FINAL REPORT

Foundation Investigation and Design Report

Culvert Replacement (Site No. 21X-0471/C0)
Highway 401, Station 18+349 Cramahe Township, Northumberland County
MTO GWP 4054-17-00, Agreement No. 4016-E-0034-11

Submitted to:

WSP Canada Inc.

100 Commerce Valley Drive West, Thornhill, Ontario L3T 0A1

Submitted by:

WSP Canada Inc.

6925 Century Avenue, Suite 600, Mississauga, ON L5N 7K2

1773612-471

March 8, 2024

GEOCRES No.: 31C04-001
Latitude: 44.053400°
Longitude: -77.824860°

Distribution List

1 e-Copy - MTO Eastern Region

1 e-Copy - MTO Foundations Section

1 e-Copy - WSP Canada Inc.

Table of Contents

REFERENCES

DRAWINGS

PHOTOGRAPHS

TABLES

FIGURES

APPENDICES

APPENDIX A – BOREHOLE RECORDS

APPENDIX B – GEOTECHNICAL LABORATORY TEST RESULTS

APPENDIX C – ANALYTICAL LABORATORY TEST RESULTS

APPENDIX D – GLOBAL STABILITY ANALYSES

APPENDIX E – SPECIAL PROVISIONS

PART A

FOUNDATION INVESTIGATION REPORT
CULVERT REPLACEMENT (SITE NO. 21X-0471/C0)
HIGHWAY 401, STATION 18+349 CRAMAHE TOWNSHIP, NORTHUMBERLAND COUNTY
MTO GWP 4054-17-00, AGREEMENT NO. 4016-E-0034-11

1.0 INTRODUCTION

WSP Canada Inc. (WSP, formerly Golder Associates Ltd., amalgamated with WSP in 2023) is working as part of the WSP Total Project Management team on behalf of the Ministry of Transportation, Ontario (MTO) to support the rehabilitation and widening of Highway 401 from 0.8 km east of Percy Street to 0.4 km west of Christiani Road in Northumberland County, Ontario. The foundations scope of work includes preliminary design services for the replacement of three underpass structures and detail design services for the replacement of four structural culverts.

This report presents the results of the foundation investigation carried out to support the detail design of replacement of Culvert 21X-0471/C0. The foundation investigation services for this project have been delivered under MTO Agreement No. 4016-E-0034 Assignment #11 as part of GWP 4054-17-00.

2.0 SITE DESCRIPTION

The orientation (i.e., north, south, east, west) stated in the text of the report is referenced to project north and, therefore, may differ from magnetic north. For the purpose of this report, Highway 401 is described as oriented in a west-east direction. The existing culvert is oriented in a northwest-southeast direction at an approximately 110° skew to the highway, and the proposed culvert, which is positioned on a new alignment west of the existing structure at a 125° skew to the highway, is described as being oriented in a north-south direction.

The existing Culvert 21X-0471/C0 is located on Highway 401 approximately 0.9 km east of Lake Road and 5 km west of County Road 30, at about Station 18+349 Cramahe Township in Northumberland County. The site location is shown on Drawing 1. The existing culvert consists of a 4.3 m wide by 2.4 m high (interior dimensions) reinforced concrete box structure, which was constructed in 1958. The culvert extends below the Highway 401 westbound lanes (WBL), center median, and eastbound lanes (EBL) over a total length of 60 m. According to the original drawings, the culvert inlet and outlet were designed at Elevation 170.85 m and 170.69 m respectively. Based on MTO's 2019 OSIM report, the culvert is good to fair condition but is close to its 75-year design service life.

At the culvert location, Highway 401 has a four-lane cross-section with paved shoulders separated by a paved median and a concrete tall wall barrier. The highway grade is at approximately Elevation 175 m, and there is approximately 3 m of fill above the top of the existing culvert. The EBL embankment is about 3 m high with the south side slope inclined at about 5 horizontal to 1 vertical (5H:1V). The WBL embankment is about 3 m high with the north side slope inclined at 4H:1V. The existing conditions at the culvert location are shown in Photographs 1 to 3 following the text of this report.

The watercourse flow from south to north at this site. Meadow marsh wetlands are present both upstream and downstream of the culvert, south and north of Highway 401, and Little Lake is present approximately 150 m upstream of the culvert on the south side of Highway 401 (as measured from the existing culvert inlet). The existing natural ground surface at the toe of the Highway 401 embankment is at approximately Elevation 172 m at the culvert site, rising to approximately Elevation 174 m to 176 m; the Little Lake water level is at approximately Elevation 174 m.

Based on our site observations at the time of the field investigation and a review of the available site photographs/satellite images, the existing Highway 401 embankments in the culvert area appear to be performing satisfactorily. There was no visual evidence of instability (i.e., soil movement) on the embankment side slopes, nor tension cracks near the embankment crest that would be indicative of instability.

The new replacement culvert will be on an alternate alignment approximately 17 m west of the existing culvert as measured from centreline to centreline. Based on the General Arrangement drawing, the proposed culvert flows from south to north with inlet and outlet inverts of 170.60 m and 170.36 m, respectively.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between June 30 and July 26, 2022, and included advancing four boreholes (471-22-01 to 471-22-04) in the general location of the proposed culvert alignment. The borehole locations are shown on Drawing 1.

Boreholes 471-22-01 to 471-22-03 were advanced with a CME55 truck-mounted drill rig, and Borehole 471-22-04 was advanced with a LAD Multipower track-mounted drill rig. The drilling equipment was supplied and operated by CCC Geotechnical & Environmental Drilling Ltd. (CCC) of Ottawa, Ontario.

Soil samples were obtained using a 50 mm outer diameter split-spoon sampler in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586[1]). Soil samples were obtained at vertical sampling intervals of about 0.76 m.

A monitoring well was installed in Borehole 471-22-01 to observe the stabilised groundwater level at the site. The monitoring well consists of 52 mm outside diameter PVC tubing with a 1.5 m long slotted screen. The boreholes without a monitoring well were backfilled with bentonite mixed with soil cuttings, in general accordance with the intent of Ontario Regulation (O.Reg.) 903, as amended. The site conditions were restored following completion of the field work.

The field work was supervised on a full-time basis by members of WSP's technical staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers, and transported to WSP's laboratory in Ottawa for further examination and testing. Index and classification tests consisting of water content determinations, grain size distribution analyses, and Atterberg limits testing were carried out on selected soil samples in accordance with MTO and/or ASTM Standards, as applicable.

One soil sample was sent to Eurofins Environmental Testing Canada Inc. (Eurofins) for basic chemical analysis related to potential corrosion of buried steel elements and sulfate attack on buried concrete elements (corrosion and sulphate attack).

The borehole locations and elevations were surveyed by WSP using a Trimble R10 GPS unit referenced to the NAD83 CSRS CBNv6-2010.0 MTM Zone 9 geodetic datum. The Trimble R10 GPS data have a vertical accuracy of approximately 0.1 m and a horizontal accuracy of approximately 0.5 m in accordance with the requirements of MTO's Guideline for Foundation Engineering Services (Version 3.0). The borehole locations, including northing and easting coordinates, ground surface elevations, and drilled depths are summarized below.

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

Borehole No.	NAD83 CSRS CBNv6-2010.0 MTM Zone 9		Ground Surface Elevation (m)	Drilled Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
471-22-01	4880121.3 (44.05340)	198643.5 (-77.824860)	172.5	5.9[1]
471-22-02	4880103.3 (44.053240)	198646.9 (-77.824820)	175.4	10.0
471-22-03	4880070.9 (44.052950)	198648.3 (-77.824790)	174.8	11.7
471-22-04	4880052.9 (44.052790)	198651.2 (-77.824750)	172.3	6.7

Note:
1. Borehole 471-22-01 was terminated due to auger refusal at 5.9 m

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The culvert lies at the boundary of the physiographic regions known as the Iroquois Plain and South Slope, as delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984)[2]. The Iroquois Plain physiographic region extends around the western part of Lake Ontario, from Niagara River to Trent River. The width of the plain varies from a few hundred meters to approximately 13 km north of the current Lake Ontario shoreline, and it extends inland to include a large area in the Trent River valley. The eastern portion of the South Slope in Northumberland County is covered by large drumlins that are generally oriented northeast-to-southwest.

In the area east of Colborne, the surficial glaciolacustrine deposits of the plain consist of sand, gravelly sand, and gravel, as well as nearshore and beach deposits. Non-cohesive (silt, sand and gravel) till deposits are present below the beach/plain deposits, and at ground surface in proximity to drumlins.

Based on geological mapping by the Ministry of Northern Development and Mines (MNDM)[3], the site is underlain by bedrock from the Middle Ordovician era consisting of limestone, dolostone, shale, arkose, and sandstone from the Ottawa Group, Simcoe Group and Show Lake Formation.

4.2 General

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in-situ and laboratory tests are provided on the borehole records presented in Appendix A. The detailed results of the geotechnical laboratory testing on soil samples are presented on the laboratory test figures in Appendix B. The results of the in-situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in Section 4 are uncorrected and are based on use of an automatic hammer. The results of the analytical testing are provided in Appendix C.

---

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

[3] Ministry of Northern Development of Mines. Bedrock Geology of Ontario – Southern Sheet, Ontario Geological Survey - Map 2544.

The borehole locations and the interpreted stratigraphic profile projected along the proposed culvert alignment are provided in Drawing 1.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic section in Drawing 1 are inferred from observations of the drilling progress and noncontinuous soil sampling and therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

4.3 Site Stratigraphy Overview

At the borehole locations, the subsurface conditions encountered at this site consist of surficial fill (topsoil and fill in off-road boreholes or pavement structure and embankment fill in highway boreholes) underlain by non-cohesive glacial till deposits comprising silty sand to sand to gravelly sand to sand and gravel containing cobbles and boulders, up to the termination depth of the boreholes. More detailed descriptions of the major soil layers encountered in the boreholes are provided in the following sections.

4.3.1 Surface Cover/ Surficial Materials

Topsoil with thickness of approximately 50 mm was encountered at the surface of Boreholes 471-22-01 and 471-22-04 which were drilled near the north and south toes of the Highway 401 embankment, respectively. Approximately 0.8 m sand to clayey silt fill containing some silt, gravel, organics, cobbles, and boulders was encountered beneath the topsoil in Borehole 471-22-01.

4.3.2 Existing Pavement Structure

An approximately 200 mm to 300 mm thick layer of asphalt pavement was encountered at the ground surface in Boreholes 471-22-02 and 471-22-03. Approximately 0.3 m of granular material consisting of gravelly sand was encountered beneath the asphalt in both boreholes.

4.3.3 Silty Sand (SM) to Sand (SW) Fill

Underlying the existing pavement structure, non-cohesive fill consisting silty sand to sand containing some gravel to gravelly was encountered in Boreholes 471-22-02 and 471-22-03. This fill was encountered at depths of 0.6 m and 0.5 m below ground surface and was about 2.5 m and 1.5 m thick extending to Elevation 172.4 m and 172.5 m at the borehole locations on the north and south shoulders of Highway 401, respectively.

The SPT ‘N’-values measured within this fill range from 16 blows per 0.3 m of penetration to greater than 100 blows for less than 0.3 m of penetration indicating a compact to very dense state of compactness. Within the fill layers, auger grinding was observed between depths of 0.6 m and 3.1 m in Borehole BH471-22-02, and this is interpreted to represent the presence of gravel, cobbles and/or boulders within the fill; in addition, the high SPT ‘N’-values (70 and 121 blows per 0.1 m penetration and 50 blows per 0.3 m of penetration) are considered to represent the presence of gravel, cobbles and/or boulders and may not represent the state of compactness of the fill matrix.

The water content measured on select samples of the fill ranges between approximately 4% and 5%. The results of grain size distribution carried out on three samples of the silty sand fill are shown on Figure B1 in Appendix B.

4.3.4 Silty Sand (SM) to Sand (SW) Till

Glacial till consisting of silty sand to sand was encountered below fill materials or topsoil in all drilled boreholes. The silty sand to sand till generally contained some gravel and trace to some clay, and the presence of cobbles and boulders was inferred based on auger grinding and difficult drilling, as well as auger refusal in Borehole 471-22-01. This till also contains zones or layers of gravelly sand to sand and gravel, which are discussed in the sub-section below. All four boreholes terminated within the till deposit (inclusive of the gravelly zones) after penetrating it for a thickness of approximately 5.1 m to 9.4 m, extending to a depth of 5.9 m to 11.7 m (to Elevation 166.6 m to 163.1 m).

