inlet and outlet) is at Elevation 190.1 m; however, the current project survey determined the existing culvert invert (at the inlet and outlet) is at Elevation 189.5 m. Based on discussions with AECOM, we understand that the site has been resurveyed and the current project survey information is considered to be correct.

Based on the Ontario Structure Inspection Manual (OSIM) report, dated August 21, 2015, the existing culvert, wingwalls, and embankments are generally in good condition; however, as noted in Section 2.0, the southwest wingwall has separated from the culvert and rotated forward by approximately 6 degrees. Based on observations during the field subsurface investigation program, the highway embankment is considered to be performing adequately without showing evidence of slope instability or pavement distress.

Based on discussion with AECOM, it is our understanding that the separation and rotation of the southwest wingwall is a result of rotational failure and that the foundations of the wingwalls have performed well. We understand that the northwest, northeast, and southeast wingwalls have been performing the intended function adequately; however, it is understood that the northwest wingwall will also be replaced at the same time the southwest wingwall is replaced.

As indicated by AECOM, the replacement wingwalls are to have a similar configuration to the existing wingwalls and will connect to the existing culvert. The replacement wingwalls are to be 3.25 m wide, 5 m long, and vary in

height from 4.3 m (nearest the culvert) to 2.5 m (furthest from the culvert). The replacement wingwalls are to retain about 8 m to 9 m of embankment fill. A concrete shear key at the toe of the wall footing will extend 1.5 m below the footing founding level. The existing highway embankment geometry will be maintained, such that the west embankment side slopes are inclined at about 1.6H:1V to 2H:1V.

It is anticipated that during rehabilitation works (concrete repairs and wingwall replacement), water flow will be maintained within one of the culvert cells while the repairs/replacements are carried out in or adjacent the other cell.

6.2 General Foundation Design Context

6.2.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *2014 Canadian Highway Bridge Design Code* CAN/CSA S6-14 (2014 CHBDC) and its *Commentary*, the culvert and its foundation system (including wingwalls) may be classified as having large traffic volumes and their performance as having potential impacts on other transportation corridors, resulting in a “typical consequence level” associated with exceeding limit states design.

Based on the level of foundation investigation, in comparison to the degree of site understanding in Section 6.5 of 2014 CHBDC, the level of confidence for design for the culvert wingwalls has been assessed as “typical degree of site and prediction model understanding”.

The corresponding consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC (2014), have been used for design, as indicated in the sections below.

6.3 Wingwall Foundations

6.3.1 Founding Elevation

Strip footings should be founded below any topsoil or fill, on the native loose to compact gravel and sand and/or compact to very dense silty sand to sand and silt deposit.

The frost penetration depth at the site is estimated to be 2.2 m, as per Ontario Provincial Standard Drawing (OPSD) 3090.100 (Foundation Frost Penetration Depths for Northern Ontario). Therefore, to minimize the potential for damage/maintenance, due to frost action, strip footings should be provided with at least 2.2 m of conventional soil cover or an equivalent combination of insulation and soil cover. However, we understand that it would be more cost effective to establish the footings at a higher elevation to minimize excavation depth and the need to provide temporary shoring/dewatering requirements and to prevent undermining of the existing culvert. As such, alternative footing founding elevations above the depth of frost penetration and at the depth of frost penetration are provided below.

Founding Depth Relative to Creek Bed ¹	Founding Elevation (m)	Founding Soil Strata
0.1 Below	188.9 (same elevation as existing footing)	Loose to compact gravel
0.5 Below	188.5	Loose to compact gravel

Founding Depth Relative to Creek Bed ¹	Founding Elevation (m)	Founding Soil Strata
1.0 Below	188.0	Compact to very dense silty sand to sand
2.2 Below	186.8	Compact to very dense silty sand to sand

Note 1: The creek bed is estimated to be at Elevation 189.0 m.

It is noted that although the founding strata, comprised of the gravel deposit or silty sand to sand deposit, are granular in nature and are classified as having a low to moderate susceptibility to frost heave, respectively, based on selected grain size distribution tests (MTO Pavement Design and Rehabilitation Manual, 2013), footings placed above the frost penetration depth (i.e., within the zone of frost penetration), may still be susceptible to frost heave and seasonal movement. The water level at the culvert outlet is estimated to be about Elevation 189.0 m (i.e., 0.9 m above the proposed footing founding elevation), and based on field observations in January 2018, the creek water at this location does not freeze during winter conditions; therefore, the risk of seasonal movements, due to frost penetration/heave, could reasonably be considered to be low. Further, given that the existing northwest, northeast, and southeast wingwall foundations appear to have performed adequately and are also founded above the depth of frost penetration at about Elevation 188.9 m, the risks associated with frost movement for footings founded above the depth of frost penetration are expected to be relatively low for the southwest and northwest replacement wingwalls.

Prior to placement of the footings, any topsoil, fill or deleterious material must be removed, and the subgrade should be inspected by a qualified geotechnical engineer. Foundation excavation and backfill should be in accordance with OPSS 902 (*Excavating and Backfilling Structures*), as amended by SP 109S12 and NSSP FOUN0003. A copy of NSSP FOUN0003 is included in Appendix E. Where any sub-excavation is required, the sub-excavated areas should be backfilled with granular material meeting OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II, placed and compacted, in accordance with OPSS.PROV 501 (*Compacting*), as amended by SP 105S22. In wet conditions, it is recommended that Granular B Type II be used.

6.3.2 Geotechnical Resistance

The proposed 3.25 m wide by 5 m long footings, founded at the elevations noted in Section 6.3.1, may be designed based on the factored ultimate geotechnical resistance and factored serviceability geotechnical resistance (for 25 mm of settlement) provided below.

Founding Depth Relative to Creek Bed ¹ (m)	Founding Elevation (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa)
0.1 Below	188.9	120	100
0.5 Below	188.5	190	100
1.0 Below	188.0	270	105
2.2 Below	186.8	375	110

Note 1: The creek bed is estimated to be at Elevation 189.0 m.

Based on discussions with AECOM, we understand that a factored ultimate geotechnical resistance of 145 kPa and a factored serviceability geotechnical resistance of 136 kPa are required for design of the replacement wingwall footings. In order to achieve the required factored ultimate geotechnical resistance, we recommended that the footings be founded at or below Elevation 188.5 m. Based on the soil conditions and proposed founding levels (i.e., at or below Elevation 188.5 m), calculated settlements of 28 mm to 32 mm are estimated for the anticipated loading conditions. Actual settlements may be less than 25 mm, as the serviceability geotechnical resistance has been factored in accordance with the CHBDC (2014) and considering that the loading conditions for the replacement footings will be similar or less than those for the existing footings given that a larger foundation footprint is being proposed and the embankment geometry is being maintained (i.e., no grade raise and or widening).

The factored geotechnical resistances provided above are based on the loading applied perpendicular to the base of the footings. Inclination of the load should be taken into account, in accordance with Section 6.10.4 and Section C6.10.4 of CHBDC (2014) and its Commentary. The factored geotechnical resistances should be reviewed if the founding elevation and/or the foundation widths differ from those given above.

6.3.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of the concrete footings and the subgrade, should be calculated in accordance with Section 6.10.5 of the 2014 CHBDC. For cast-in-place concrete, the factored coefficient of friction, $\tan \phi'$, of the footing to the native gravel deposit and silty sand to sand deposit can be taken as 0.5 and 0.35, respectively, as interpreted from NAVFAC (1982).

6.4 Global Stability

As noted in Section 2.0 and Section 6.1, the existing embankments in the culvert area generally appear to be performing satisfactorily with little to no evidence of soil movement, tilted vegetation, or tension cracks near the embankment crest that would be indicative of instability. However, surficial embankment sloughing was noted along the southwest embankment side slope along with signs of erosion (due to surface water run-off) in the upper portion of the embankment, which can likely be attributed to the outward movement of the southwest wingwall at the toe of the embankment slope, rather than being indicative of global embankment instability.

A slope stability analysis of the proposed wingwalls was carried out using the commercially available program SLIDE 2018, produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For the analysis, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factored FoS of 1.54 is adopted for the design of retaining walls under static conditions at the end of construction as per the 2014 CHBDC for a “typical consequence level”. This FoS is considered adequate for the wingwalls at this site, considering the design requirements and the field data available.

The stability analysis was carried out based on the steepest slope profile behind the proposed walls, as shown by cross-section C-C' on Drawing 1. The analysis assumes the embankment side slopes to the top of the wingwalls will remain as existing (i.e., the embankment slope is at about 1.6H:1V and the wingwalls will support about 8 m of fill behind the wall). The analyses assume a water/groundwater level ranging from Elevation 190.8 m at the culvert

inlet to Elevation 189.0 m at the culvert outlet. The parameters used in the analyses are based on correlations with SPT 'N' values and experience with laboratory testing in similar soil conditions.

Stability analyses were carried for the proposed wingwall geometry founded at Elevation 188.9 m, with a 1.5 m deep shear key, and for a wingwall geometry founded at Elevation 188.5 m, with a 2.0 m deep shear key. The results of the analyses are provided on Figures D-1 and D-2 in Appendix D and are summarized below.

Foundation Depth Relative to Creek Bed (m)	Foundation Elevation (m)	Shear Key Depth (m)	Factor of Safety	Figure No.
0.1 Below	188.9	1.5	1.47	D-1
0.5 Below	188.5	2.0	1.54	D-2

Based on the analyses, the proposed wingwalls founded at Elevation 188.9 m will have a FoS less than 1.54, which does not meet the requirements as outlined in the CHBDC (2014) for a “typical consequence level”, whereas the proposed wingwalls founded at Elevation 188.5 m will have a FoS of 1.54 and therefore meet the requirements of the CHBDC (2014) for a “typical consequence level”. Further, it must be noted that the footing founding level at Elevation 188.9 m (i.e., 0.1 m below the creek bed), the factored geotechnical axial resistance at ULS (and SLS) are less than the required design geotechnical axial resistance(s) as discussed in Section 6.3.2.

It is noted that if the wingwalls are considered to have a “low consequence level”, a target minimum FoS of 1.34 would apply and the proposed wingwall geometry founded at Elevation 188.9 m would meet the requirements of the CHBDC (2014). Consideration may also be given to leaving the temporary protection systems in place in order to provide additional support for global stability.