The SPT 'N' values measured in the till range from 5 to greater than 100 blows per 0.3 m penetration; however, the lower SPT 'N' values of 5 to 8 blows per 0.3 m of penetration are interpreted to have been affected by groundwater disturbance during sampling, and the more typical SPT 'N' values are on the order of 12 blows to greater than 100 blows per 0.3 m of penetration, suggesting a compact to very dense state of compactness. As noted above, frequent auger grinding was observed in both boreholes, indicating the presence of gravel, cobbles and boulder with the glacial till deposit.

The water content measured on samples of the silty sand to sand till range from 5% to 12%. The results of grain size distribution testing carried out on eight samples of this portion of the till are presented on Figure B2.

4.3.5 Gravelly Sand (SP) to Sand and Gravel (SP/SW) Till Zones

Gravelly sand to sand and gravel till layers, at least 0.7 m to 2.1 m in thickness, were encountered within the silty sand to sand till in Borehole 471-22-04 and at the base of the penetrated thickness of the till in Boreholes 471-22-01 and 471-22-03. The gravelly sand to sand and gravel till contained some silt and trace clay. Boreholes 471-22-01 and 471-22-03 were terminated within these zones of the till at 5.9 m depth (Elevation 166.6 m) and 11.7 m depth (Elevation 163.1 m) respectively. Similar gravelly zones of till should be expected between and beyond the borehole locations, and should be expected at varying depths/elevations within the till deposit.

Frequent auger grinding was observed in both boreholes, indicative of the presence of cobbles and boulder with these zones of the glacial till deposit. The SPT 'N' values measured in these layers of the till range from 12 blows per 0.3 m of penetration to greater than 50 blows for less than 0.3 m penetration suggesting a compact to very dense state of compactness.

The water content measured on samples of the gravelly sand to sand and gravel till ranges from 5% to 7%. The results of grain size distribution testing carried out on three samples of this portion of the till are presented on Figure B3.

4.4 Groundwater Conditions

A monitoring well was installed in Borehole 471-22-01 to measure the stabilized groundwater level at the site. The groundwater levels measured in the monitoring well are presented in table below.

Borehole	Screened Interval	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Groundwater Elevation (m)	Date
471-22-01	Silty Sand Till	172.5	0.3	172.2	December 14, 2022

In addition, wet soils were encountered between approximately 1.5 m and 4.0 m depth in the boreholes as noted on the borehole records, corresponding to approximately Elevation 170.8 m to 171.4 m. It is expected that the groundwater levels will be subject to fluctuations both seasonally and as a result of precipitation events.

4.5 Analytical Testing

One soil sample was submitted to Eurofins for chemical testing/analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The test results are provided in Appendix C and are summarized below.

Borehole	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
471-22-03	1.5-2.1	0.016	0.01	0.44	9.24	2,273

5.0 CLOSURE

This Foundation Investigation Report was prepared by Pouya Pishgah, P.Eng., a Principal Geotechnical Engineer with WSP. Ms. Lisa Coyne, P.Eng., Geotechnical Engineering Fellow and MTO Principal Foundations Contact conducted an independent technical and quality review of this report.

WSP Canada Inc.

Pouya Pishgah, P.Eng.
Principal Geotechnical Engineer

Lisa Coyne, P.Eng.
Fellow, MTO Principal Foundations Contact

PP/LCC/al

https://golderassociates.sharepoint.com/sites/11407g/wo11 colborne to brighten/3. reporting/4-culvert 471/3-final/1773612 gwp 4045-17-00 fidr rev0 culvert 471 2024-03-07.docx

PART B

FOUNDATION DESIGN REPORT
CULVERT REPLACEMENT (SITE NO. 21X-0471/C0)
HIGHWAY 401, STATION 18+349 CRAMAHE TOWNSHIP, NORTHUMBERLAND COUNTY
MTO GWP 4054-17-00, AGREEMENT NO. 4016-E-0034-11

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for the detailed design of the replacement of Culvert 21X-0471/C0. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced as part of the current investigation and the design information in the General Arrangement drawing provided by WSP.

The Foundation Design Report (Part B of this report) including the discussion and recommendations are intended for the use of the MTO and their detail designers and shall not be used or relied upon for any other purpose or by any other parties, including the future construction contractor. Contractors undertaking this work 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 to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.2 Project Understanding

It is understood that Highway 401 is to be rehabilitated and widened from the existing four-lane configuration to a proposed interim six-lane configuration and ultimate eight-lane configuration (i.e., interim three lanes then ultimate four lanes in each direction) at the site. The existing grade on Highway 401 will be maintained. This interim configuration will require approximately 4 m to 5 m of embankment widening to the outside for both WBL and EBL, with placement of less than approximately 1.5 m of fill on the existing embankment side slopes. The ultimate configuration will require a further widening of approximately 3 m to 4 m to the outside on both sides of the highway, with placement of up to approximately 1 m of additional fill atop the side slopes associated with the interim grading.

It is further understood that Culvert 21X-0471/C0 is to be replaced due to the age of the existing structure. This culvert is proposed to be replaced on a new alignment approximately 17 m west of the existing culvert (as measured midpoint-to-midpoint along the Highway 401 centreline), with a skew of approximately 125° as dictated by drainage, fluvial and fisheries requirements. The replacement culvert will be approximately 87.7 m long to accommodate the ultimate highway widening. Based on the Highway 401 right-of-way limits and topography, concrete headwalls and retaining walls are required at the north and south ends of the culvert to retain the embankment fill.

As the culvert will be replaced on a new alignment, watercourse flows can be maintained through the existing culvert throughout construction. It is anticipated that the culvert will be replaced via open-cut excavations in two stages, with traffic initially shifted toward the median to permit construction of both ends of the culvert, then traffic shifted to the newly constructed outside portions to permit construction of the section within the median. Temporary protection systems will be required along Highway 401 between the stages.

The existing culvert, a 4.3 m wide by 2.4 m high (interior dimension), 60 m long reinforced concrete box constructed in 1958, can be decommissioned by removal or by abandoning in place via grouting up the culvert.

6.3 Culvert Replacement and Foundation Options

From a geotechnical/foundation perspective, pipe culverts, a closed-bottom box culvert or an open-footing culvert (arch or box) are considered feasible alternatives for this culvert replacement. The culvert types are briefly summarized below and a comparison of advantages, disadvantages and risks is provided in Table 1 following the text of this report.

- Multiple pipe culverts would likely be required to provide a similar flow-through capacity compared to an open-footing or closed-bottom box culvert option. Further, if constructed from steel, pipe culverts will likely have a shorter design life compared to concrete structures.
- A closed-bottom concrete box culvert can be formed of pre-cast segments that can be placed more expeditiously compared to a cast-in-place option, offering schedule advantages with respect to construction/traffic staging and dewatering. Concrete boxes can typically be founded at a shallower level compared to open footing culverts, reducing excavation and dewatering requirements compared to that option. Soil materials can be incorporated above the base slab to create a more natural substrate for fisheries.
- An open footing culvert will typically require deeper foundation excavations as compared to a box culvert. This culvert type is typically cast in place and thus will extend the construction schedule and increase the excavation, dewatering, and shoring requirements compared to a concrete box culvert. There can also be a slightly higher risk of erosion/scour and undermining of foundations along the length of an open footing culvert, compared to a box culvert in which erosion and scour protection is required only at the inlet and outlet.

Based on the above considerations, a closed-bottom concrete box culvert (similar to the existing) is preferred from a geotechnical/foundation perspective. However, other culvert types may be preferred due to construction staging or other considerations, such as fisheries requirements related to natural channel substrate.

Based on the General Arrangement (GA) drawing provided by WSP, it is understood that a precast reinforced concrete box culvert has been selected as the preferred structure replacement type. The culvert will be 4.8 m wide by 2.4 m high (interior dimensions, corresponding to 5.4 m by 3.1 m outside dimensions) based on hydraulic requirements, with the invert varying from approximately Elevation 170.60 m at the south (inlet) end to Elevation 170.36 m at the north (outlet) end. Natural substrate materials will be provided at the stream bed level within the culvert.

6.4 General Foundation Design Context

6.4.1 Consequence and Site Understanding Classification

As the proposed replacement culvert crosses Highway 401, which carries large traffic volumes with the potential to impact alternative transportation corridors, a “typical consequence level” is considered appropriate for this project, as outlined in Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2019) and its Commentary. Further, given the level of foundation investigation and laboratory testing completed to date as presented in Sections 3.0 and 4.0, a “typical degree of site and prediction model understanding” has been utilized. Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, Φ_{gu} and Φ_{gs}, from Tables 6.1 and 6.2 of the CHBDC have been used for design.

For seismic design, the consequence factor, ψ, and resistance factor, ϕ_{gu}, should be taken as unity, as per Section 6.14.4 of CHBDC.

6.4.2 Seismic Design

The seismic hazard values associated with the design earthquakes are those established for the National Building Code of Canada (NBC 2020) by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 6th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2020.

6.4.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the energy-corrected average standard penetration resistance, $\overline{N}_{60}$, below the founding level, the site may be classified as Site Class D in accordance with Clause 4.4.3.2 and Table 4.1 of CHBDC (2019), in the absence of site-specific geophysical testing. Geophysics testing such as Multi-Channel Analysis of Surface Waves (MASW) or vertical seismic profiling may provide a more favourable average shear wave velocity.

6.4.2.2 Spectral Response Values

In accordance with Section 4.4.3.1 of the CHDBC and based on the location of the proposed structure, the Class D peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca) are provided below.

Parameter	2% Probability of Exceedance in 50 Years (2,475-year return period) (g)
PGA	0.204
Sa(0.2)	0.351
Sa(0.5)	0.333
Sa(1.0)	0.199
Sa(2.0)	0.0951
Sa(5.0)	0.0254
Sa(10.0)	0.00795
PGV [m/s]	0.217

6.4.3 Soil Liquefaction

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil which may lead to potentially large surface deformations, and under undrained conditions generate excess pore water pressures that can lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (analogous to slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of slopes often referred to as “flow slides”.

In general, the fill materials and native soils at this culvert site consist of compact to dense silty sand to sand till and gravelly sand to sand and gravel till. Based on the compactness of the soils and the site-specific PGA, the soils at this site are considered to have a low potential for liquefaction during a seismic event.

6.4.4 Frost Protection

The frost penetration depth in this area is approximately 1.4 m as interpreted from Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Penetration Depths for Southern Ontario). However, it is not necessary to ensure that the full length of the replacement culvert is founded below this frost depth for frost protection purposes, as box culverts are tolerant of small magnitudes of movement related to freeze-thaw cycles, should these occur.

6.5 Culvert Foundation Design Recommendations

6.5.1 Culvert Subgrade Preparation

Prior to placing the bedding/levelling course and installing the replacement culvert, it is recommended that all organic material (i.e., topsoil, peat and/or mixed organic soils), existing fill, and any disturbed materials encountered below the culvert footprint be sub-excavated and replaced with Ontario Provincial Standard Specification, Provincial Oriented (OPSS.PROV) 1010 Granular 'A or Granular 'B' Type II fill. Granular 'B' Type II fill (and not clear stone) is recommended for placement in wet conditions.

6.5.2 Box Culvert Bedding and Levelling Layer Requirements

The bedding and levelling pad requirements for a pre-cast box culvert should be in accordance with OPSS.PROV 422 (Pre-cast Reinforced Concrete Box Culverts).

Provided adequate dewatering is in place, a minimum 150 mm thick layer of OPSS.PROV 1010 (Aggregates) Granular A material is recommended for bedding purposes. The bedding should be placed in maximum 200 mm thick loose lifts and be compacted to at least 98% of the material's Standard Proctor Maximum Dry Density (SPMDD) in accordance with OPSS.PROV 501 (Compacting).

In addition, a 75 mm thick uncompacted levelling pad consisting of OPSS.PROV 1010 (Aggregates) Granular 'A' or fine concrete aggregate meeting the grading requirements specified in OPSS.PROV 1002 (Aggregates – Concrete) should be provided with a geometry similar to that provided on OPSD 803.010 (Backfill and Cover for Concrete Culverts).

6.5.3 Box Culvert Founding Elevation and Axial Geotechnical Resistances

Based on a 350 mm thick concrete bottom slab and the recommended bedding and levelling layer thicknesses (minimum 150 mm and 75 mm, respectively), the founding subgrade level for the replacement culvert will be at approximately Elevation 170.03 m at the south (inlet) end and Elevation 169.81 m at the north (outlet) end. For the proposed box culvert within an overall footprint width of 5.4 m (exterior dimension) founded on the properly prepared granular bedding/levelling course overlying the native soils at the above-noted elevations, the following factored geotechnical resistances may be used for design:

- Factored ultimate geotechnical resistance: 500 kPa
- Factored serviceability geotechnical resistance (for 25 mm of settlement): 250 kPa

The factored serviceability geotechnical resistance takes into account the embankment unloading associated with a replacement culvert installed along the proposed new alignment.

6.5.4 Open Footing Culvert Founding Level and Factored Axial Geotechnical Resistances

Strip footings should be placed on the properly prepared native subgrade soils below the frost penetration depth. Based on the invert elevations as summarized in Section 6.3, the footings should be founded at about Elevation 169.0 m to provide a minimum 1.4 m of soil cover for frost protection. If precast footings are utilized, it is recommended that a minimum 150 mm thick bedding layer and 75 mm thick levelling layer (as discussed in Section 6.5.2) be incorporated directly below the underside of the footings to facilitate their placement.

For 1.2 m wide footings founded on the properly prepared native soils at Elevation 169.0 m, the following factored geotechnical resistances may be used for design:

- Factored ultimate geotechnical resistance: 150 kPa
- Factored serviceability geotechnical resistance (for 25 mm of settlement): >150 kPa

The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance. As such, ULS conditions will govern for the open footing design. The factored geotechnical resistances are dependent on the footing width and founding elevation and as such, the geotechnical resistances should be reviewed if the footing width or founding elevations differ from those given above. In addition, these geotechnical resistances are based on loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.10.5 of the CHDBC (2019) and its Commentary.

6.5.5 Retaining Wall Founding Level and Factored Axial Geotechnical Resistances

Retaining walls are required on both sides of the replacement culvert at the upstream and downstream ends to retain the Highway 401 embankment fills within the MTO right-of-way, and separate the fills from the watercourse channel. It is understood the north end of culvert will require retaining walls and a header wall, whereas the south end will require retaining walls and no header wall. It is understood that each of the walls will be approximately 5 m long, with a maximum height on the order of 4 m relative to the ground surface in front of the wall.

Based on the borehole results, the retaining wall footings should be founded at or below the elevations given below; the footings may need to be founded deeper to achieve a minimum depth of 1.4 m below lowest surrounding grade to provide adequate protection against frost penetration. The following factored ultimate and serviceability geotechnical resistances will apply:

Retaining Wall Area	Founding Elevation (m)	Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa)
South (Inlet)	170.6	2	425	250
		3	450	200
North (Outlet)	170.4	2	450	275
		3	475	225

The factored geotechnical resistances provided above for both the box culvert and retaining walls are based on a "typical" consequence level, Ψ = 1.0, and a "typical" degree of site understanding with corresponding geotechnical resistance factors for "Shallow Foundations" of Φ_{gu} = 0.50 for "Bearing" and Φ_{gs} = 0.80 for "Settlement" as per Table 6.2 of CHBDC (2019).

The factored geotechnical resistances provided above are also based on the loading applied perpendicular to the base of the culvert walls; where applicable, inclination of the load should be taken into account in accordance with Section 6.10.2 and Section C6.10.5 of CHBDC (2019) and its Commentary. The factored geotechnical resistances should be reviewed if the founding elevation and/or the foundation width differ from those indicated above.

6.5.6 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance should be calculated in accordance with Section 6.10.4 of CHBDC (2019), applying the appropriate consequence and degree of site understanding factors, as noted above in Section 6.2. The following interface friction angle(s) and interface shear strengths may be utilized to assess the critical conditions for sliding resistance.

Interface	Interface Strength
Between pre-cast concrete and underlying granular bedding/levelling layer	$\delta'_i = 20^o$, $c'_i = 0$ kPa,
Between the granular bedding layer and underlying silty sand to sand till	$\phi' = 32^o$, $c' = 0$ kPa
Between the cast-in-place culvert and/or retaining wall footings and native silty sand to sand till	$\phi' = 32^o$, $c' = 0$ kPa

6.5.7 Culvert Backfill

Backfill above/behind the culvert walls, headwalls and retaining walls should consist of granular fill meeting the specifications for OPSS.PROV 1010 (Aggregates) Granular A or Granular B Type I or II. The backfill should be placed in maximum 300 mm thick loose lifts and compacted to not less than 98% of the material's SPMDD in accordance with OPSS.PROV 501 (Compacting). The fill should also be placed concurrently on both sides of the culvert, ensuring that the backfill depth on one side does not exceed the other side by more than 400 mm as per OPSS.PROV 422 (Precast Reinforced Concrete Box Culverts). Embankment restoration after completion of the culvert replacement should be carried out in accordance with OPSS.PROV 206.

6.5.8 Culvert Erosion and Scour Protection

To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles which could lead to the formation of sinkholes), consideration should be given to the use of concrete cut-off walls, retaining walls and/or a clay seal. Based on the preliminary GA drawing and as noted in Section 6.5.3, it is understood that approximately 5 m long retaining walls and 1.2 m deep concrete cut-off walls are to be constructed at the both the inlet and outlet ends of the replacement culvert.

If a clay seal is included in the design, in lieu of or in addition to cut-off walls and retaining walls, the clay material should meet the requirements of OPSS.PROV 1205 (Clay Seal), and the seal should be a minimum of 1 m thick, whether constructed of natural clay or soil-bentonite mix. Alternatively, a geosynthetic clay liner (GCL) may be incorporated, and this is generally considered the preferred alternative as it is much thinner (only a few millimeters thick) than the standard natural clay or soil-bentonite layer, thus requiring a shallower excavation into the slope, and is much easier to install. The clay seal or GCL should extend a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and from a depth of 1 m below the scour level up to a minimum vertical height on the embankment side slopes equivalent to the high-water level. If a GCL is utilized, the GCL should be constructed within the embankment slope to allow for a minimum 0.3 m thick granular (embankment) fill cover to be placed over the GCL to provide for protection from the requisite overlying erosion protection material. Rip-rap/rock fill slope protection material should be placed on the granular cover layer and not directly on the GCL.

As a minimum, rip-rap treatment for the outlet of the culvert should be consistent with the standard presented in OPSD 810.010 (Rip Rap Treatment). Erosion protection for the inlet of the culvert could also follow the standard presented in OPSD 810.010 (Rip Rap Treatment) similar to the outlet, but with the rip-rap placed up to the toe of slope level, in combination with the cut off measures noted above.

The requirements for, and design of erosion protection measures for the culvert and re-constructed embankment side slopes should be assessed by the Drainage and Hydrology engineers. If additional erosion protection is required, consideration could be given to the use rip-rap, rock protection, or granular sheeting meeting the requirements of OPSS.PROV 1004 (Aggregates – Miscellaneous), placed and constructed in accordance with OPSS.PROV 511 (Rip-Rap, Rock Protection, and Granular Sheeting).

6.6 Lateral Earth Pressures

The lateral earth pressures acting on the headwalls and side walls of the culvert will depend on the type and method of placement of backfill materials, the nature of the soils/embankment fill 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.

The following recommendations are made concerning the design of the replacement culvert and associated headwalls and retaining walls.

- Select, free draining, non-frost susceptible granular fill meeting the requirements of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ (Type I or II) should be used as backfill behind the culvert walls and associated headwalls and retaining walls, as well as on top of the culvert for a minimum thickness of 300 mm in a similar configuration to that shown in OPSD 803.010 (Backfill and Cover for Concrete Culverts).
- 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 2019 CHBDC Section 6.12.3 and Figure 6.8. Hand-operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, the granular fill should be placed in a zone with the width equal to at least 1.4 m behind the back of the wall (see Figure C6.31(a) of the Commentary to CHBDC). For unrestrained walls, the 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 (see Figure C6.31(b) of the Commentary to CHBDC). However, where side slopes inclined at 3H:1V or flatter are required for open-cut excavations extending below the groundwater level, in accordance Ontario Regulation 213, Ontario Occupational Health and Safety Act (OHSA) for Construction Projects (as amended), consideration could be given to backfilling the full open-cut excavation area with OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ (Type I or

II) in order to satisfy both the backfilling requirements outlined in the Commentary to the CHBDC and the open-cut excavation requirements outlined in the OHSA.

The following parameters and lateral earth pressure coefficients may be used in the design of culvert walls, headwalls and retaining walls:

The lateral earth pressure coefficients provided in the table below have been developed for flat (i.e., non-sloping) ground above/behind the culvert walls, as well as for a 2H:1V slope condition for unrestrained walls as applicable for the retaining walls at the ends of the replacement culvert. If the inclination of the slope above the wall differs, revised lateral earth pressures parameters will need to be calculated in accordance with CHBDC Clause C6.12.1, Figures C6.28 (active earth pressure) and C6.29 (passive earth pressure), and Clause C6.12.2.2 (at-rest earth pressure).

If the wall does not allow lateral yielding (i.e., a 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 wall allows lateral yielding (i.e., unrestrained structure), active earth pressures should 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 C6.12 of the Commentary to CHBDC (2019).

Wall Movement Condition	Restrained Wall	Unrestrained Wall			
Fill Material	Existing Embankment Fill Behind Granular, Φ'=32°	Granular A and B Type II Φ'=36°		Granular B Type I Φ'=32°	
Unit Weight (kN/m3)	19	22	22	21	21
Ground Surface Above Top of Wall	Horizontal	Horizontal	2H:1V	Horizontal	2H:1V
Active Earth Pressure (K_a)	-	0.26	0.36	0.31	0.46
At-Rest Earth Pressure (K_o)	0.47	-	-	-	-
Passive Earth Pressure (K_p) [1]	3.25	3.85	-	3.25	-

Note:

1. The total passive resistance may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.27 of the CHBDC (2019) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

6.7 Embankment Widening, Stability, and Settlement

6.7.1 Embankment Subgrade Preparation and Construction

Prior to construction of the embankment widening, it is recommended that all topsoil/peat and loose or disturbed soil be removed from the widening footprint.

Fill for construction of the widened embankments may consist of Granular A or Granular B Type I or Type II meeting the specifications of OPSS.PROV 1010 (Aggregates), or alternatively earth fill or select subgrade material (SSM). Fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading).

Where earth fill or select subgrade material is used for embankment construction, the exposed materials will be susceptible to erosion and shallow ravelling. To reduce surface water erosion and ravelling on the embankment side slopes or cut slopes, treatment per OPSS.PROV 804 (Temporary Erosion Control) and OPSS.PROV 803 (Vegetative Cover) must be provided. If slope protection is not in place prior to winter or periods of excessive precipitation, alternate protection measures such as gravel sheeting per OPSS 511 (Rip-Rap, Rock Protection and Granular Sheeting) and OPSS.PROV 1004 (Aggregates – Miscellaneous) will be required to reduce the potential for erosion and associated requires for remedial works on the slope faces prior to topsoil dressing and seeding.

6.7.2 Global Stability of Widened Embankment Including Retaining Walls

Based on site observations at the time of the foundation investigation and available site photographs/satellite images, the existing highway embankments in the culvert area appear to be performing satisfactorily. There was no evidence of instability or settlement (i.e., soil movement) of the existing embankment side slopes.

The existing embankments are up to approximately 4 m to 4.5 m in height relative to the surrounding ground surface. Based on the GA drawing, it is understood that the existing embankment heights at the replacement culvert location will generally be maintained (i.e., no significant grade raise). We further understand the existing embankment side slopes will generally be maintained (or slightly flattened on the north side) following the proposed embankment widening.

The global stability of the proposed Highway 401 embankments side slopes including retaining walls at the ends of the culvert was evaluated using limit equilibrium analysis with GeoStudio 2023.1.0 Slope/W software. The geometry used in the stability analysis was based on the topographic survey for the site, the soil stratigraphy encountered at the site as outlined in Section 4.0 and information provided on the General Arrangement drawing.