The resulting FoS is highly dependent on the the wall/footing geometry, the final grade/ground surface geometry in front of the wall, and the embankment side slope inclination above the top of the wall, and therefore, the global stability should be re-assessed if the wall/footing geometry, embankment geometry, and/or final grade surrounding the wingwalls are revised from those noted above.

6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the wingwalls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the wingwalls:

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Materials Specification for *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*), as amended by SP105S22.
- 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 the 2014 CHBDC Section 6.12.3 and Figure 6.6. Hand-operated compaction equipment should be used to compact the backfill soils, immediately behind the

walls as per OPSS.PROV 501, as amended by SP105S22. Other surcharge loadings should be accounted for in the design, as required.

- For wingwalls (retaining walls that are retained), granular fill should be placed in a zone with the width equal to at least 2.2 m behind the back of the wall on Figure C6.20(a) of the Commentary to the 2014 CHBDC. For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn flatter than 1 horizontal to 1 vertical (1H:<1V) extending up and back from the rear face of the footing or pile cap on Figure C6.20(b) of the Commentary to the 2014 CHBDC.

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 pressure coefficients assume that the ground above the wall will be sloped at 1.6H:1V to 2H:1V for the southwest and northwest retaining walls, respectively, as derived from CFEM (2006).

- For a restrained wall, the pressures are based on the fill behind the granular backfill zone, and the following parameters (unfactored) may be used assuming the existing embankment was constructed using a Granular 'B' Type I material:

Material	Existing Embankment Fill (assumed Granular B Type I)
Soil Unit Weight:	21 kN/m ³
Coefficients of static lateral earth pressure: Active, K_a (Active, K_a for 1.6:1 slope) Passive, K_p (Active, K_p for 1.6:1 slope) At Rest, K_o	0.22 (0.34) 4.55 (2.12) 0.36

- For an unrestrained wall, the pressures are based on the engineered granular fill within the backfill zone, and the following parameters (unfactored) may be used:

Material	Granular 'A'	Granular 'B' (Type I or II)
Soil Unit Weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure: Active, K_a (Active, K_a for 1.6:1 slope) Passive, K_p (Active, K_p for 1.6:1 slope) At Rest, K_o	0.22 (0.34) 4.55 (2.12) 0.36	0.22 (0.34) 4.55 (2.12) 0.36

- If the wingwall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- If the wingwall allows for lateral yielding, active earth pressures may be used in the geotechnical design of the structure. 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 2014 CHBDC.

6.6 Construction Considerations

6.6.1 Temporary Protection/Dewatering Systems

Temporary protection/dewatering systems will be required for surface water/groundwater cut-off, to allow for construction of the wingwall foundations (including shear key) in dry conditions. Further, it is anticipated that temporary protection systems may also be required parallel to the highway, to allow for excavation into the existing embankment side slope for wingwall replacement.

The selection and design of the temporary protection and dewatering systems will be the responsibility of the Contractor. A sheet pile dewatering system is considered feasible for this site; however, there is a risk that the sheet pile installation may be impeded by the presence of cobble and potentially boulder obstructions, which are inferred present from grinding of the drill casing during borehole advancement. If a sheet pile dewatering system is to be used, consideration should be given to the use a heavier sheet pile section to mitigate the risk of potential damage to the sheeting, if heavy driving is required during the installation. Consideration could also be given to the use of a sandbag or inflatable bladder system for surface water control. It is noted that due to the upward groundwater gradient from the sand deposit underlying the silty sand to sand and silt deposit at the site and the overall permeable nature of the site soils, groundwater seepage through the base of the excavation and/or below/beneath the base of temporary protection/dewatering system should be considered in the selection and design of the temporary protection/dewatering system. If a sheet pile dewatering system is selected, the sheet piles should extend to a sufficient depth, in order to cut-off the upward groundwater gradient; alternately, a tremie concrete plug could be implemented.

The dewatering system shall be designed, constructed, maintained, monitored, and removed in accordance with the Non-Standard Special Provision (NSSP) for Cofferdams, included in Appendix C, which should be included in the Contract Documents. The cofferdam system shall be designed to achieve Performance Level 2 requirements as outlined in the NSSP.

Although the contractor is responsible for the complete detail design of the temporary protection and dewatering systems, the following parameters are provided to enable the structural designer(s) to develop a conceptual design and assess the approximate construction costs for the temporary protection and dewatering system. As noted in Section 4.2.4, artesian groundwater conditions were noted at three borehole locations, estimated to be up to about 0.6 m above ground surface, that is between Elevations 189.3 m and 191.1 m, compared to a creek water levels between about Elevations 189.0 m and 190.6 m. Therefore, for design of the temporary protection and dewatering systems, a groundwater level at Elevation 191 m should be used.

Soil Type	Unit Weight (γ , kN/m ³)	Internal Angle of Friction (ϕ , degrees)	Coefficients of Earth Pressure ¹ (Adjusted Coefficients of Earth Pressure ²)		
			Active, K_a	At Rest, K_o	Passive, K_p
New Fill (Granular A)	22	40	0.22 (0.34)	0.36	4.55 (2.12)
New Fill (Granular B Type I or II)	21	40	0.22 (0.34)	0.36	4.55 (2.12)

Soil Type	Unit Weight (γ , kN/m ³)	Internal Angle of Friction (ϕ , degrees)	Coefficients of Earth Pressure ¹ (Adjusted Coefficients of Earth Pressure ²)		
			Active, K_a	At Rest, K_o	Passive, K_p
Assumed Existing Granular Embankment Fill	21	40	0.22 (0.34)	0.36	4.55 (2.12)
Loose to Compact Gravel	21	40	0.22 (0.34)	0.36	4.55 (2.12)
Compact to Very Dense Silty Sand / Silt and Sand	19	37	0.25 (0.42)	0.40	4.00 (1.71)
Compact to Dense Sand	20	35	0.27 (0.50)	0.43	3.65 (1.44)

Notes: 1. Coefficients of Earth Pressure based on flat ground behind the excavation.

2. Adjusted Coefficients of Earth Pressure based on a sloping ground behind the excavation at 1.6H:1V.

The temporary protection system should be assessed for both the drained (ϕ) and undrained cases (S_u), based on the more conservative earth pressure conditions. Additionally, the lateral movement of the temporary protection system shall meet Performance Level 2, as specified in OPSS.PROV 539 (*Temporary Protection Systems*), as amended by SP 105S09, provided that any adjacent utilities can tolerate this magnitude of deformation.

It is understood that consideration is being given to only partially removing the temporary protection and dewatering systems upon completion of construction or each stage of construction in order to enhance the global stability of the wingwalls/embankment slope. If partial removal is ultimately selected rather than full removal, the partial removal shall be carried out as per the NSSP for Protection Systems, amending OPSS.PROV 539, which should be included in the Contract; an example NSSP is included in Appendix C. Vibration and noise controls during installation and extraction of any temporary systems, shall meet the same tolerable limits used for installation.

6.6.2 Surface Water and Groundwater Control

It is anticipated that during rehabilitation works, water flow will be maintained within one of the culvert cells, while the repairs are being carried out in or adjacent to the other cell.

For concrete repairs of the culvert, it may be feasible to divert the creek flow and/or pump creek water from behind the temporary dewatering system, depending on the creek water level and flow conditions and groundwater level at the time of construction. However, excavations within the temporary protection/dewatering system that extend below the creek/lake bottom or groundwater level, will require active dewatering to be carried out in advance of any excavation reaching the base level. Alternatively, a tremie concrete plug may be placed at the base of the temporary protection/dewatering system to facilitate working in dry conditions.

Where required, active dewatering methods should draw down the groundwater level approximately 1 m below the base of the excavation and shall be designed/carried out in accordance with OPSS.PROV 517 (Dewatering), as amended by Special Provision SP517F01, and excavations for foundations shall be carried out in accordance with OPSS 902, as amended by SP 109S12 and NSSP FOUN0003, which is provided in Appendix E. The return period flow estimates in Table A of SP517F01 should be filled in by the hydraulic design engineer. Given the

loosening potential of the silty sand to sand and silt and sand subgrade at this site, the Foundation Designer fill in for Note 1 in Table A should indicate “yes”. Given the apparent lack of infrastructure in the vicinity of the culvert, a preconstruction survey is not considered to be required at this site. Further, considering that relatively minor groundwater lowering is anticipated to be required to facilitate the culvert rehabilitation, the risk of settlement impacts is considered to be low from a foundation perspective, so long as pumping is carried out from properly filtered sumps/well points. As such, the foundation designer fill-in in Table A of SP 517F01 should indicate that the preconstruction survey distance is not applicable.

It should be noted that construction water takings in excess of 50 m³/day are regulated by the Ministry of the Environment, Construction and Parks (MECP). Takings of groundwater/stormwater for construction dewatering purposes with a combined total less than 400 m³/day, qualify for self-registration on the MECP’s Environmental Activity and Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking less than 400 m³/day and a Section 53 approval for discharge to the environment; however, a “Water Taking Plan” and a “Discharge Plan” are required by the MECP if water is taken in accordance with an EASR. If construction water taking will be required at this site, the construction water taking permit and registration should be prepared adequately in advance of site excavation work, so as not to unduly affect the construction schedule.

Surface water should be directed away from the excavation areas to prevent ponding of water, that could result in disturbance and weakening of the foundation subgrade.

6.6.3 Excavations

Temporary excavations for the wingwall replacements will be made through existing embankment fill, native gravel deposit, and into the compact to very dense silty sand to sand deposit. As no boreholes were drilled from the Highway 17 road grade, the depth and type of embankment fill is not known. However, based on our observations of the embankment side slopes at the site and MTO construction practices at culvert sites, it is anticipated that the embankment fill is comprised of granular material.

Excavation works must be carried out in accordance with the guidelines outlined in the *Occupational Health and Safety Act* (OHSA) and Regulations for Construction Projects. The existing embankment fill and native soils at this site, should be classified as Type 3 soil according to the OHSA. Where space permits, temporary open-cut excavations through these materials should be made with side slopes formed no steeper than 1H:1V, assuming proper groundwater and surface water control is in place.

6.6.4 Obstructions

Cobbles and boulders should be anticipated within the native site soils, as observed and inferred from casing grinding observed during drilling. The presence of cobbles and boulders will affect excavation efforts and installation of temporary protection systems. It is recommended that a Notice to Contractor be included in the Contract Documents to warn the Contractor of the presence of cobbles and boulders within the site soils; an example Notice to Contractor is presented in Appendix E.

6.6.5 Backfill Materials

Backfill against the wingwalls should consist of free draining granular fill, meeting the specifications for OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type I, II, or III. For backfilling below the water level,

Granular 'B' Type II should be utilized. The granular backfill shall be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*) as amended by SP105S22.

The existing site soils that may have to be excavated from within the footprint of the wingwall foundations may also be used as backfill within the temporary protection system areas.

6.6.6 Erosion Protection

The requirements for and design of erosion protection measures for the replacement wingwall footings (and existing culvert) should be assessed by the hydrology design engineer. Due to the fast-flowing creek water cascading from the culvert outlet and the lack of existing soil cover or creek bed material, consideration should be given to the use of erosion protection (i.e., Rip Rap or Rock Protection) at the base of the wingwalls and at the culvert outlet to reduce the potential for localized erosion to occur. We would further suggest that, as minimum, the erosion protection could consist of R-50 Rip Rap or Rock Protection as per OPSS.PROV 1004 (Aggregates – Miscellaneous) placed in a similar configuration as that shown on OPSD 810.010 (Rip-Rap Layout).

6.6.7 Corrosion Assessment and Protection

The results of an analytical test on one sample of the site soil from one borehole, are summarized in Section 4.2.5 and presented in Appendix C. The potential for sulphate attack and corrosion are discussed in the following sub-sections; however, it is ultimately up to the structural 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 when designing concrete elements.

6.6.7.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 (*"Additional requirements for concrete subjected to sulphate attack"*) for potential sulphate attack on concrete. The sulphate concentration measured in the tested sample is below the exposure class of S-3 (Moderate). Therefore, based on the one soil sample tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

6.6.7.2 Potential for Corrosion

The test results indicate a pH of 5.3 and a resistivity of 24,000 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), a pH below 5.5 is considered strongly acidic, and therefore has an increased potential for corrosion. The resistivity is greater than 10,000 ohm-cm, which indicates that the soil corrosiveness is less than very low ($10,000 > R > 2,000$ ohm-cm), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014). The wingwalls will be located adjacent to the road shoulder and be exposed to de-icing salt. Therefore, concrete should be designed for a "C" type exposure class, as defined by CSA A23.1 Table 1.

6.7 Recommendations for Further Work During Detail Design

For consideration, laboratory testing (direct shear testing) could be carried out on select soil samples to further refine/confirm the parameters used in the stability analyses. Further, consideration should be given to advance an additional borehole on the roadway platform, to confirm the composition of embankment fill material and to confirm the soil parameters used for design of the temporary protection system and dewatering systems.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Anastasia Poliacik, P.Eng. Mr. Jorge Costa, P.Eng., an MTO Foundations Designated Contact and Senior Consultant, conducted an independent quality control review of this report.

Signature Page

Golder Associates Ltd.


Anastasia Poliacik, P.Eng.
Geotechnical Engineer



David Muldowney, P.Eng.
Senior Geotechnical Engineer



Jorge M. A. Costa, P.Eng.
MTO Foundations Designated Contact, Senior Consultant

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ASTM International

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

Ontario Provincial Standard Drawings (OPSD) OPSD 810.010 General Rip-Rap Layout for Sewer and Culvert Outlets

OPSD 3090.100 Foundation, Frost Penetration Depths for Northern Ontario

Ontario Provincial Standard Specifications (OPSS) – Provincial Oriented

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Special Provisions

SP 105S22	Amendment to OPSS 501
SP 109S12	Amendment to OPSS 902
SP 517F01	Dewatering System / Temporary Flow Passage System (Amendment to OPSS 517)

Ontario Water Resource Act

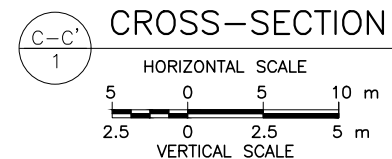
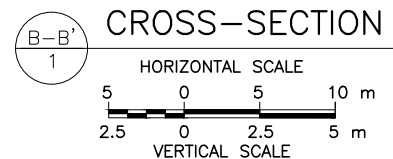
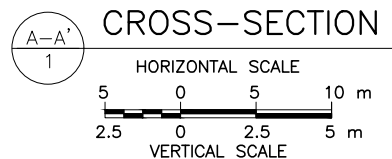
Regulation 903 Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)




Proprietary Software:

Rocscience Inc. SLIDE 2018



GOLDER



-  Borehole – Current Investigation
 N Standard Penetration Test Value
16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
 WL upon completion of drilling
* Indicates N value may have been taken in a disturbed stratum due to flowing/heaving sand conditions

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 13)			
No.	ELEVATION	NORTHING	EASTING
AB-1	192.3	5225177.8	252402.9
AB-2	190.5	5225169.0	252394.9
AB-3	189.3	5225192.1	252327.1
AB-4	189.6	5225181.1	252326.7

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.

Base plans provided in digital format by AECOM CANADA LTD. drawing file no. 17SLA GWP 5376-11-00.dwg, received APRIL 10, 2019 and drawing file no. 60569837-Alona GA to Golder.dwg, received November 18, 2019

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Photograph 1: Highway 17 Roadway Platform, Facing Southwest (July 2019)



Photograph 2: Highway 17 Roadway Platform, Facing Northeast (July 2019)



Photograph 3: Culvert Inlet, Facing Northwest (July 2019)



Photograph 4: Culvert Outlet, Facing East (July 2019)



Photograph 5: Alona Bay Creek, Facing Northeast (January 2018)



Photograph 6: Culvert Outlet, Facing Northeast (January 2018)

APPENDIX A

Record of Borehole Sheets

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

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

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

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

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Cone Penetration Test (CPT)

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

Dynamic Cone Penetration Resistance (DCPT); 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

SAMPLES

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

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, 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
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

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

COARSE-GRAINED SOILS

Compactness¹

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

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

4. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

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

Field Moisture Condition

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

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

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 $= (w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_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 $= \sigma'_p / \sigma'_{vo}$

(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

$$\tau = c' + \sigma' \tan \phi'$$





$$\text{shear strength} = (\text{compressive strength})/2$$

PROJECT 1790414		RECORD OF BOREHOLE No AB-1				1 OF 1 METRIC													
W.P. 5170-16-01		LOCATION N 5225177.8; E 252402.9 NAD83 MTM ZONE 13 (LAT. 47.163912; LONG. -84.691119)				ORIGINATED BY SA													
DIST _____ HWY 17		BOREHOLE TYPE 254 mm O.D. Hollow Stem Augers, Mud Rotary, NW Casing				COMPILED BY TR													
DATUM GEODETIC		DATE May 9, 2019				CHECKED BY DAM													
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 (%)	
192.3	GROUND SURFACE							20	40	60	80	100							
0.0	GRAVEL (GW-GM) and sand, trace organics Loose Brown Wet		1	SS	4	▽	192												
191.6	SILTY SAND (SM), trace gravel, trace clay Compact Brown to grey Wet		2	SS	16		191											7 56 28 9	
0.7	- Some cobbles from 0.7 m to 3.0 m depth		3	SS	27		190												
	- Casing grinding on inferred cobbles / boulders from 2.3 m to 3.0 m depth		4	SS	26		189												
			5	SS	26		188												
			6	SS	21		187												
187.8	SAND (SP-SM) Compact to dense Grey Wet			7	SS		26*	186											0 95 (5)
4.5	- Approximately 150 mm of heave inside casing at 6.1 m depth			8	SS		35*	185											
	- Approximately 300 mm of heave inside casing at 7.6 m depth			9	SS		32*	184											
	- Approximately 300 mm of heave inside casing at 9.1 m depth			10	SS		40*	183											
182.6	END OF BOREHOLE																		
9.8	NOTES: 1. Mud rotary methods used below 3.0 m depth. 2. * Indicates "N" values may have been taken in a disturbed stratum due to flowing/heaving sand conditions. 3. Water level measured inside casing at a depth of 1.2 m below ground surface (Elev. 191.1 m) upon completion of drilling.																		






SUD-MTO 001 S:\CLIENTS\TOH\HWY17&10102_DATA\GINT\1790414.GPJ GAL-MISS.GDT 11-19-19 TR

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

PROJECT 1790414		RECORD OF BOREHOLE No AB-3		1 OF 1 METRIC	
W.P. 5170-16-01		LOCATION N 5225192.1; E 252327.1 NAD83 MTM ZONE 13 (LAT. 47.164035; LONG. -84.692121)		ORIGINATED BY SA	
DIST _____ HWY 17		BOREHOLE TYPE 254 mm O.D. Hollow Stem Augers, Mud Rotary, NW Casing		COMPILED BY TR	
DATUM GEODETIC		DATE May 8, 2019		CHECKED BY DAM	