For the stability analyses, and in the context of the CHBDC (2019), the target Factor of Safety (FoS) is defined as being equal to the inverse of the product of the consequence factor, Ψ and the geotechnical resistance factor, ϕ_{gu}, (i.e., FoS = 1 / (Ψ * ϕ_{gu})). Accordingly, for a ‘typical’ consequence level and a ‘typical’ degree of site and prediction model understanding, a target minimum FoS of 1.33 and 1.54 has been used for the design of the widened embankment and retaining walls, considering global stability for temporary (short-term) and permanent (long-term) conditions, respectively, per Table 6.2 of CHBDC (2019).

The proposed embankment widening was analyzed under drained (long-term) and seismic design conditions using the following assumptions, and with soil parameters as shown on the stability analysis figures in Appendix D:

- The soil stratigraphy was based on Profile A-A’ shown in Drawing 1 following the text of this report, with the footing founding level per Section 6.5.5 of this report.
- The groundwater level was assumed to be at Elevation 172 m, at or near the ground surface beyond the embankment footprint.
- A seismic horizontal loading of 0.101g, equal to one-half of the site-specific PGA value (0.5 of 0.203 g Site Class C) was used for seismic analysis (see Section 6.4.2 of this report).

- The retaining walls were assumed to be up to about 4.5 m in total height, and a footing width of approximately 3.5 m has been assumed in this global stability analysis.

The results of the long-term/effective stress stability analysis indicate that the embankment widening, including the retaining walls at the culvert ends, has a factor of safety of greater than 1.5 for a deep-seated slip surface that could affect the stability of the highway embankment and/or the retaining wall. Under the design of earthquake loading, the approach embankments have a factor of safety of greater than 1.1. The results of the stability analyses are provided in Figures D1 and D2 in Appendix D. If the wall geometry changes significantly in the future detail design, the global stability of the embankment/retaining wall system should be rechecked by the detail design team.

6.7.3 Embankment Settlement

6.7.3.1 Methods and Parameters

To accommodate the ultimate eight-lane configuration, an approximately 6 m and 12 m widening is proposed along the outside of the WBL and EBL embankments, respectively, resulting in grade raises of about 2.2 m and 1.8 m at the north and south toes of slope, respectively.

To estimate the magnitude of the settlement as a result of the proposed embankment widening, analyses were carried out both the north and south toes of slope, where the highest grade raise is anticipated to occur. The settlement analyses were carried out using the commercially available computer program Settle3 (Version 5.012) from Rocscience Inc., as well as hand calculations. The settlement analysis discussed below assumes that all organics within the footprint of the widened embankments will be sub-excavated and replaced with granular fill prior to placement of any new embankment fill material for the widening.

The immediate compression of the native soil deposits was modelled based on typically accepted correlations with the obtained SPT ‘N’ values as presented in Bowles (1984) and by Kulhawy and Mayne (1990) together with engineering judgment based on experience in similar subsurface conditions. The unit weight and associated stiffness (moduli) are summarized in the table below. Additional details related to the selected moduli values, based on correlated SPT “N” values, are presented on Figure 1. The groundwater level was assumed to be at approximately Elevation 172.5 m (i.e., roughly at ground surface beyond the embankment toe) to account for higher groundwater levels than have been measured to date in the standpipe piezometer installed in Borehole 471-22-01.

Material	Unit Weight (kN/m3)	Elastic Modulus (MPa)
Silty Sand to Sand Till (generally compact to dense)	21	40 to 80

6.7.3.2 Results of Analyses

The total and differential settlement of the existing site soils under the loading imposed by the widened approach embankments is presented below.

Location	Existing Centerline	Existing Shoulder	Widened Shoulder	Differential Settlement (2)
Westbound Lane				
Existing Culvert (1)	<10 mm	~ 15 mm	~10 mm	~ 450:1
Replacement Culvert (2)	< 10 mm	~ 10 mm	~15 mm	~ 570:1
Eastbound Lane				
Existing Culvert (1)	< 10 mm	~ 15 mm	~10 mm	~ 550:1
Replacement Culvert (2)	~ 5 mm	~ 5 mm	~ 10 mm	~ 700:1

Notes:

1. Settlement values for the existing culvert at the "widened shoulder" reflect the anticipated settlements at the existing culvert ends (i.e., near the existing toe of slope).
2. The differential settlement has been calculated over a length of approximately 6.75 m (i.e., one lane at 3.75 m plus a 3 m shoulder) from the existing WBL/EBL centreline to the existing edge of shoulder calculated over a length of approximately 14.25 m (i.e., three lanes at 3.75 m each plus a 3 m shoulder) extending from the current WBL/EBL centreline to the new shoulder following embankment widening.

The above-noted magnitudes of settlement are expected to be elastic and to occur during and immediately following construction of the embankment widening, with no long-term settlements anticipated.

The above preliminary estimates do not include compression of the fill itself, which would occur during construction of the embankment depending on the type of material used. The magnitude of granular fill compression may range from 0.5% to 1% of the height of the embankment, assuming approximately 98% compaction of the embankment fill is achieved during construction, relative to the material's standard Proctor maximum dry density (SPMDD). In this case, settlement of the granular fill itself is expected to occur essentially during embankment construction. Non-granular earth fill materials are not recommended for embankment construction as they may exhibit some additional settlement over time depending on their gradation, plasticity, and field compaction effort. Although not anticipated, should rock fill be considered, long term settlement of the rock fill would need to be considered.

6.7.3.3 Comparison to MTO's Settlement Criteria

Based on MTO's Embankment Settlement Criteria for Design (MTO, July 2010), the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments at this site.

Location	Maximum Limits During Pavement Design Life	
	Total	Differential
Longitudinal Transitions (Freeways)	25 mm (0 to 20 m from structure)	n/a
Widened Embankments (Freeways)	50 mm	200:1

Based on the results of the analyses, the estimated settlements meet MTO's settlement criteria and no settlement mitigation will be required for the existing culvert or proposed replacement culvert structure.

6.8 Analytical Testing for Construction Materials

The results of analytical tests on one sample of native silty sand recovered in Borehole 471-22-03 is summarized in Section 4.4 and included in Appendix C. The potential for sulphate attack and corrosion are discussed in the following paragraphs; however, it is ultimately up to the designer to determine the appropriate construction materials, including the exposure class, and ensuring that all aspects of CSA A23.1-14 (2014) Section 4.1.1 “Durability Requirements” are followed when designing concrete elements, as applicable.

6.8.1 Potential for Sulphate Attack

The analytical test result was compared to Table 3 of CSA A23.1-09 Concrete Materials and Methods of Construction for the potential sulphate attack on concrete. The water soluble-sulphate concentration measured in the soil sample is 0.01%, which is considered Negligible according to Table 7.2 in the MTO Gravity Pipe Design Guidelines (2014). Given that the culvert location will be exposed to de-icing salts, it is recommended that a C-1 (reinforced concrete) or C-2 (non-structurally reinforced concrete) class exposure concrete be considered, as appropriate.

6.8.2 Potential for Corrosion

The soil has a pH of 9.24 and according to the MTO Gravity Pipe Design Guidelines (2014), pH values of 8.5 and greater are strongly alkaline. The exact role pH plays in corrosion is inconclusive; however, in general, a pH reading that is strongly acidic is indicative of an increased potential for corrosion. The measured resistivity, R, of 2,273 ohm-cm indicates that the soil corrosiveness is moderate (2,000<R<4,500), as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014).

6.9 Construction Considerations

6.9.1 Construction Staging and Temporary Roadway Protection

The temporary excavations for the culvert replacement will extend through the existing granular embankment fill and into the native subgrade soils. The granular fill and native soils at this site are considered to be Type 3 soil above the groundwater table and Type 4 soil below the groundwater table. Temporary open-cut excavations in Type 3 soils should remain stable if side slopes are excavated no steeper than 1H:1V. In Type 4 soils, the side slopes should be excavated no steeper than 3H:1V. All excavations must be carried out in accordance with Ontario Regulation 213, Ontario Occupational Health and Safety Act for Construction Projects (as amended).

Based on the GA drawing provided by WSP, it is understood that the existing embankments will be temporarily widened to a six-lane configuration to facilitate a stage construction approach for the culvert replacement. If temporary roadway protection systems are required, the protection systems could consist of either driven sheet piling or soldier piles and lagging where H-piles would be driven to a suitable depth, with horizontal lagging installed as the excavation proceed. Support to the system could be in the form of struts, wales, rakers, or anchors. Based on the encountered subgrade soil conditions and anticipated excavation requirements, a sheet pile shoring system would be considered more practical and more cost effective. The installation of sheet piles could potentially be impeded by the presence of cobbles and/or boulder obstructions but the risks are anticipated to be relatively low.

Although the Contractor is responsible for the selection and detail design of the temporary protection/dewatering systems, the following parameters are provided to enable the detail designers, to develop a conceptual design and asses the approximate construction costs for the project system, if adopted at this site.

Soil Type	Bulk Unit Weight, γ (kN/m³)	Internal Angle of Friction ϕ (degrees)	Lateral Earth Pressure Coefficients (1)		
			Active, K_a	At Rest, K_o	Passive, K_p (2)
New Granular A or B Type I or II Fill (compacted)	22	35	0.27	0.43	3.69
Existing Granular Fill (loose to dense)	20	32	0.31	0.47	3.25
Silty Sand to Sand Till (compact to dense)	21	34	0.28	0.44	3.54

Notes:

(1) The lateral earth pressure coefficients presented above are based on a horizontal surface behind the excavation. If sloped surfaces are present, the coefficients should be corrected accordingly.

(2) The total passive resistance below the base of the excavation adjacent to the temporary protection system may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.27 of the CHBDC (2019) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

Temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the temporary protection systems should meet Performance Level 2 as specified in OPSS.PROV 539. Design of the temporary support system should include an evaluation of base stability and hydraulic uplift stability, as defined in the Canadian Foundation Engineering Manual (CFEM 2006).

6.9.2 Control of Groundwater and Surface Water

This culvert is being replaced on a new alignment, and it is anticipated that the creek flow could be maintained within the existing culvert while the replacement culvert is being constructed; however, culvert construction staging, timing for decommissioning of the existing culvert and maintenance of flow will need to be confirmed as part of detail design.

Given the permeable subgrade soils encountered at this site and the anticipated depth of the excavations (i.e., extending about 2 m below the measured groundwater level), a temporary dewatering system potentially in conjunction with a cofferdam/cut-off system is anticipated to be required to maintain a dry and stable subgrade.

Depending on the depth of the cofferdam installations, it may be possible to dewater the excavation using properly filtered pumps/sumps; however, given the permeable nature of the silty sand to sand deposit at this site, an active dewatering system may be required. Where required, active dewatering methods should draw down the groundwater level to approximately 1 m below the base of the excavation to maintain the integrity of the foundation subgrade. The extent/depth of dewatering requirements shall be reviewed by the contractor, based on their proposed construction methods/ procedures.

An Environmental Activity Section Registry (EASR) is not required for the temporary surface water diversion through an existing culvert. However, if active dewatering is required, an EASR (for pumping volumes greater than 50 m^3/day) or PTTW (for pumping volumes greater than 400 m^3/day) may be required, depending on the groundwater conditions at the time of construction and estimated pumping volumes. The Contractor should be required to evaluate the estimated seepage and groundwater removal quantity, based on their proposed construction methods/procedures and the groundwater conditions at the time of construction, to make the final assessment/determination whether an EASR (or PTTW) is ultimately required.

Dewatering of all excavations should be carried out in accordance with OPSS.PROV 517 (Dewatering), as modified by SP 517F01, a copy of which is included in Appendix E. Given the cohesionless subgrade conditions encountered at this site, as well as the absence of any settlement-sensitive infrastructure in the vicinity of the culvert, the risk of settlement impacts is considered to be low from a foundation perspective provided the pumping is carried out from properly filtered sumps/well points. As such, the geotechnical/foundation fill-in SP 517F01 should indicate that a preconstruction survey is not applicable. Any temporary flow bypass requirements should be assessed and confirmed by drainage engineers during the future detail design for inclusion in SP 517F01.

6.9.3 Subgrade Preparation

Prior to placing the levelling pad/bedding layer material and/or precast culvert, all existing fill, organic materials (including topsoil, peat, and/or mixed organic soil), and any disturbed/loosened native soils should be sub-excavated from below the plan limits of the proposed works to expose the undisturbed native subgrade soil within the plan limits of the culvert. The subgrade should be inspected to ensure that all organics and other unsuitable materials have been removed, in accordance with OPSS.PROV 422 (Precast Reinforced Concrete Box Culverts) and/or OPSS.PROV 902 (Excavating and Backfilling – Structures).

Following inspection, the sub-excavated area should be backfilled with granular material meeting the requirements of an OPSS.PROV 1010 Granular ‘A’ or Granular ‘B’ Type II that is placed and compacted in accordance with OPSS.PROV 501 (Compacting), as amended by SSP 105S22. The use of Granular ‘B’ Type II fill (and not clear stone) is recommended in wet conditions or below water.

6.9.4 Obstructions

The contractor should be alerted to the potential presence of cobble and boulder obstructions within the fill material at this site, as inferred to be present based on instances of split-spoon refusal in Boreholes 471-22-01 and 471-22-02. Silty sand to sand till was encountered below fill materials in all drilled boreholes, and glacial tills inherently contain cobbles and boulders. The extent and depth of the obstructions may vary beyond and between the borehole locations. A sample Notice to Contractor is included in Appendix D.

7.0 CLOSURE

This Foundation Design Report was prepared by Pouya Pishgah, P.Eng., a Principal Geotechnical Engineer with WSP. Lisa Coyne, P.Eng., a Geotechnical Engineering Fellow and MTO Principal Foundations Contact with WSP Golder, conducted an independent technical and quality review of this report.

WSP Canada Inc.

Pouya Pishgah, P.Eng.
Principal Geotechnical Engineer

Lisa Coyne, P.Eng.
Fellow, MTO Principal Foundations Contact

PP/LCC/al

https://golderassociates.sharepoint.com/sites/11407g/wo11 colborne to brighten/3. reporting/4-culvert 471/3-final/1773612 gwp 4045-17-00 fidr rev0 culvert 471 2024-03-07.docx

REFERENCES

Bowles, Joseph, E., 1997. Foundation Analysis and Design, Fifth Edition. McGraw-Hill International Editions, Civil Engineering Series, Singapore.

Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition

Canadian Standards Association (CSA), 2019. Canadian Highway Bridge Design Code and Commentary on CSA S6:19.

Canadian Standards Association (CSA), 2014. CSA A23.1-09 Concrete Materials and Methods of Construction (R2014).

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

Kulhawy, F.H. and Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design. Electric Power Research Institute EL-6800s.

Ministry of Northern Development of Mines. Bedrock Geology of Ontario – Southern Sheet, Ontario Geological Survey – Map 2544.

Ministry of Transportation, MTO Gravity Pipe Design Guidelines, MTO Drainage and Hydrology Design and Contract Standards Office, May 2014

Occupational Health and Safety Act and Regulation for Construction Projects (as amended)

ASTM International

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

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 422	Construction Specification for Installation of Precast Reinforced Concrete Box Culverts with Span 3m or Less in Open Cut
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.MUNI 802	Construction Specification for Topsoil
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 1002	Material Specification for Aggregates – Concrete
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS.PROV 1205	Material Specification for Clay Seal
OPSS.PROV 1860	Material Specification for Geotextiles

Ontario Provincial Standard Drawings (OPSD)

OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0 m
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario

Ontario Water Resource Act

Regulation 903	Wells (as amended)

Table 1: Comparison of Alternative Culvert and Foundation Types

Option	Advantages	Disadvantages	Risks/Consequences
Precast Box Culvert	▪ Minimizes depth of excavation, protection systems (if required), and dewatering requirements compared to open-footing option. ▪ Allows faster construction resulting in shorter duration for dewatering and surface water pumping. ▪ More tolerant of total and differential settlements. ▪ A portion of the backfill/bedding under the culvert could be placed in-the-wet (i.e., Granular 'B' Type II) potentially reducing unwatering requirements. ▪ Allows for greater flow volume than circular/arch CSP.	▪ May not satisfy fisheries requirements related to natural channel substrate, if applicable. ▪ Cut-off wall (or clay seal) likely required at inlet to mitigate potential scour under the culvert. ▪ Transportation to site, and on-site lifting of large precast sections will be required.	▪ Lower risk of disturbance of the native subgrade soils during construction; can be mitigated with the use of a granular working pad/bedding layer or concrete working slab. ▪ Low risk related to settlement performance as box segments can accommodate some total and differential settlements.
Open Footing Culvert	▪ May be feasible to construct the culvert on precast footing sections to accelerate construction schedule and reduce time for dewatering/unwatering (pumping). ▪ Readily suitable for construction using concrete or metal sections. ▪ Would likely satisfy fisheries requirements related to natural channel substrate, if applicable. ▪ Allows for greater flow volume than circular/arch CSP.	▪ Excavation depths are greater than for a box culvert option, resulting in increased excavation support, cofferdam and dewatering requirements, and additional spoil material to be disposed off-site. ▪ Constructing footings in the dry will take longer, due to requirements for installation of a groundwater and surface water control system, dewatering and surface water pumping, and excavation in a confined space. ▪ Less tolerant of total and differential settlements, especially if the highway embankment is raised or widened at the culvert site.	▪ Higher risk of disturbance of the native subgrade soils during construction; can be mitigated with use of a granular working pad/bedding layer or concrete working slab. ▪ May require greater depth of dewatering for footing construction. ▪ Culvert joints may be required to accommodate the anticipated total and differential settlement.

Option	Advantages	Disadvantages	Risks/Consequences
Pipe Culvert(s)	▪ Allows for faster construction resulting in shorter duration for unwatering and surface pumping compared to an open-footing and box culverts. ▪ More tolerant of total and differential settlement. ▪ Backfill under the culvert may be placed in-the-wet (i.e., Granular ‘B’ Type II) potentially reducing unwatering requirements.	▪ Reduced flow-through capacity compared to box culvert and open-footing options with a similar span – additional flow through capacity may have to be provided by multiple pipes. ▪ Cut-off wall or clay seal may be required at inlet to mitigate potential scour under the culvert(s). ▪ Difficult to compact backfill materials to level of culvert springline if not done in the dry. ▪ CSP does not have as long of a design life compared to concrete options.	▪ Lower risk of disturbance of the native subgrade soils during construction; can be mitigated with the use of a granular working pad/bedding layer or concrete working slab. ▪ Lower risk related to anticipated total and differential settlement compared to box or open-footing option.

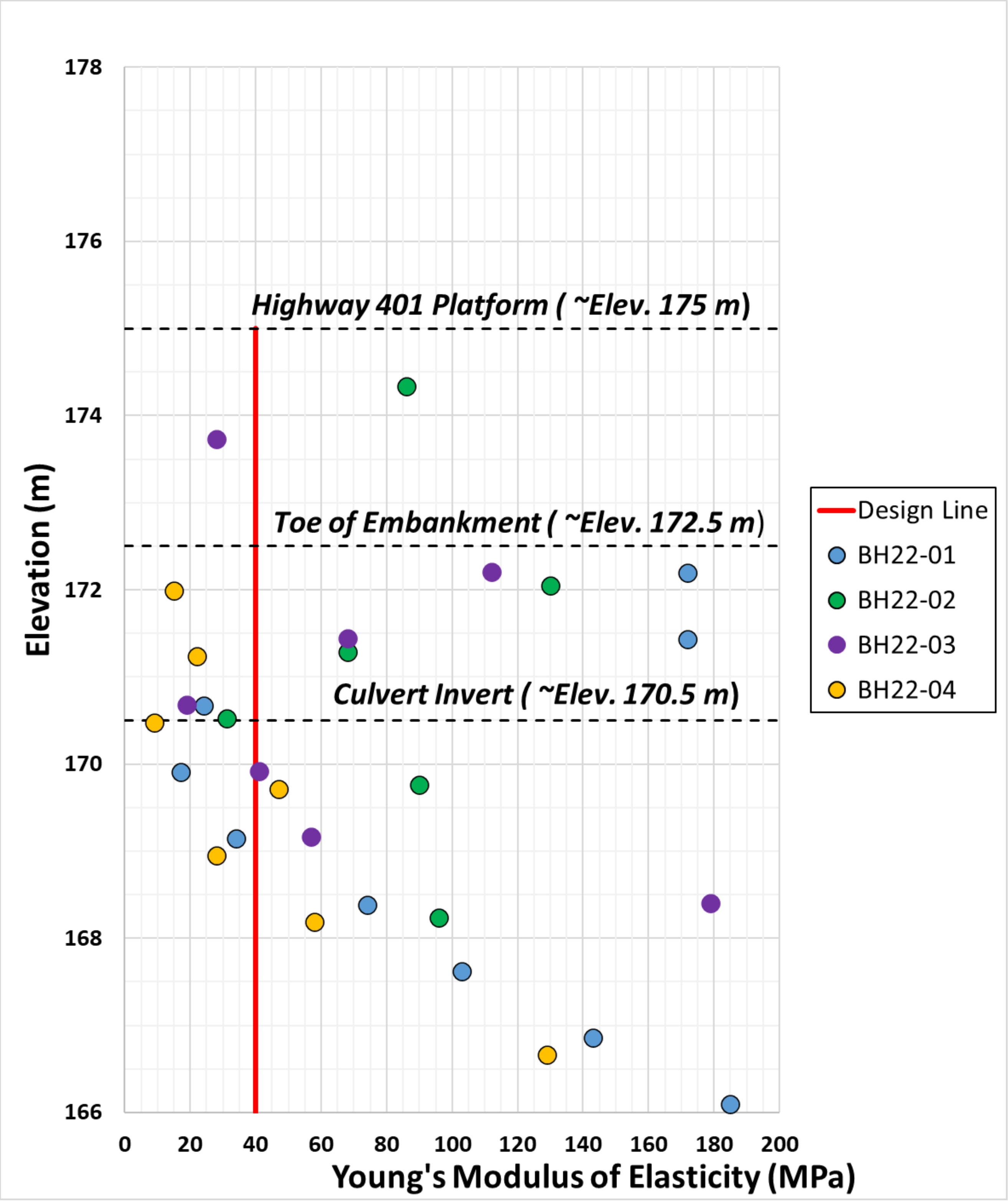
Photograph 1: Looking north towards Borehole 471-22-01

Photograph 2: Looking east along Highway 401 from north shoulder of westbound lanes; Borehole 471-22-02 in foreground

Photograph 3: Looking east along Highway 401 from south shoulder; Borehole 471-22-03 in foreground, and existing culvert outlet visible

Settlement Parameters - Modulus of Elasticity

Figure 1



Date: April 2023
Project No: 1773612 WO#11

WSP GOLDER

Analysis By: DAM
Reviewed By: LCC

APPENDIX A

Borehole Records

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS
MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS[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 cm^2 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, Gs)
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)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness[1]

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

1. 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 grainsize. 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.
2. 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.

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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_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

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

(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
$C_{\alpha(e)}$	secondary compression index
C_α	rate of secondary compression
$C_{\alpha(\varepsilon)}$	modified secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

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

RECORD OF BOREHOLE No 471-22-01

SHEET 1 OF 1 **METRIC**

PROJECT 1773612
G.W.P. 4054-17-00
LOCATION N 4880121.3; E 198643.5 MTM NAD ZONE 9 (LAT. 44.053400; LONG. -77.824860)
ORIGINATED BY JS
DIST Eastern HWY 401
BOREHOLE TYPE LAD Multipower Track Mounted, 83 mm ID Hollow Stem Augers
COMPILED BY ZS
DATUM GEODETIC
DATE June 30, 2022
CHECKED BY PP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
172.5	GROUND SURFACE						
0.0	TOPSOIL - SAND to CLAYEY SILT (SM-ML), some silt, some gravel, contains organics, cobbles and boulders (FILL) Brown to black/grey Moist to w<PL	1	SS	53/0.13	172		
171.7		2	SS	8/0.13			
0.8	SILTY SAND (SM), some gravel to gravelly, contains cobbles and boulders (TILL) Loose to very dense Grey-brown Moist to wet	3	SS	14		o	17 57 (26)
		4	SS	7	171		
		5	SS	15	170	o	28 42 23 7
		6	SS	28	169		
		7	SS	41	168		
167.3		8	SS	59		o	30 43 (27)
5.2	SAND and GRAVEL (SP/SW), some silt, trace clay, contains cobbles and boulders (TILL) Very dense Brown-grey to grey Wet	9	SS	71	167		
166.6 / 5.9	END OF BOREHOLE AUGER REFUSAL						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³.