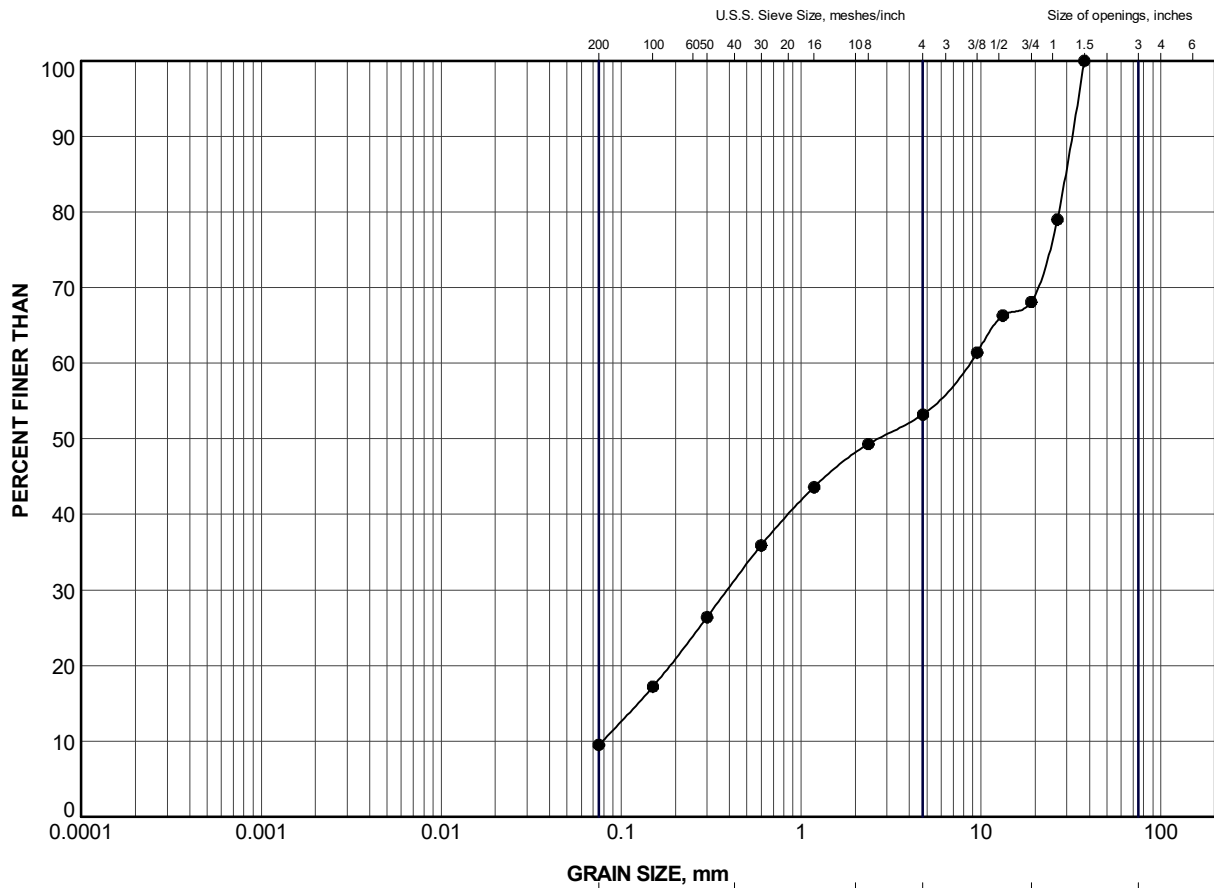
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W _p	W	W _L					
189.3	GROUND SURFACE																			
0.0	GRAVEL (GW-GM) and sand, trace silt, trace organics Loose Brown Moist to wet		1	SS	4															
188.1			2A	SS	7															
1.2	SILTY SAND (SM), trace clay Compact to dense Brown to grey Wet		2B																	
			3	SS	18												0 63 30 7			
			4	SS	25															
	- Casing grinding on inferred cobbles / boulders from 3.0 m to 3.8 m depth		5	SS	33												0 56 37 7			
			6	SS	25												0 56 34 10			
			7	SS	27															
183.7	SAND (SP), trace silt Compact to dense Grey Wet																			
5.6	- Approximately 90 mm of heave inside casing at 6.1 m depth		8	SS	52*															
	- Approximately 75 mm of heave inside casing at 7.6 m depth		9	SS	27*															
	- Approximately 460 mm of heave inside casing at 9.1 m depth		10	SS	32*												0 96 (4)			
179.5	END OF BOREHOLE																			
9.8	NOTES: 1. Mud rotary methods used below 3.0 m depth. 2. * Indicates "N" values may have been taken in a disturbed stratum due to flowing/heaving sand conditions. 3. Water level measured in casing at a depth of 0.2 m below ground surface (Elev. 189.1 m) upon completion of drilling. Artesian condition noted at ground surface (Elev. 189.3 m) inside borehole and outside of casing at time of boreole abandonment.																			

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PROJECT 1790414		RECORD OF BOREHOLE No AB-4				1 OF 1 METRIC								
W.P. 5170-16-01		LOCATION N 5225181.1; E 252326.7 NAD83 MTM ZONE 13 (LAT. 47.163936; LONG. -84.692125)				ORIGINATED BY SA								
DIST _____ HWY 17		BOREHOLE TYPE 254 mm O.D. Hollow Stem Augers, Mud Rotary, NW Casing				COMPILED BY TR								
DATUM GEODETIC		DATE May 8, 2019				CHECKED BY DAM								
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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
189.6 0.0	GROUND SURFACE GRAVEL (GW-GM) and sand, trace organics Compact Brown Moist to wet - Approximately 100 mm diameter cobble encountered at 0.5 m depth		1	SS	16	189							47 43 (10)	
			2	SS	27									
188.1 1.5	SILTY SAND (SM), trace clay Compact Brown to grey Wet		3	SS	20	188							0 57 36 7	
187.4 2.2	SAND (SP), trace silt Compact to very dense Brown to grey Wet		4	SS	28	187							0 96 (4)	
			5	SS	54	186								
185.6 4.0	SILTY SAND (SM), trace clay Compact Brown to grey Wet		6A	SS	19	185							0 59 34 7	
			6B											
			7	SS	24	184								
184.0 5.6	SAND (SP-SM) Dense Grey Wet		8	SS	44*	183								
						182							0 88 (12)	
			9	SS	40*									
						181								
	- Approximately 300 mm of heave inside casing at 9.1 m depth		10	SS	31*	180								
179.8 9.8	END OF BOREHOLE													
NOTES: 1. Mud rotary methods used below 3.0 m depth. 2. * Indicates "N" values may have been taken in a disturbed stratum due to flowing/heaving sand conditions. 3. Water level measured inside casing at a depth of 0.1 m below ground surface (Elev. 189.5 m) upon completion of drilling. Artesian condition noted at ground surface (Elev. 189.6 m) inside borehole and outside of casing at time of borehole abandonment.														

APPENDIX B

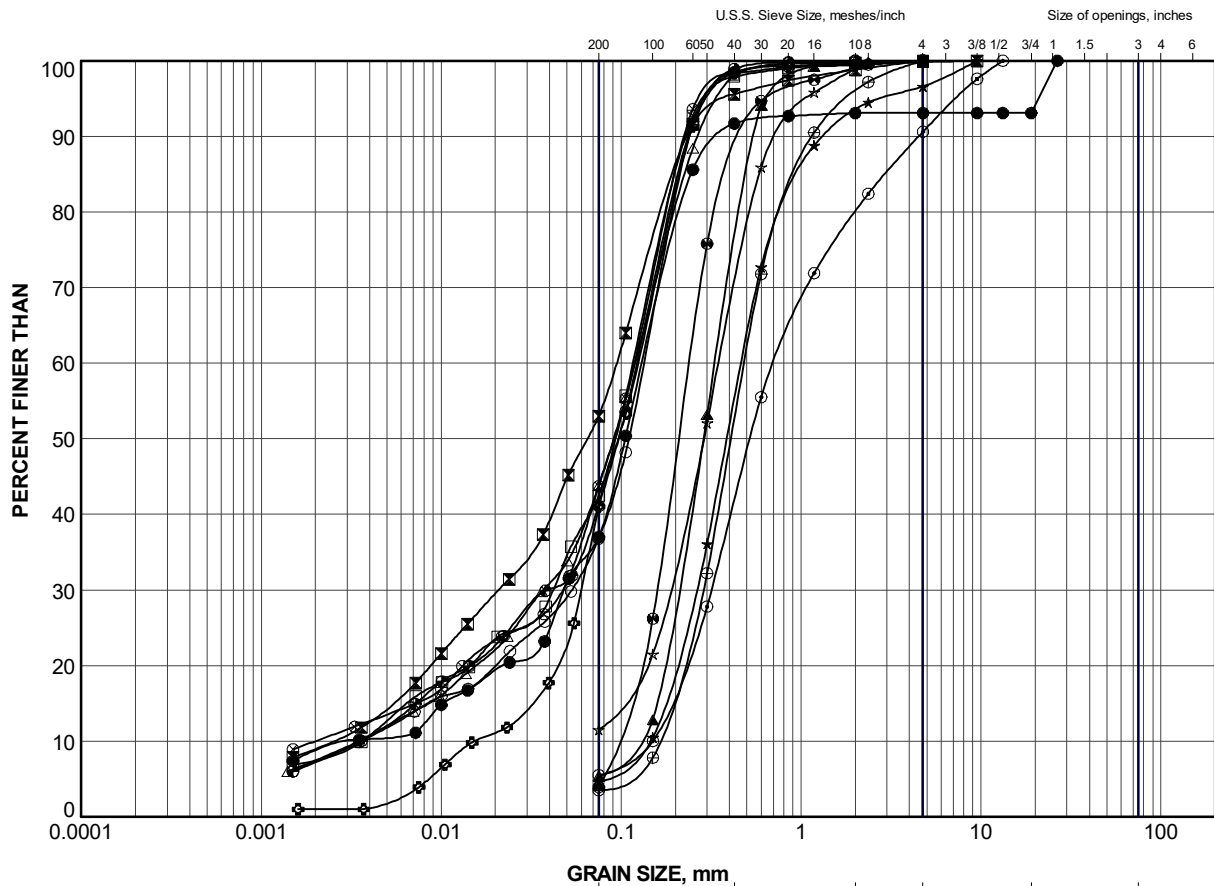
Geotechnical Laboratory Test Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	AB-4	1	189.3

PROJECT					
HIGHWAY 17 ALONA BAY CREEK CULVERT					
TITLE					
GRAIN SIZE DISTRIBUTION GRAVEL (GW-GM)					
PROJECT No. 1790414			FILE No. 1790414.GPJ		
DRAWN	TR	Oct 2019	SCALE	N/A	REV.
CHECK	DAM	Oct 2019			
APPR	JMAC	Oct 2019			
GOLDER			FIGURE B-1		
SUDBURY, ONTARIO					

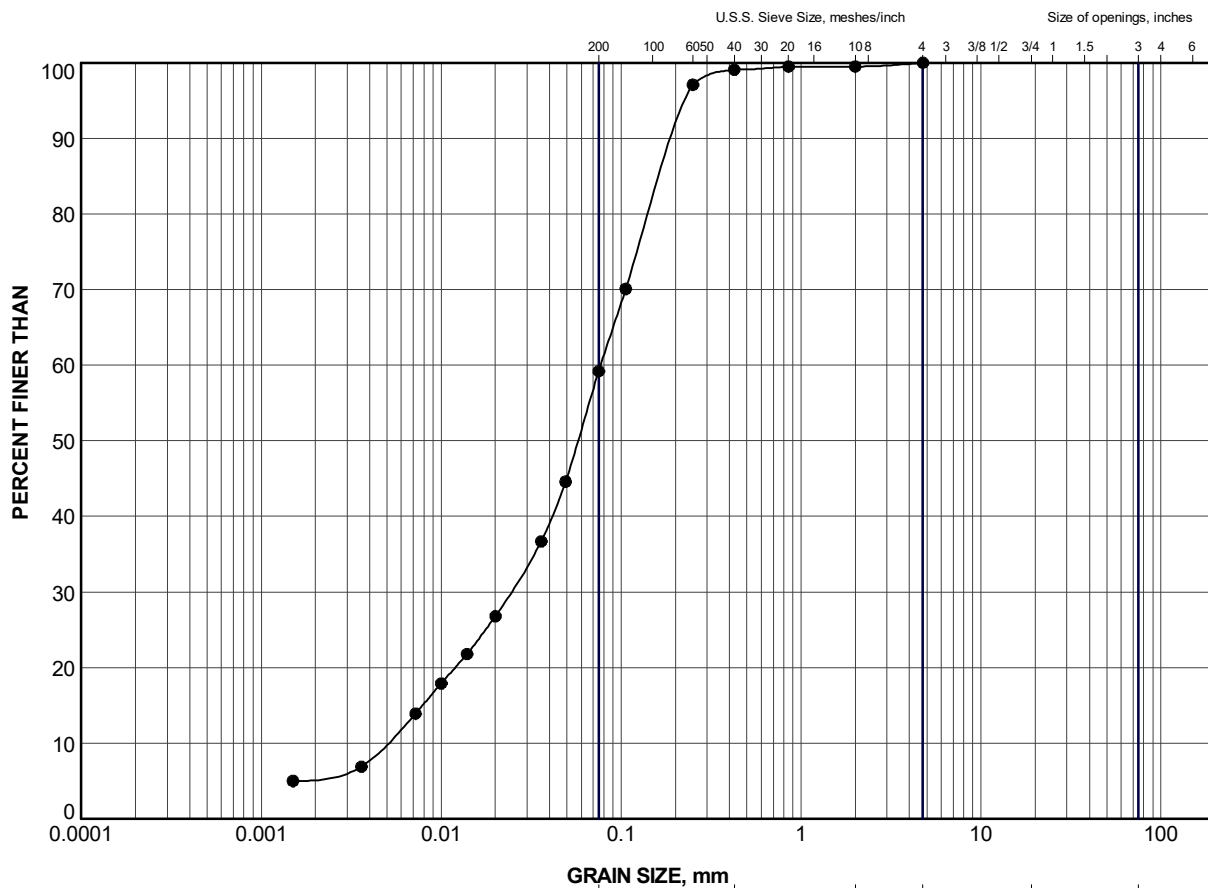


CLAY AND SILT						Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	AB-1	2	191.2
⊠	AB-1	4	189.7
▲	AB-1	7	187.4
★	AB-2	2	188.7
⊙	AB-2	4A	187.2
⊗	AB-2	7	184.1
○	AB-3	3	187.5
△	AB-3	5	186.0
⊗	AB-3	6	185.2
⊕	AB-3	10	179.9
□	AB-4	3	187.8
⊗	AB-4	5	186.3
⊙	AB-4	7	184.7
☆	AB-4	9	181.7

PROJECT					
HIGHWAY 17 ALONA BAY CREEK CULVERT					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY SAND (SM) to SAND (SP)					
PROJECT No.		1790414		FILE No.	
DRAWN		TR		Oct 2019	
CHECK		DAM		Oct 2019	
APPR		JMAC		Oct 2019	
SCALE		N/A		REV.	
GOLDER		SUDBURY, ONTARIO		FIGURE B-2	



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	AB-2	5	186.4

PROJECT					
HIGHWAY 17 ALONA BAY CREEK CULVERT					
TITLE					
GRAIN SIZE DISTRIBUTION SILT (ML) and SAND					
PROJECT No. 1790414			FILE No. 1790414.GPJ		
DRAWN	TR	Oct 2019	SCALE	N/A	REV.
CHECK	DAM	Oct 2019			
APPR	JMAC	Oct 2019			
GOLDER SUDBURY, ONTARIO			FIGURE B-3		

APPENDIX C

Analytical Laboratory Test Results

Your Project #: 1790414
Your C.O.C. #: 712313-01-01

Attention: David Muldowney

Golder Associates Ltd
33 Mackenzie Street
Suite 100
Sudbury, ON
Canada P3C 4Y1

Report Date: 2019/05/29
Report #: R5729598
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B9C8163

Received: 2019/05/13, 15:20

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	3	2019/05/15	2019/05/16	CAM SOP-00463	SM 4500-Cl E m
Conductivity	3	2019/05/17	2019/05/17	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 3)	3	2019/05/21	2019/05/21	BBY8SOP-00017	BCMOE BCLM Dec2000 m
Sulphide in Soil (1)	3	2019/05/21	2019/05/23	BBY6SOP-00052 BBY6SOP-00006	EPA-821-R-91-100 m
pH CaCl ₂ EXTRACT	3	2019/05/15	2019/05/15	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2019/05/14	2019/05/17	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	3	2019/05/15	2019/05/16	CAM SOP-00464	EPA 375.4 m
Redox Potential (2, 4)	3	N/A	N/A		

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing. Maxxam is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by Maxxam, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

- (1) This test was performed by Campo to Burnaby - Offsite
- (2) This test was performed by Sub from Campo to Env. Testing Canada (Eurofins)
- (3) Offsite analysis requires that subcontracted moisture be reported.

Your Project #: 1790414
Your C.O.C. #: 712313-01-01

Attention: David Muldowney

Golder Associates Ltd
33 Mackenzie Street
Suite 100
Sudbury, ON
Canada P3C 4Y1

Report Date: 2019/05/29
Report #: R5729598
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B9C8163

Received: 2019/05/13, 15:20

(4) Oxidation-Reduction Potential (ORP) values are determined using a Ag/AgCl reference electrode.

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Alisha Williamson, Project Manager

Email: AWilliamson@maxxam.ca

Phone# (613)274-0573

=====

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

RESULTS OF ANALYSES OF SOIL

Maxxam ID		JSA209		JSA210			JSA210		
Sampling Date		2019/05/09 11:00		2019/05/11 10:30			2019/05/11 10:30		
COC Number		712313-01-01		712313-01-01			712313-01-01		
	UNITS	AB-1	QC Batch	NR-2	RDL	QC Batch	NR-2 Lab-Dup	RDL	QC Batch
CONVENTIONALS									
Sulphide	ug/g	<0.30	6139910	<0.30	0.30	6139910			
Calculated Parameters									
Resistivity	ohm-cm	24000	6120894	5800		6120894			
Inorganics									
Soluble (20:1) Chloride (Cl-)	ug/g	<20	6122300	60	20	6122300			
Conductivity	umho/cm	42	6127141	174	2	6127141	177	2	6127141
Available (CaCl2) pH	pH	5.34	6122353	6.96		6122355			
Soluble (20:1) Sulphate (SO4)	ug/g	<20	6122302	<20	20	6122302			
Physical Testing									
Moisture-Subcontracted	%	16	6139909	9.9	0.30	6139909			
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate									

Maxxam ID		JSA211			JSA211		
Sampling Date		2019/05/11 15:00			2019/05/11 15:00		
COC Number		712313-01-01			712313-01-01		
	UNITS	NR-5	RDL	QC Batch	NR-5 Lab-Dup	RDL	QC Batch
CONVENTIONALS							
Sulphide	ug/g	<0.50	0.50	6139910	<0.50	0.50	6139910
Calculated Parameters							
Resistivity	ohm-cm	8100		6120894			
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g	41	20	6122300			
Conductivity	umho/cm	123	2	6127141			
Available (CaCl2) pH	pH	7.31		6122353			
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	6122302			
Physical Testing							
Moisture-Subcontracted	%	18	0.30	6139909			
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate							

TEST SUMMARY

Maxxam ID: JSA209
Sample ID: AB-1
Matrix: Soil

Collected: 2019/05/09
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6122300	2019/05/15	2019/05/16	Deonarine Ramnarine
Conductivity	AT	6127141	2019/05/17	2019/05/17	Kazzandra Adeva
Moisture (Subcontracted)	BAL	6139909	2019/05/21	2019/05/21	William Zou
Sulphide in Soil	SPEC/UVVS	6139910	2019/05/21	2019/05/23	David Huang
pH CaCl2 EXTRACT	AT	6122353	2019/05/15	2019/05/15	Gnana Thomas
Resistivity of Soil		6120894	2019/05/17	2019/05/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6122302	2019/05/15	2019/05/16	Deonarine Ramnarine
Redox Potential	COND	6146214	2019/05/29		Katherine Szozda

Maxxam ID: JSA210
Sample ID: NR-2
Matrix: Soil

Collected: 2019/05/11
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6122300	2019/05/15	2019/05/16	Deonarine Ramnarine
Conductivity	AT	6127141	2019/05/17	2019/05/17	Kazzandra Adeva
Moisture (Subcontracted)	BAL	6139909	2019/05/21	2019/05/21	William Zou
Sulphide in Soil	SPEC/UVVS	6139910	2019/05/21	2019/05/23	David Huang
pH CaCl2 EXTRACT	AT	6122355	2019/05/15	2019/05/15	Gnana Thomas
Resistivity of Soil		6120894	2019/05/17	2019/05/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6122302	2019/05/15	2019/05/16	Deonarine Ramnarine
Redox Potential	COND	6146214	2019/05/29		Katherine Szozda

Maxxam ID: JSA210 Dup
Sample ID: NR-2
Matrix: Soil

Collected: 2019/05/11
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	6127141	2019/05/17	2019/05/17	Kazzandra Adeva

Maxxam ID: JSA211
Sample ID: NR-5
Matrix: Soil

Collected: 2019/05/11
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6122300	2019/05/15	2019/05/16	Deonarine Ramnarine
Conductivity	AT	6127141	2019/05/17	2019/05/17	Kazzandra Adeva
Moisture (Subcontracted)	BAL	6139909	2019/05/21	2019/05/21	William Zou
Sulphide in Soil	SPEC/UVVS	6139910	2019/05/21	2019/05/23	David Huang
pH CaCl2 EXTRACT	AT	6122353	2019/05/15	2019/05/15	Gnana Thomas
Resistivity of Soil		6120894	2019/05/17	2019/05/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6122302	2019/05/15	2019/05/16	Deonarine Ramnarine
Redox Potential	COND	6146214	2019/05/29		Katherine Szozda

Maxxam Job #: B9C8163
Report Date: 2019/05/29

Golder Associates Ltd
Client Project #: 1790414
Sampler Initials: SA

TEST SUMMARY

Maxxam ID: JSA211 Dup
Sample ID: NR-5
Matrix: Soil

Collected: 2019/05/11
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphide in Soil	SPEC/UVVS	6139910	2019/05/21	2019/05/23	David Huang

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	5.0°C
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Results relate only to the items tested.