NOTES:

1. Borehole was terminated due to auger refusal at 5.9 m.

2. Water level measured in standpipe piezometer:

Date	Depth(m)	Elev.(m)
Dec.14/22	0.3	172.2
May.16/23	0.4	172.1

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

Foundation Design

RECORD OF BOREHOLE No 471-22-02

SHEET 1 OF 1 — **METRIC**

PROJECT 1773612
G.W.P. 4054-17-00
DIST Eastern HWY 401
DATUM GEODETIC

LOCATION N 4880103.3; E 198646.9 MTM NAD ZONE 9 (LAT. 44.053240; LONG. -77.824820)
BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID Hollow Stem Augers
DATE July 25, 2022

ORIGINATED BY JS
COMPILED BY ZS
CHECKED BY PP

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT	GR	SA	SI	CL
175.4	GROUND SURFACE									
175.2 / 0.0	ASPHALTIC CONCRETE									
0.3	Gravelly SAND (SW) (FILL), Brown, Moist	1	AS	-	175					
174.8 / 0.6	SILTY SAND (SM), some gravel to gravelly, contains cobbles and boulders (FILL), Very dense, Brown, Moist	2	SS	50		○	17	48	(35)	
		3	SS	70/0.10	174					
		4	SS	121/0.13	173	○	25	50	(25)	
172.4 / 3.1	SILTY SAND (SM), some gravel to gravelly, trace to some clay, contains cobbles and boulders (TILL), Very dense to compact, Grey to brown, slightly mottled, Moist, becoming wet at 4.0 m	5	SS	67	172					
		6	SS	30	171	○	22	44	(34)	
		7	SS	14						
	- Dense to very dense below to 5.2 m	8	SS	42	170					
		9	SS	50/0.13	169					
		10	SS	42	168	○	24	40	(36)	
		11	SS	107/0.10						
		12	SS	100/0.05	167					
		13	SS	100/0.05	166					
165.4 / 10.0	End of Borehole	14	SS	100/0.13						

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+^{3}, ×^{3}: Numbers refer to Sensitivity   ○ $^{3\%}$ STRAIN AT FAILURE

Foundation Design

RECORD OF BOREHOLE No 471-22-03 SHEET 1 OF 2 METRIC

PROJECT 1773612

G.W.P. 4054-17-00 | LOCATION N 4880070.9; E 198648.3 MTM NAD ZONE 9 (LAT. 44.052950; LONG. -77.824790) | ORIGINATED BY JS

DIST Eastern HWY 401 | BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID Hollow Stem Augers | COMPILED BY ZS

DATUM GEODETIC | DATE July 26, 2022 | CHECKED BY PP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
174.8	GROUND SURFACE									
0.0	ASPHALTIC CONCRETE									
0.2 / 174.3 0.5	Gravelly SAND (SW), asphalt present (FILL) Brown to black Moist	1	SS	-						
173.6 1.2	SAND (SW), some gravel (FILL) Compact Brown Moist	2	SS	16	174					
172.5 2.3	SAND to SILTY SAND (SW/SM), some gravel (FILL) Compact to very dense Brown to grey-brown Moist	3	SS	88	173	○	19	53	(28)	
169.6 5.2	SILTY SAND (SM), some clay, some gravel, contains cobbles and boulders (TILL) Very dense to loose Brown to grey-brown Moist, becoming wet at approximately 3.8 m depth.	4	SS	54	172					
		5	SS	34		○	13	43	(44)	
		6	SS	8*	171					
		7	SS	18	170					
167.6 7.2	SAND to SILTY SAND (SW/SM), some clay, some gravel, contains cobbles and boulders (TILL) Compact to very dense Grey-brown Moist	8	SS	26	169					
		9	SS	75	168					
166.3 8.5	SAND (SW), some silt, some gravel, trace clay, contains cobbles and boulders (TILL) Very dense Grey Wet	10	SS	96	167					
164.1	SILTY SAND (SW), some clay, some gravel, contains cobbles and boulders (TILL) Very dense Grey Moist	11	SS	50/0.08	166					
		12	SS	50/0.05	165	○	7	49	(44)	
10.9 / 163.1 11.7	SAND to SILTY SAND, some clay, some gravel, contains cobbles and boulders (TILL) Very dense Grey Moist	13	SS	50/0.05	164					
		14	SS	50/0.08		○	40	41	(19)	

Continued Next Page

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL; GROUND WATER CONDITIONS; STRAT PLOT.

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

RECORD OF BOREHOLE No 471-22-03 SHEET 2 OF 2 METRIC

PROJECT 1773612
G.W.P. 4054-17-00
DIST Eastern HWY 401
DATUM GEODETIC
LOCATION N 4880070.9; E 198648.3 MTM NAD ZONE 9 (LAT. 44.052950; LONG. -77.824790)
BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID Hollow Stem Augers
DATE July 26, 2022
ORIGINATED BY JS
COMPILED BY ZS
CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND and GRAVEL (SW/GW), some silt, contains cobbles and boulders (TILL) Very dense Grey Moist										
	END OF BOREHOLE NOTES: 1. * SPT "N" value may be affected by disturbance due to groundwater inflow to borehole										

$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 471-22-04 SHEET 1 OF 1 METRIC

PROJECT 1773612
G.W.P. 4054-17-00 LOCATION N 4880052.9; E 198651.2 MTM NAD ZONE 9 (LAT. 44.055130; LONG. -77.817650) ORIGINATED BY JS
DIST Eastern HWY 401 BOREHOLE TYPE LAD Multipower Track Mounted, 83 mm ID Hollow Stem Augers COMPILED BY BW
DATUM GEODETIC DATE July 4, 2022 CHECKED BY PP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI CL
172.3	GROUND SURFACE								
0.0	TOPSOIL - SILTY SAND (SM), contains organics, Loose, Brown, Moist	1	SS	9	172				
0.1	SILTY SAND to SAND (SM/SW), some gravel to gravelly, trace clay, contains cobbles and boulders (TILL), Loose to compact, Grey-brown, Moist, becoming wet at approximately 1.5 m depth	2	SS	13	171	o	27	45	(28)
		3	SS	5*	170				
		4	SS	19					
169.3 / 3.1	Gravelly SAND (SW), some silt, trace clay, contains cobbles and boulders (TILL), Compact to very dense, Grey, Wet to moist	5	SS	12	169	o	31	44	(25)
		6	SS	22	168				
		7	SS	82/0.10		o	32	45	(23)
167.1 / 5.2	SAND (SW), some silt, some grave, trace clay, contains cobbles and boulders (TILL), Very dense, Grey, Moist to wet	8	SS	53	167				
		9	SS	94	166				
165.6 / 6.7	END OF BOREHOLE								

Column headings also include: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

NOTES:

1. * SPT "N" value may be affected by disturbance due to groundwater inflow to borehole

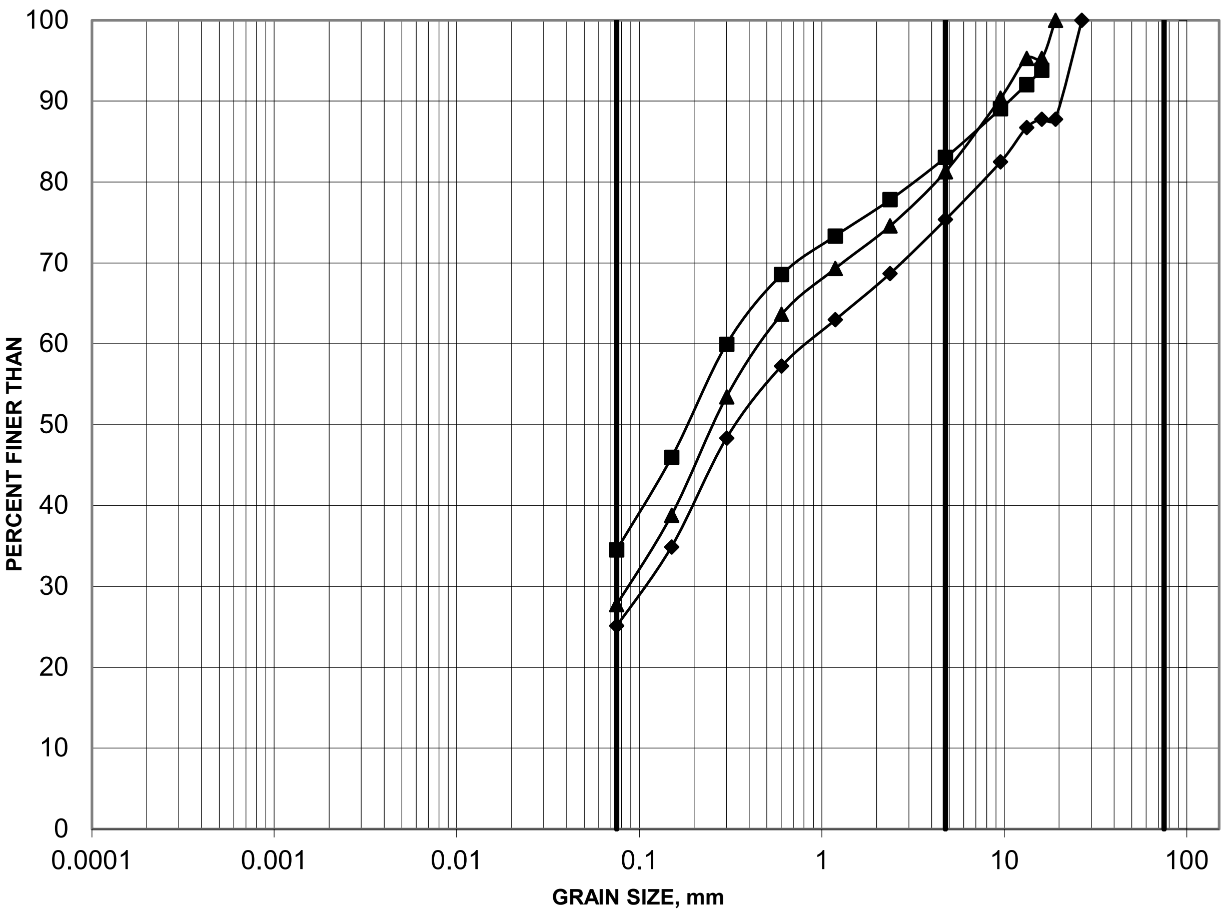
+3, ×3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

SILTY SAND FILL



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

Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
■ 471-22-02	2	0.76-1.37	17	48	35	
◆ 471-22-02	4	2.29-2.90	25	50	25	
▲ 471-22-03	3	1.52-2.13	19	53	28	