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1790414
Sampler Initials: SA

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
6122300	Soluble (20:1) Chloride (Cl-)	2019/05/16	114	70 - 130	106	70 - 130	<20	ug/g	NC	35
6122302	Soluble (20:1) Sulphate (SO4)	2019/05/16	101	70 - 130	101	70 - 130	<20	ug/g	20	35
6122353	Available (CaCl2) pH	2019/05/15			100	97 - 103			0.68	N/A
6122355	Available (CaCl2) pH	2019/05/15			100	97 - 103			0.025	N/A
6127141	Conductivity	2019/05/17			104	90 - 110	<2	umho/cm	1.8	10
6139909	Moisture-Subcontracted	2019/05/21					<0.30	%		
6139910	Sulphide	2019/05/23	89	75 - 125	107	75 - 125	<0.50	ug/g	NC	30

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference $\leq 2 \times \text{RDL}$).

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).



Anastassia Hamanov, Scientific Specialist




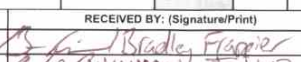
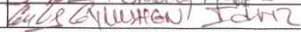


David Huang, BBY Scientific Specialist

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

CHAIN OF CUSTODY RECORD

Page of

INVOICE TO:			REPORT TO:			PROJECT INFORMATION:			Laboratory Use Only:		
Company Name: #7575 Golder Associates Ltd			Company Name: Golder Associates			Quotation #: B80683			Maxxam Job #:		Bottle Order #:
Attention: Accounts Payable			Attention: David Muldowney			P.O. #:					
Address: 33 Mackenzie Street Suite 100			Address: 33 Mackenzie St. Suite 100			Project: 1790414					712313
Sudbury ON P3C 4Y1			Sudbury ON P3C 4Y1			Project Name:			COC #:		Project Manager:
Tel: (705) 524-6861 Fax: (705) 524-1984			Tel: 705 524-6861 Fax:			Site #:					Alisha Williamson
Email: AP_CustomerService@golder.com			Email: D-Muldowney@golder.com			Sampled By:			CH712313-01-01		
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY											
Regulation 153 (2011)			Other Regulations			Special Instructions					
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Medium/Fine			<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw			Field Filtered (please circle): Metals / Hg / Cr VI					
<input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse			<input type="checkbox"/> Reg 558 <input type="checkbox"/> Storm Sewer Bylaw								
<input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other <input type="checkbox"/> For RSC			<input type="checkbox"/> MISA Municipality								
<input type="checkbox"/> Table			<input type="checkbox"/> PWQO								
<input type="checkbox"/> Other			<input type="checkbox"/> Other								
Include Criteria on Certificate of Analysis (Y/N)?											
Sample Barcode Label		Sample (Location) Identification		Date Sampled	Time Sampled	Matrix	Soil Corrosivity Package				
1 AB-1		Alona Bay CK HWY 17 Culvert		May 09/19	11:00	Soil	<input checked="" type="checkbox"/>				
2 NR-2		NAT RIVER Bridge HWY 101		May 11/19	10:30	Soil	<input checked="" type="checkbox"/>				
3 NR-5		NAT RIVER Bridge HWY 101		May 11/19	15:00	Soil	<input checked="" type="checkbox"/>				
4											
5											
6											
7											
8											
9											
10											
* RELINQUISHED BY: (Signature/Print)				Date: (YY/MM/DD)		Time		RECEIVED BY: (Signature/Print)		Date: (YY/MM/DD)	
				19/05/13		15:20				20/05/13 15:20	
										20/05/14 08:56	
								# jars used and not submitted		Laboratory Use Only	
										Time Sensitive	
										Temperature (°C) on Receipt	
										5.5, 5°C	
										Custody Seal	
										Present	
										Intact	
										Yes	
										No	
* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO MAXXAM'S STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.MAXXAM.CA/TERMS.											
* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.											
** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT HTTP://MAXXAM.CA/WP-CONTENT/UPLOADS/ONTARIO-COC.PDF.											
White: Maxxa Yellow: Client											
SAMPLES MUST BE KEPT COOL (< 10°C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM											