Project: 1773612_WO 11

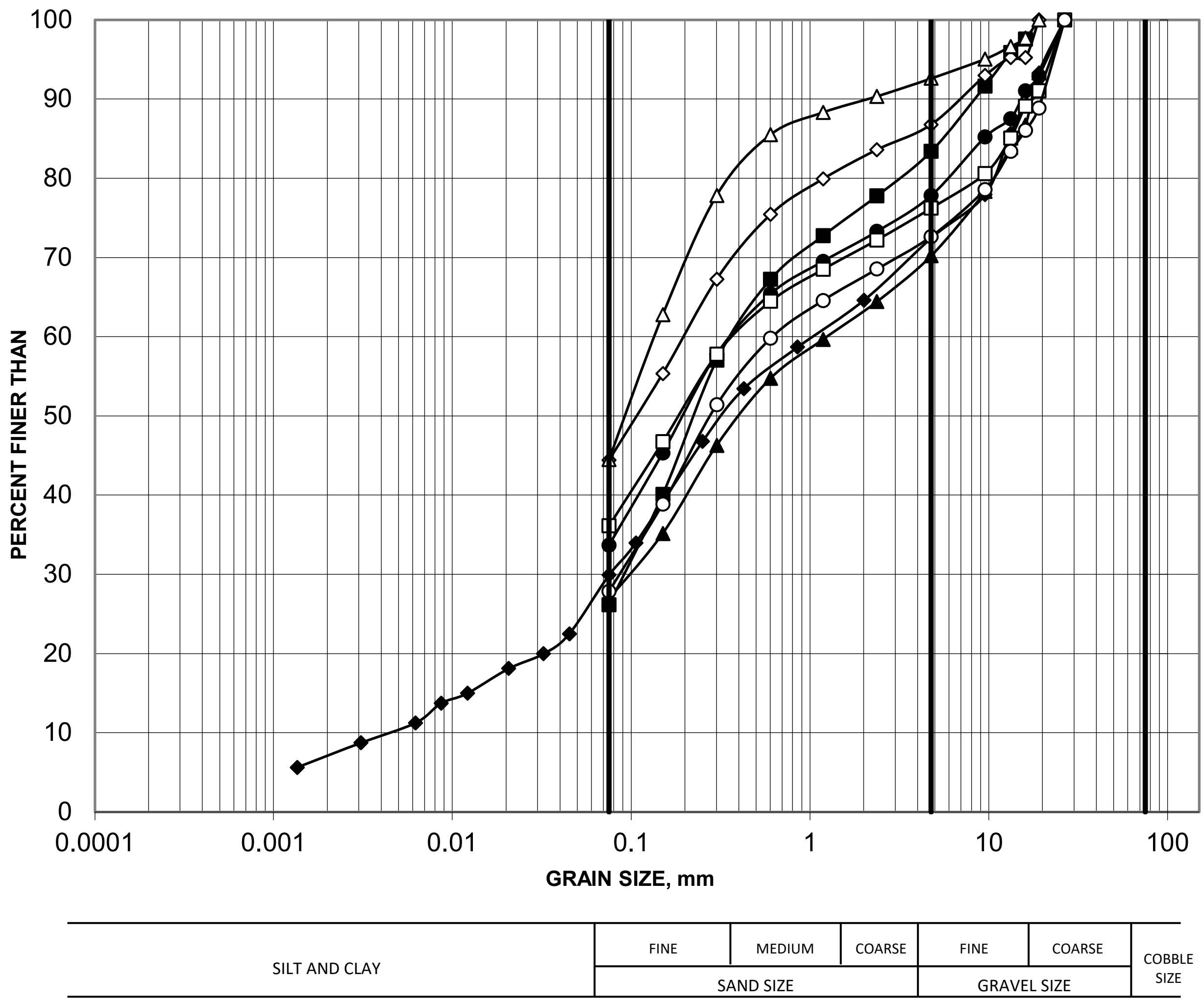
GOLDER
MEMBER OF WSP

Created by:	BW
Checked by:	PP

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab/4-Culvert 471/Figures/

GRAIN SIZE DISTRIBUTION	FIGURE B2

SILTY SAND TO SAND TILL



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■ 471-22-01	3	0.91-1.52	17	57	26	
◆ 471-22-01	5	2.29-2.90	28	42	23	7
▲ 471-22-01	8	4.57-5.18	30	43	27	
● 471-22-02	6	3.81-4.42	22	44	34	
□ 471-22-02	10	6.86-7.47	24	40	36	
◇ 471-22-03	5	3.05-3.66	13	43	44	
△ 471-22-03	12	9.91-10.11	7	49	44	
○ 471-22-04	2	0.76-1.37	27	45	28	

Project: 1773612_WO 11

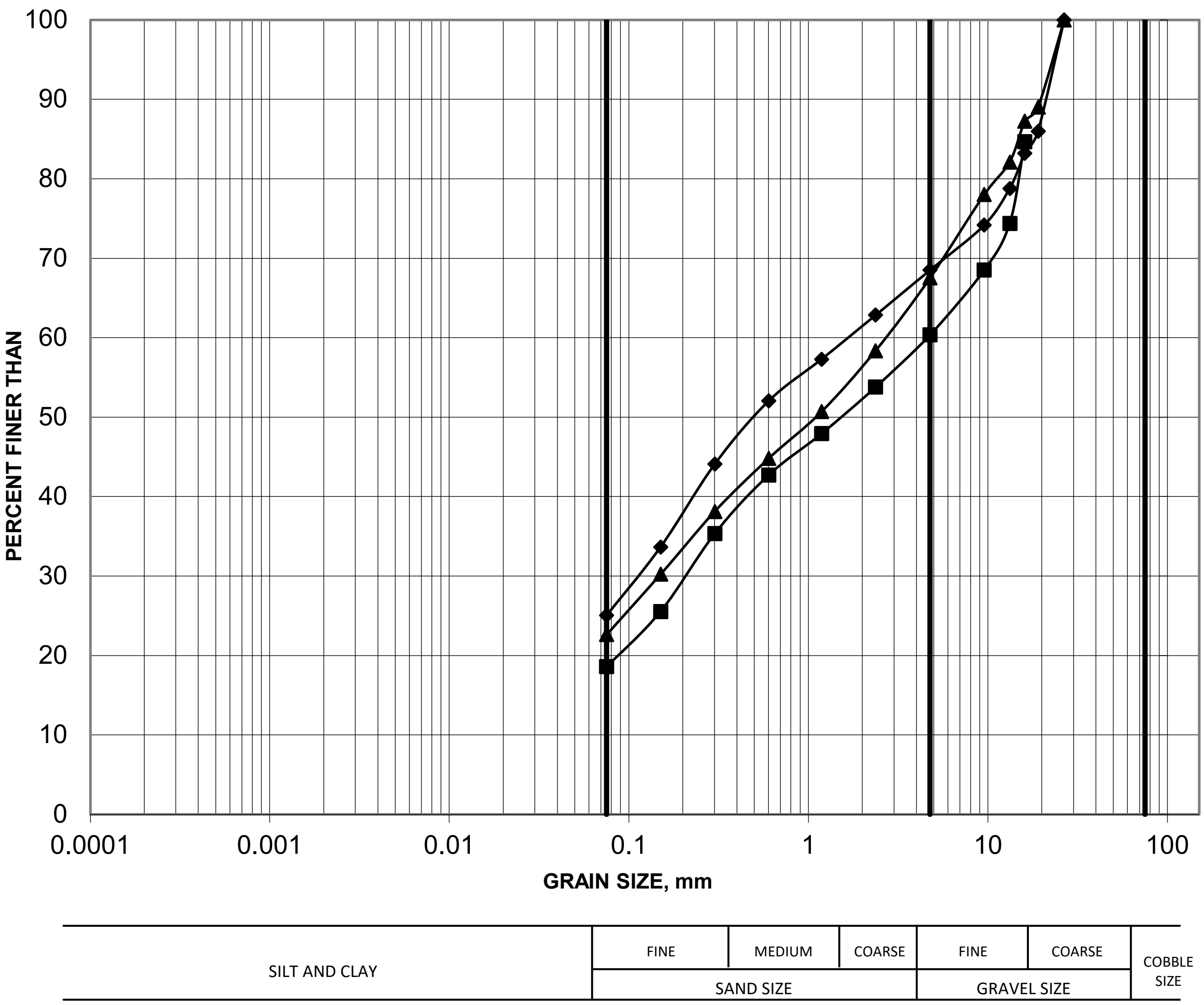
GOLDER MEMBER OF WSP

Created by:	BW
Checked by:	PP

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab/4-Culvert 471/Figures/

GRAIN SIZE DISTRIBUTION	FIGURE B3

GRAVELLY SAND TO SAND AND GRAVEL TILL



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■ 471-22-03	14	11.43-11.66	40	41	19	
◆ 471-22-04	5	3.05-3.66	31	44	25	
▲ 471-22-04	7	4.57-5.18	32	45	23	

Project: 1773612_WO 11

GOLDER
MEMBER OF WSP

Created by: BW
Checked by: PP

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab/4-Culvert 471/Figures/

APPENDIX C

Analytical Laboratory Test Results

Certificate of Analysis

Environment Testing

Client:	Golder Associates Ltd (Ottawa) 1931 Robertson Road, Ottawa, Ontario .		Report Number: Date Submitted: Date Reported: Project: COC #:	1985544 2022-09-07 2022-09-15 1773612-W011 899907
Attention:	Mr. Kenton Power			
PO#:				
Invoice to:	Golder Associates Ltd			

Dear Kenton Power:

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

Report Comments:

APPROVAL: ____________________________

Emma-Dawn Ferguson, Chemist

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: https://directory.cala.ca/.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Certificate of Analysis

eurofins | Environment Testing

Client:	Golder Associates Ltd (Ottawa) 1931 Robertson Road, Ottawa, Ontario .	Report Number:	1985544
Attention:	Mr. Kenton Power	Date Submitted:	2022-09-07
PO#:		Date Reported:	2022-09-15
Invoice to:	Golder Associates Ltd	Project:	1773612-W011
		COC #:	899907

Group	Analyte	MRL	Units	Guideline	~~1649736~~	1649737	~~1649738~~	1649739
				Lab I.D.	1649736	1649737	1649738	1649739
				Sample Matrix	Soil	Soil	Soil	Soil
				Sample Type				
				Sampling Date	2022-07-14	2022-07-20	2022-07-19	2022-07-26
				Sample I.D.	CR26-22-01 Sa3/5-7'	H-22-02 Sa2/2.5-4.5'	L-22-01 Sa2/2.5-4.5'	471-22-03 Sa3/5-7'
Anions	Cl	0.002	%		0.058	0.005	0.007	0.016
	SO4	0.01	%		0.01	0.01	<0.01	0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		1.27	0.25	0.23	0.44
	pH	2.00			8.88	9.89	9.32	9.24
	Resistivity	1	ohm-cm		787	4000	4348	2273

Group	Analyte	MRL	Units	Guideline	~~1649740~~	~~1649741~~	~~1649742~~
				Lab I.D.	1649740	1649741	1649742
				Sample Matrix	Soil	Soil	Soil
				Sample Type			
				Sampling Date	2022-07-06	2022-07-27	2022-07-04
				Sample I.D.	472-22-04 Sa2/2.5-4.5'	473-22-03 Sa2/2.5-4.5'	474-22-04 Sa3/5-7'
Anions	Cl	0.002	%		0.014	0.011	0.013
	SO4	0.01	%		0.06	<0.01	0.13
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.55	0.36	0.89
	pH	2.00			8.15	9.01	8.15
	Resistivity	1	ohm-cm		1818	2778	1124

Guideline = *** = Guideline Exceedence**

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

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

Certificate of Analysis

Client:	Golder Associates Ltd (Ottawa) 1931 Robertson Road, Ottawa, Ontario .	Report Number:	1985544
		Date Submitted:	2022-09-07
		Date Reported:	2022-09-15
Attention:	Mr. Kenton Power	Project:	1773612-W011
PO#:		COC #:	899907
Invoice to:	Golder Associates Ltd		

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 429467 **Analysis/Extraction Date** 2022-09-13 **Analyst** IP **Method** Cond-Soil			
Electrical Conductivity		90	90-110
pH	7.24	101	90-110
Resistivity			
Run No 429500 **Analysis/Extraction Date** 2022-09-14 **Analyst** IP **Method** AG SOIL			
SO4	<0.01 %	104	70-130
Run No 429575 **Analysis/Extraction Date** 2022-09-14 **Analyst** CK **Method** C CSA A23.2-4B			
Chloride	<0.002 %		90-110