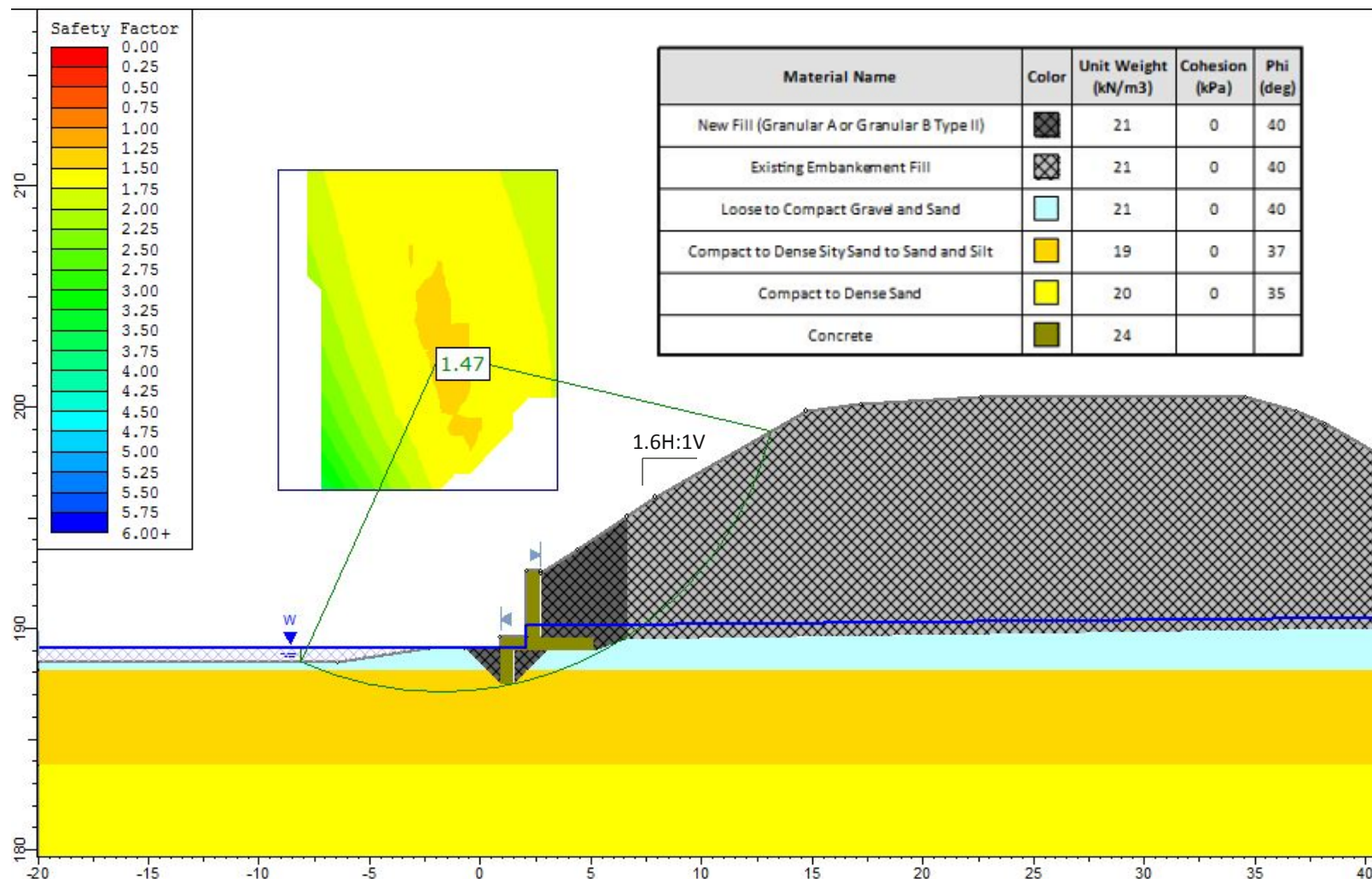
APPENDIX D

Global Stability Analysis

Global Stability Analysis Proposed Wingwall

Figure D-1

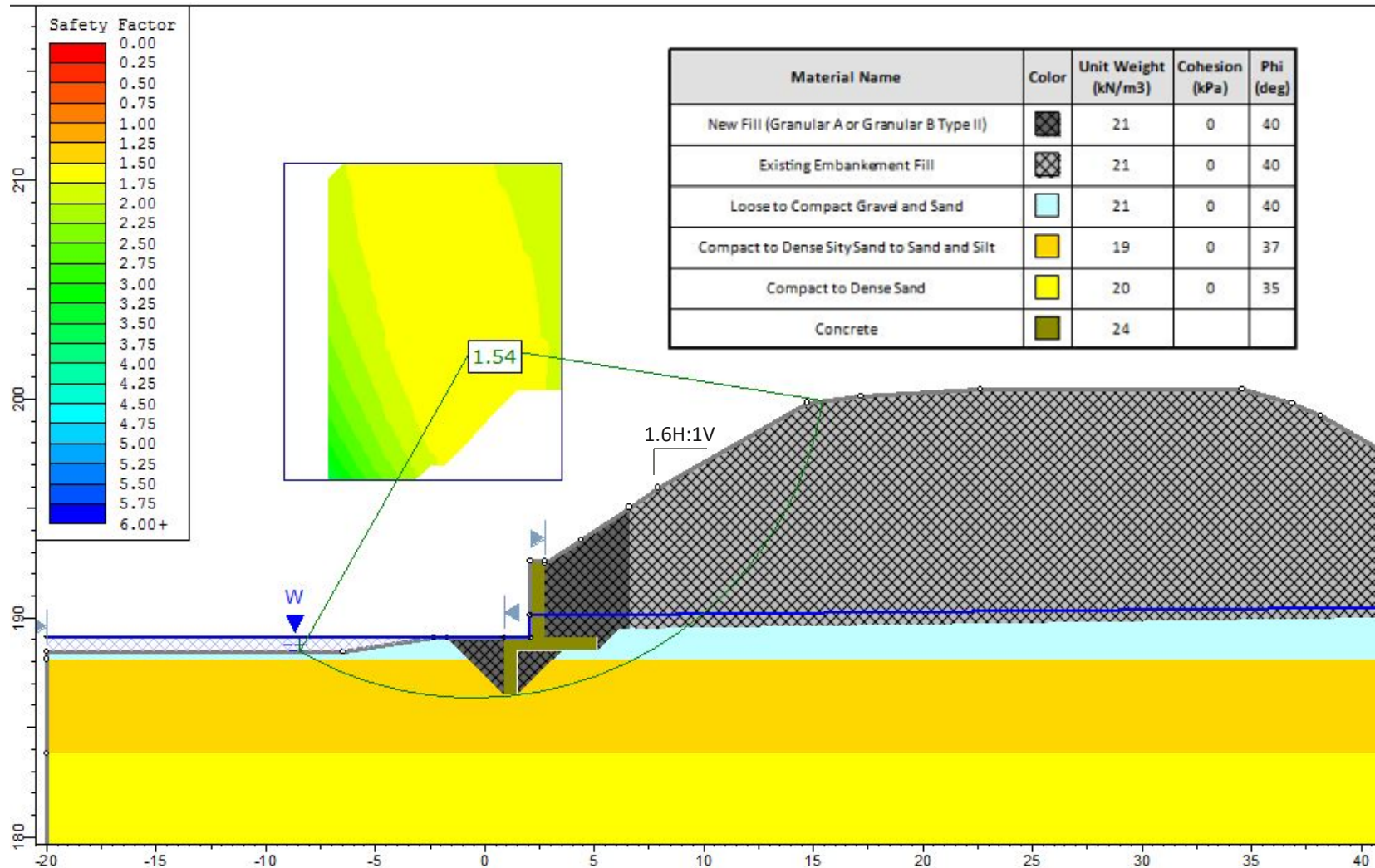
Founded at Elevation 188.9m, 1.5m Shear Key



Global Stability Analysis Proposed Wingwall

Figure D-2

Founded at Elevation 188.5m, 2.0m Shear Key



APPENDIX E

Non-Standard Special Provisions

COFFERDAMS – Item No.

Non-Standard Special Provision

SCOPE

This specification covers the requirements for the design, construction, maintenance, monitoring, and removal of a cofferdam made necessary by excavation, embankment construction, dewatering, unwatering, or other work.

As part of the work under this item, the Contractor shall:

- Design, supply, install, maintain, monitor and remove cofferdams as required to construct, repair, rehabilitate structures and all associated works in the dry for the following culverts shown on the Contract Drawings:
 - Alona Bay Creek Culvert Rehabilitation (Site No. 38S-0353/C0)

REFERENCES

Ontario Provincial Standard Specifications, Construction

OPSS.PROV 517	Dewatering
OPSS 805	Temporary Erosion and Sediment Control Measures
OPSS.PROV 539	Temporary Protection Systems
OPSS 903	Deep Foundations
OPSS.PROV 904	Concrete Structures
OPSS 906	Structural Steel for Bridges
OPSS 942	Prestressed Soil and Rock Anchors
OPSS.PROV 404	Support Systems

Ontario Ministry of Labour

Occupational Health and Safety Act, R.S.O. 1990, c.O.1, as amended

CSA Standards

S6-06 Canadian Highway Bridge Design Code

DEFINITIONS

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

Anchor means a) A system consisting of prestressed tendons or non-prestressed rods installed in predrilled holes and encapsulated in grout or concrete. A system that derives its load carrying capacity in bond between the grout and concrete body and the surrounding soil or rock; or b) A tie back to a deadman.

Bracing means the system of walers, struts, anchorages, and like members that connect frames or shoring panels of a sheathing system to resist external pressures and to provide stability against lateral movement.

Certificate of Conformance means a document issued by the Cofferdam Design Engineer and accepted by the Contract Administrator that the specified components of the Work are in General Conformance with the requirements of the Contract Documents.

Cofferdam means a watertight enclosure as defined in OPSS.PROV 539.

Cofferdam Design Engineer means an engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of bridges / structures / cofferdams. In addition, the Cofferdam Design Engineer shall have had relevant experience in the design of at least 5 other cofferdams. The Contractor shall retain the Cofferdam Design Engineer to ensure conformance with the contract documents and issue certificate(s) of conformance for the design.

Dredge Line means the exposed lower limit of the cofferdam or protection system.

Erector means a supervisory person that undertakes the construction of a cofferdam or protection system.

Protection System means the construction necessary to mechanically support existing or proposed work so that its function shall not be affected, or construction necessary to support work such as open excavations / cofferdam systems during actual construction operations for safety and convenience.

Raker means a structural member inclined to the front of the shoring wall providing lateral support.

Shoring Wall means a structural wall consisting of wood, steel, or concrete or any combination of these materials that supports earth or rock and any structure, materials, utilities, or other facility contained in or on the supported earth or rock mass.

Stamped refers to drawings or details that have been reviewed and stamped "Conforms With Contract Documents". The stamp shall include the date and signature of a Professional Engineer registered in Ontario (P.Eng.).

Top of Shoring Wall means the upper limit of the protection system or cofferdam.

DESIGN AND SUBMISSION REQUIREMENTS

General

The cofferdam shall be designed to be a watertight enclosure from which the body of water within the enclosed area can be pumped out to create a dry work environment for the work shown in the Contract Documents to be carried out. The cofferdam includes any unwatering and dewatering required to provide safety and protection for any excavations or work zones at the locations as specified in the Contract Documents and at any other location where the stability, safety, or function of an existing structure or utility may be impaired by construction work.

A cut-off and/or dewatering system shall be designed as part of the cofferdam to control water seepage and the flow of water into the excavation or work zone, prevent disturbance of the foundation and work zone subgrade (including instability or basal heave or excessive seepage from unbalanced water pressures), permit the placing of materials and concrete in the dry, and complete the excavating and backfilling for the work shown on the Contract Drawings.

The cofferdam shall be designed for the performance level as specified in the Contract Documents or as specified by the Cofferdam Design Engineer.

Except for Owner designed cofferdams, the Contractor shall be responsible for the complete detailed design of the cofferdam required to carry out the work as specified in the Contract Documents.

Cofferdams that are not as specified in the Contract Documents shall be assigned an appropriate performance level for design by the Cofferdam Design Engineer. The Contract Administrator shall review the performance level selected at the time of submission of the specified Working Drawings.

The geotechnical and foundation portions of the design shall be based on a method published in AASHTO Guide Design Specification for Bridge Temporary Works and in general conformance with CAN/CSA-S6. The design shall be appropriate for the specific site conditions. Design methods not meeting the AASHTO and CSA design specifications may only be used on this Contract, if approved by the Owner.

The foundation soils may contain construction debris, possible cobbles/cobbles, wood layers and fragments, and fibrous peat soils. Reference to the Foundation Investigation Reports shall be made to ensure cofferdam design and construction methods are suitable for the anticipated subsurface conditions.

Dewatering, unwatering, and activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS.PROV 517.

The cofferdam system (including any cut-off / dewatering / unwatering) 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.

Cofferdam systems may require design and installation as temporary protection systems. The temporary slope geometry used to determine requirements of the protection system shall be according to the Occupational Health and Safety Act. Temporary protection systems must be designed and constructed in accordance with OPSS.PROV 539.

Performance levels for cofferdams and any related temporary protection systems are as follows:

Performance Level	Maximum Angular Distortion	Maximum Horizontal Displacement
1a	1:1000	5 mm
1b	1:1000	10 mm
2	1:200	25 mm
3	1:100	50 mm

Where:

Angular Distortion = $\pm \Delta/H$

Δ = Horizontal displacement in mm at height H

H = Height in mm above dredge line / bottom of cofferdam to point of measurement or height above the nearest system restraining support.

When Performance Level 1a is specified, the bracing system shall be preloaded.

Where the bracing systems are preloaded, the effects of the preload shall not cause damage to adjacent facilities.

Cofferdam systems with a face within a horizontal distance of $1/3H$ of any part of a structure foundation shall be designed for Performance Level 1a.

Designer Qualifications

The Cofferdam Design Engineer and Design-Checking Engineer shall have demonstrated expertise for the work. As well, the design Engineer and design-checking Engineer shall have a minimum of 5 years experience in designing cofferdams of similar nature and scope to the required work.

One person shall not perform both the design Engineer and design-checking Engineer roles for a protection system.

Design Assumptions

The design assumptions shall accurately represent the subsurface conditions and water levels prevalent at the site and shall be specific to the type of cofferdam system used. The design shall address the subsurface conditions and water conditions at the project site as specified in the Contract Documents.

Vertical and Horizontal Loadings

Vertical and horizontal design loadings used shall represent existing conditions and accepted design practice. Future loadings that are known and may affect the cofferdam during its useful life shall be considered. Settlement of the cofferdam during its service life and the influence on adjacent settlement sensitive structures / utilities must meet tolerable criteria associated with each structure / utility.