Guideline = *** = Guideline Exceedence**

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

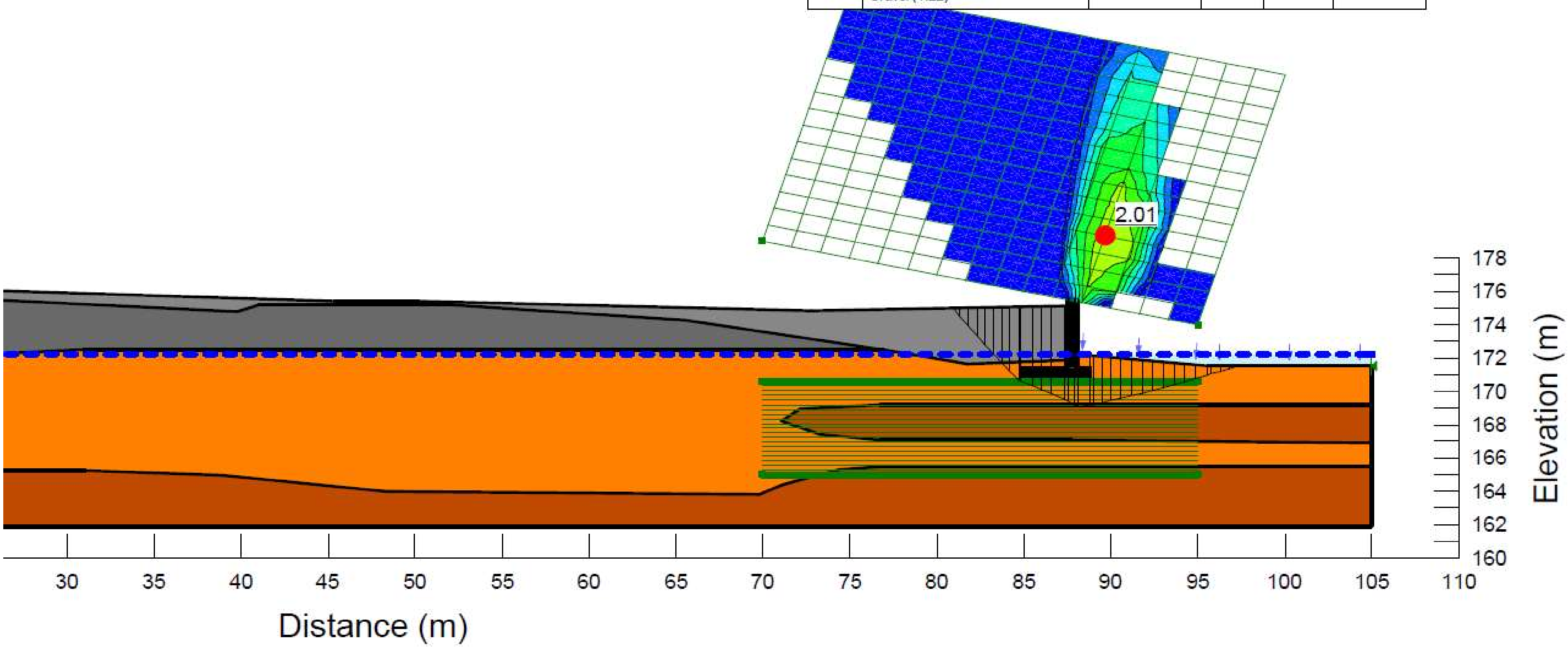
MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired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

APPENDIX D

Global Stability Analyses

Name: 1-Proposed Side Slope CV471 with RW - Static South Embankment
Analysis Type: Morgenstern-Price
Groundwater Elev.172.2 m
Direction of movement: Left to Right
Horz Seismic Coef.: 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Friction Angle (°)	Piezometric Surface
	Retaining Wall	High Strength	30		
	1. New Granular A or B Type 1 or Type 2 FILL (Compacted)	Mohr-Coulomb	22	35	1
	2. Existing Granular Fill (Loose to dense)	Mohr-Coulomb	20	32	1
	3. Silty Sand to Sand (TILL)	Mohr-Coulomb	21	34	1
	4. Gravelly Sand to Sand and Gravel (TILL)	Mohr-Coulomb	22	34	1



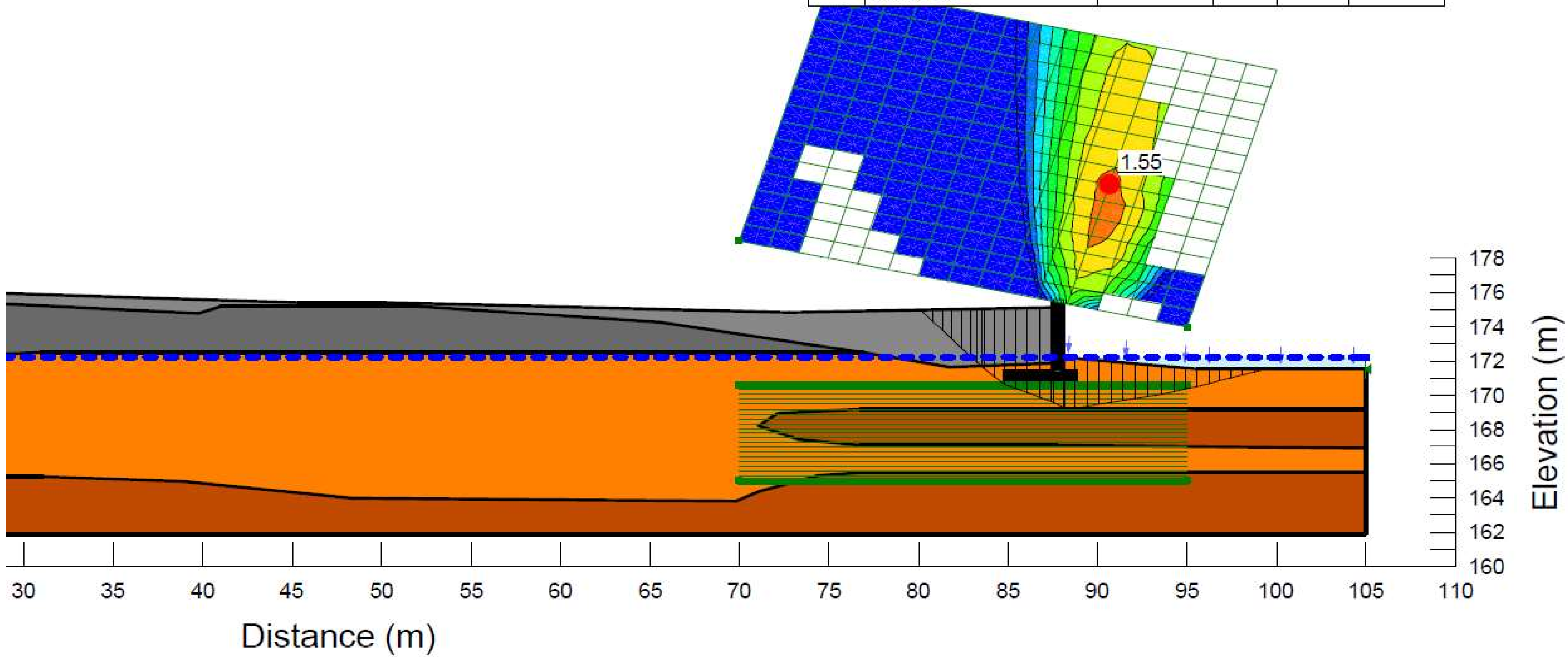
Replacement of Highway 401 Culvert Site 21X-0471/C0
Cramahe Township, Northumberland County, Ontario
MTO GWP 4054-17-00, Agreement No. 4016-E-0034-11
Global Stability - South Embankment Widening and Retaining Wall - Static Analysis

Project No:	1773612
Drawn:	BW
Date:	March 8, 2024
Checked:	KCP
Review:	LCC

FIGURE D1

Name: 2-Proposed Side Slope CV471 with RW - Siesmic South Embankment
Analysis Type: Morgenstern-Price
Groundwater Elev.172.2 m
Direction of movement: Left to Right
Horz Seismic Coef.: 0.101

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Friction Angle (°)	Piezometric Surface
	Retaining Wall	High Strength	30		
	1. New Granular A or B Type 1 or Type 2 FILL (Compacted)	Mohr-Coulomb	22	35	1
	2. Existing Granular Fill (Loose to dense)	Mohr-Coulomb	20	32	1
	3. Silty Sand to Sand (TILL)	Mohr-Coulomb	21	34	1
	4. Gravelly Sand to Sand and Gravel (TILL)	Mohr-Coulomb	22	34	1



Replacement of Highway 401 Culvert Site 21X-0471/C0
Cramahe Township, Northumberland County, Ontario
MTO GWP 4054-17-00; Agreement NO. 4016-E-0034-11
Global Stability - South Embankment Widening and Retaining Wall - Seismic Analysis

Project No:	1773612
Drawn:	BW
Date:	March 8, 2024
Checked:	KCP
Review:	LCC

FIGURE D2

APPENDIX E

Special Provisions

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

Special Provision No. 517F01 February 2024

Amendment to OPSS 517, November 2023

Return Period Flow and Preconstruction Survey Distance

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Clause 517.04.01.01 of OPSS 517 is amended by deleting the second last paragraph in its entirety and replacing it with the following:

The temporary flow passage system shall allow the work to be conducted as specified in the Contract Documents. Design flow shall include groundwater discharge and flow resulting from a minimum 2 year return period design storm, except for the work specified in Table 1. For the work specified in Table 1, design flow shall include groundwater discharge and flow resulting from a design storm of the minimum return period specified in Table 1. A longer return period shall be used when determined appropriate for the work.

The flow estimates as specified in Table 1 do not include flow volumes from groundwater discharge.

The Owner specifically excludes flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

TABLE 1
Site Location and Reference Information

<table>
<tr><th colspan="9">TEMPORARY FLOW PASSAGE SYSTEMS</th></tr>
<tr><th colspan="9">Source of Return Period Flow Estimates:</th></tr>
<tr><th rowspan="2">Site Name / Station Reference</th><th rowspan="2">Minimum Return Period (Years)</th><th colspan="4">Return Period Flow Estimates (m^3/s) (Note 1)</th><th rowspan="2">Design Engineer Requirements (Note 2)</th><th rowspan="2" colspan="2">Fish Passage Required (Note 3)</th></tr>
<tr><th>2 Year</th><th>5 Year</th><th>10 Year</th><th>25 Year</th></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td colspan="2"></td></tr>
<tr><th colspan="9">DEWATERING SYSTEMS</th></tr>
<tr><th>Site Name / Station Reference</th><th colspan="2">Preconstruction Survey Distance (m) (Note 4)</th><th colspan="4">Minimum Lowered Groundwater Depth Below Base of Excavation or Work Area (m) (Note 5)</th><th colspan="2">Design Engineer Requirements (Note 2)</th></tr>
<tr><td>Culvert 21X-0471/C0</td><td colspan="2">N/A</td><td colspan="4">1 m</td><td colspan="2">Yes</td></tr>
</table>

Notes:

1. a) The Design Engineer is to satisfy themselves to the accuracy and applicability of the provided flows.
   b) The intensity-duration-frequency (IDF) information can be accessed through MTO's IDF Curve Lookup web-based application tool at https://idfcurves.mto.gov.on.ca/
   c) The design, operation and maintenance of the temporary flow passage system is the sole responsibility of the Contractor.
2. "Yes" means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. "No" means a minimum experience level is not required for the design Engineer and design-checking Engineer.
3. "Yes" means that the design Engineer must design the temporary flow passage system to meet the fish passage requirements. "No" means fish passage is not required.
4. "N/A" means a preconstruction survey is not required.
5. Groundwater shall be lowered within the excavation or work area to below this minimum depth.

[* Designer Fill-Ins for Table 1, See Notes to Designer]

NOTES TO DESIGNER:

Designer Fill-Ins for Table 1:

1. Fill-in the source of the return period flow estimates.

2 Fill-in the site name, work, and station reference as appropriate for the dewatering system and/or temporary flow passage system item locations. Add additional rows as necessary.

3. For temporary flow passage system item locations, fill-in the minimum return period flow for each site based on MTO Drainage Design Standard TW-1. The return period flow shall not be less than 2 years.

4. For temporary flow passage system item locations, fill-in the design flow rate estimates for the various return periods.

5. Fill-in "Yes" under Design Engineer Requirements when recommended by the Foundation Engineer. Fill-in "No" otherwise.

6. For temporary flow passage system item locations, fill-in "Yes" under Fish Passage Required, when maintaining fish passage is a condition of a permit/ authorization or as recommended by the MTO Fisheries Assessment Specialist, in consultation with the MTO Environmental Planner. Fill-in "No" otherwise.

7. Fill-in the required distance under Preconstruction Survey Distance, when recommended by the Foundation Engineer. Fill-in "N/A" if not recommended.

8. Fill-in the Minimum Lowered Groundwater Depth Below Base of Excavation or Work Area provided by the Foundation Engineer.

9. When applicable, add a point d) to Note 1 of the table notes to indicate when Return Period Flow Estimates do not include base flows, for example:

   d) The Return Period Flow Estimates do not include base flows.
   d) The Return Period Flow Estimates at [enter Site Name/Description] do not include base flows.

WARRANT: Always with these tender items.

Existing Subsurface Conditions

Notice to Contractor

The Contactor is alerted to the potential for cobble and/or boulder obstructions within the fill materials. Silty sand to sand till was encountered below the fill materials in all drilled boreholes, and glacial tills inherently contain cobbles and boulders; the till deposit at this site also contains gravelly zones. The extent and depth of obstructions and of gravelly zones which may also contain cobbles and boulders may vary between and beyond the borehole locations. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for temporary works and/or construction of the replacement culvert, as applicable.

WSP.com