Submission Requirements

The Contractor shall submit 3 sets of Working Drawings to the Contract Administrator at least 7 Days prior to commencement of the cofferdam system installation, for information purposes only. Prior to making a submission, the seals and signatures of the cofferdam design Engineer and a design-checking Engineer shall be affixed on the Working Drawings verifying that the drawings are consistent with the Contract Documents.

Where multi-discipline engineering work is depicted on the same Working Drawing and the Design Engineer or Design-Checking Engineer, or both, are unable to seal and sign the Working Drawing for all aspects of the work, the drawing shall be sealed and signed by as many additional Design and Design-Checking Engineers as necessary.

Prestressed anchor submissions shall be according to OPSS 942.

When other authorities are involved, 1 set of Working Drawings shall be submitted for each authority at least 5 weeks prior to the commencement of cofferdam construction. The requirements of each authority shall be satisfied prior to commencement of the cofferdam installation.

The Contractor shall have a copy of the Working Drawings at the site during cofferdam installation.

For cofferdams that are not specified in the Contract Documents, the Contractor shall submit the Working Drawings for these systems to the Contract Administrator at least 3 weeks prior to the commencement of any construction.

The following information and details shall be shown on the Working Drawings:

- a) Plans, Elevations, and Details
 - i. Location of cofferdam system and station limits.
 - ii. Plan and elevation of cofferdam showing the extent of the cofferdam / cut-off system / freeboard, etc.
 - iii. Details of the cofferdam system, including cross-sections.
 - iv. Details of internal / external bracing.
- b) Design Criteria
 - i. Pressure diagrams including values of horizontal and vertical loads, dead load, and live load surcharge.
 - ii. Design assumptions and parameters.
 - iii. Anchor bond stresses.
 - iv. Pile design.
 - v. Anchor system stressing schedule specifying working loads, stressing loads, and lock in loads.

- vi. Details of preload, when required.
 - vii. For cofferdam systems with performance levels not specified in the Contract Document, the performance level shall be designated.
- c) Materials
- i. Grade of structural steel and grade and species of structural wood.
 - ii. Concrete strengths.
 - iii. Grout strengths.
 - iv. Details of protection from rain and frost action.
 - v. Wood lagging and size.
 - vi. Mill certificates or test reports from an independent organization certified by the Standards Council of Canada certifying that the steel meets the requirements of the grade, where specified.
 - vii. Details of patented accessories, including load test data.
- d) Installation Procedure
- i. Installation sequence and procedure, including to the installation of piling, lagging, anchor systems, and rakers.
- e) Monitoring Method
- i. The proposed method of monitoring the performance of the cofferdams during installation and use. The method of monitoring shall be consistent with the requirements specified in the Quality Control subsection.
- f) Removal of Cofferdams
- i. The details of the procedures associated with the removal of the cofferdams indicating: method, sequence of work, and removal limits, except when the cofferdam is specified in the Contract Documents to be left in place.

Amendments to Cofferdams

Work shall not proceed on amendments to the protection system until the Contractor has received sealed and signed approval to proceed from the Design Engineer and Design-Checking Engineer and has submitted a copy of the approval to the Contract Administrator.

Amendments to the cofferdams shall be submitted to the Contract Administrator on revised Working Drawings bearing the seal and signature of the Design Engineer and Design-Checking Engineer.

Preconstruction Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator, a condition survey of property and structures that may be affected by the work. The survey shall include the locations and conditions of adjacent properties; buildings; underground structures; utility services; and structures, such as walls abutting the site within a horizontal distance of $2H_w$ from the inside face of the cofferdam, where H_w is the height of the wall from the water surface to the dredge line / watercourse bed.

Unwatering

At least two (2) weeks prior to the commencement of cofferdam construction, the Contractor shall submit a detailed unwatering scheme to the Contract Administrator, for information purposes only, showing the unwatering methods and measures to complete the work in dry conditions.

Cofferdams shall be watertight to facilitate work in dry conditions. Pumping from the interior of cofferdams shall be done in such a way as to preclude the possibility of the flow of water through any fresh concrete.

The submission shall also provide details of the proposed methods of preventing unwatering or displaced water from directly entering the culverts.

Certificates of Conformance

The Cofferdam Design Engineer shall inspect the installation of each cofferdam prior to any unwatering or lowering of the water on the inside face of the cofferdam. After the installation of the cofferdam has been completed, but before lowering the water level, the Contractor shall submit a Certificate of Conformance to the Contract Administrator, sealed and signed by the Cofferdam Design Engineer. The Certificates of Conformance shall state that the cofferdam is in place, and has been installed in conformance with the stamped shop drawings and the Contract Drawings. A separate Certificate of Conformance shall be submitted upon removal of the cofferdam system, sealed and signed by the Cofferdam Design Engineer stating that the cofferdam was installed, monitored, and subsequently removed in general conformance with the Working Drawings and as specified in the Contract Documents.

Separate Certificates of Conformance are required, each to coincide with each cofferdam location for each culvert.

Construction

General

The Contractor shall be responsible for the design, materials, construction, maintenance, monitoring, and removal of a cofferdam.

The erector shall be experienced in the method of construction of cofferdam systems. Such experience shall have been obtained within the preceding 5 years on projects of similar nature and scope to the required work.

Cofferdams shall be built according to the specifications and the Working Drawings.

Piling shall be according to OPSS.PROV 903 and the Working Drawings.

Concrete construction shall be according to OPSS 904. Concrete shall be placed in the dry.

Where cofferdams are used, they shall be sealed sufficiently to permit culvert rehabilitation / replacement and associated works to be performed in the dry.

The Contractor shall carry out dewatering, as required, to facilitate the installation of the cofferdam system. Groundwater cut-off systems may be required as part of the cofferdam system to allow for dry

conditions within the work zone.

All concrete, including tremie concrete, shall be placed according to the requirements of OPSS 904. Structural steel shall be according to OPSS 906 and the Working Drawings. Prestressed anchors shall be supplied, installed, and stressed according to OPSS 942.

The cofferdam shall be protected from the detrimental effects of rain and frost action. Material used in the cofferdam shall remain the property of the Contractor.

Water seepage through and/or loss of soil from behind or below the cofferdam shall be prevented during and following the installation.

Removal of Cofferdams

Cofferdams must be removed upon completion of the work, unless otherwise specified and accepted by the Contract Administrator.

Where cofferdam systems are left in place, the top shall be removed to at least 1.2 m below the finished grade or ground level or at least 0.6 m below the streambed / bottom of watercourse.

Where cofferdams are removed, the method and sequence of removal shall be so that there shall be no damage to the new work, existing work, and facility being protected. Removal of the dewatering / unwatering system and cofferdam shall be in such a manner to allow water elevation to slowly return to natural elevations.

All disturbed areas shall be restored to an equivalent or better condition than existed prior to the commencement of construction.

The Contractor shall remove cofferdam once rehabilitation work is complete. Care shall be taken not to damage finished material of the permanent work.

Quality Control

General

In addition to the quality control measures instituted by the Contractor, the Contractor shall complete a preconstruction condition survey and monitor the cofferdam system installation as specified herein, and as shown on the Working Drawings.

Inspection of Welds

The Contractor shall be responsible for visual inspection of all welds. Any required testing of welds shall be as specified by the design Engineer of the cofferdam system.

Monitoring

General

Monitoring shall be conducted by a Registered Ontario Land Surveyor or an Engineer according to the program submitted with the Working Drawings. The minimum requirements for monitoring shall include

the survey measurements of scaled targets attached to the cofferdam wall at the elevations specified. The scaled targets shall be placed at a maximum spacing of 6 m with targets placed at the extreme ends and the targets distributed between the upper and lower limits. The survey targets shall be monitored for horizontal and vertical displacement at the frequency specified.

All test results, observations, and records, including the preconstruction survey, taken during construction and operation of the cofferdam system shall be available on the site for review by the Contract Administrator.

If movement of the cofferdam system is more rapid than is expected, or if movement approaches the allowable limit, the Contract Administrator shall be notified immediately and suitable measures shall be taken to ensure stability of the cofferdam and to ensure movement does not exceed the performance level specified in the Contract Documents.

Retained Water Depths Less Than or Equal to Three Metres

The cofferdam systems shall be monitored during construction. Readings shall be taken during installation of the cofferdam at the top of the cofferdam at each construction stage during the installation. After installation, the above readings shall be taken every week.

Retained Water Depths Exceeding Three Metres

The cofferdam systems shall be monitored during construction. Readings shall be taken during installation of the cofferdam system at the top, at each restraint point, at the dredge line / bottom of cofferdam, and halfway between the restraint points at each construction stage during the installation of the protection system. After installation, the above readings shall be taken weekly.

Management of Excess Material

Management of excess material shall be according to the Contract Documents.

BASIS OF PAYMENT

Cofferdam - 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.

When the Contract does not contain a separate item for cofferdams, the Contract price for the items directly associated with the cofferdam shall include full compensation for all labour, Equipment, and Material to do the work described in this specification.

PROTECTION SYSTEM – Item No.

Non-Standard Special Provision

Amendment to OPSS 539, November 2014

593.07.02 Removal of Protection Systems

Subsection 539.07.02 of OPSS 539 is deleted in its entirety and replaced with the following:

Protection systems shall be removed from the right-of-way unless it is specified in the Contract Documents that the protection system may be left in place.

Where piles are left in place, the top shall be removed to at least 1.2 m below the finished grade or ground surface.

The method and sequence of removal shall be such that there shall be no damage to the new work, existing work and facility being protected.

All disturbed areas shall be restored to an equivalent or better condition than existing prior to the commencement of construction.

DEWATERING STRUCTURE EXCAVATIONS – Item No.

Special Provision No. FOUN0003

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.

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 5 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 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 100 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.

902.04.02.02 Working Drawings

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

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Subsection 902.07.04 of OPSS 902 is amended by the addition of the following clauses:

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.

NOTICE TO CONTRACTOR - Obstructions

Special Provision

The Contactor is hereby notified that the existing site soils may contain a predominant amount of gravel, cobbles and boulders, as inferred by auger grinding during borehole advancement, which will affect methods of excavation, achieving subgrade tolerances, dewatering methods, and the installation of temporary protection systems or cofferdams. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for excavation, dewatering, subgrade approval, installation and construction of foundations, and for excavation and construction of temporary works as may be required.



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