FINAL REPORT

Foundation Investigation and Design Report

Culvert Replacement (Site No. 21X-0474/C0)

Highway 401, Station 10+243 Cramahe Township, Northumberland County

MTO GWP 4054-17-00, Agreement No. 4016-E-0034-11

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Table of Contents

PART A

FOUNDATION INVESTIGATION REPORT

Culvert Replacement (Site No. 21X-0474/C0)
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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 foundation's scope of work includes preliminary design services for the replacement of three underpass structures and detailed design services for the replacement of four structural culverts.

This report presents the results of the foundation investigation carried out to support the detailed design of the replacement of Culvert 21X-0474/C0. The foundation investigation services for this project have been delivered under MTO Agreement No. 4016-E-0034-11 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 the magnetic north shown on the foundations drawing. For the purposes of this report, Highway 401 is oriented in a west-east direction with the culvert positioned on a 30° skew to the highway in a southeast-northwest orientation; for simplicity, the culvert is described as being oriented in a north-south direction.

Culvert 21X-0474/C0 is located at about Station 10+243 on Highway 401 approximately 3 km west of County Road 30, Cramahe Township in Northumberland County. The site location is shown on the key plan in 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 73.2 m. Based on the information provided in WSP's Draft Structural Design Report dated September 2022, there is no current information available with respect to the existing culvert inverts. We understand the culvert is in good to fair condition but is close to its 75-year design service life.

At the culvert location, Highway 401 has an existing four-lane cross-section with paved shoulders separated by a paved median and a concrete tall wall barrier, transitioning to a grassy median immediately east of the existing culvert. Based on the cross-section drawings provided by WSP, the highway grade is at approximately Elevation 188.0 m, and there is approximately 1 m of fill above the top of the existing culvert. The EBL embankment is about 2 m high with the south side slope inclined at about 8 horizontals to 1 vertical (8H:1V). The WBL embankment is about 3 m high with the north side slope inclined at 6H:1V. The ground surface conditions at the culvert location are shown in Photographs 1 to 6 following the text of this report.

The watercourse flows 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. The existing natural ground surface outside of the highway is at approximately Elevation 184.5 m to 185.5 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 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.

3.0 INVESTIGATION PROCEDURES

The fieldwork for this investigation consisted of four boreholes (Boreholes 474-22-01 to 474-22-04). Boreholes 474-22-01, 474-22-03, and 474-22-04 were advanced in June and July 2022, and Borehole 474-22-02 was advanced in October 2022. The approximate borehole locations are shown in Drawing 1.

Boreholes 474-22-01 and 474-22-04, which are located near the proposed culvert inlet and outlets, were advanced using a track-mounted Multipower LAD drilling rig equipped with 83 mm inside diameter hollow stem augers. Boreholes 474-22-02 and 474-22-03, located on the highway platform, were advanced using a truck-mounted CME 55 drilling rig equipped with 108 mm inside diameter hollow stem augers. Both drilling rigs were 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 and 1.5 m.

The groundwater levels in the open boreholes were observed during and upon completion of the drilling operations, as described in the borehole records in Appendix A. Further, a standpipe piezometer was installed in Borehole 474-22-01 to observe the stabilized groundwater level at the site. Piezometer installation details are shown on the borehole record in Appendix A. Boreholes 474-22-02 to 474-22-04 were backfilled in general accordance with the intent of Ontario Regulation (O.Reg.) 903, as amended, and the site conditions were restored following completion of the fieldwork.

The fieldwork was supervised on a full-time basis by members of WSP’s technical staff who located the boreholes in the field, supervised the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labeled 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 LS and/or ASTM Standards, as applicable at WSP’s Ottawa laboratory.

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

The as-drilled 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 (°))		
474-22-01	4880883.5 (44.060540)	200617.5 (-77.800380)	185.0	7.5
474-22-02	4880872.8 (44.060450)	200640.1 (-77.800100)	187.9	14.3
474-22-03	4880845.5 (44.060210)	200683.3 (-77.799550)	187.5	14.0
474-22-04	4880833.4 (44.060110)	200707.7 (-77.799250)	185.3	9.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The culvert lies in the physiographic region known as the Iroquois Plain, 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 the Niagara River to the 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.

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 in-situ and laboratory testing from the investigation are shown on the borehole records presented in Appendix A. The detailed results of the geotechnical laboratory are presented in Appendix B. The results of the in-situ field tests (SPT N-values), as presented in the borehole records and in Section 4, are uncorrected, and are based on the use of an automatic hammer. The results of the analytical testing completed on select soil samples are provided in Appendix C.

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

[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] Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1.

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 generally consist of the existing pavement structure (asphalt and pavement granular material) in boreholes advanced on the highway, or topsoil/peat in boreholes advanced at the culvert ends, underlain by a deposit of generally compact to dense silty sand to sand which contains thin silt and clayey silt interlayers. A more detailed description of the overburdened soil deposits encountered during the field investigation is provided in the following sections.

4.3.1 Pavement Structure and Embankment Fill

An approximately 75 mm and 175 mm thick layer of asphalt was encountered at the ground surface (i.e., Elevations 187.9 m and 187.5 m) in Boreholes 474-22-02 and 474-22-03, respectively, which were drilled through the outside shoulders of Highway 401 WBL and EBL.

Sand and gravel to silty sand fill were encountered below the asphalt in these boreholes. The top of this layer was encountered at Elevations 187.8 m and 187.3 m, and it extended to Elevations 185.7 m and 185.8 m with thicknesses of 2.1 m and 1.5 m in Boreholes 474-22-02 and 474-22-03, respectively. The recorded Standard Penetration Test (SPT) N-values within the fill ranged from 28 blows to 67 blows per 0.3 m of penetration indicating a compact to very dense state of compactness.

The measured moisture content of two samples of the fill was 3% and 7%. The results of grain size analysis testing carried out on one sample of the fill are shown in Figure B1 in Appendix B.

4.3.2 Topsoil/Peat

An approximately 200 mm thick layer of fibrous peat and a 300 mm thick layer of silty sand topsoil were encountered immediately below the ground surface (i.e., Elevations 185.0 m and 185.3 m) in Boreholes 474-22-01 and 474-22-03, which were advanced near the proposed north and south culvert ends, respectively.

4.3.3 Silty Sand (SM) to Sand (SP-SM)

A silty sand-to-sand deposit was encountered below the topsoil/peat in Boreholes 474-22-01 and 474-22-04 and the embankment fill in Boreholes 474-22-02 and 474-22-03. The top of this deposit was encountered between Elevations 184.8 m and 185.8 m. All boreholes were terminated in this deposit between Elevations 173.5 m and 177.5 m.

The recorded SPT N-values within the silty sand-to-sand deposit ranged from 0 blows (weight of hammer) to 79 blows per 0.3 m of penetration indicating very loose to very dense compactness conditions. The lower SPT N-values, in particular, those below approximately 3 m depth in Borehole 474-22-01 and low values at depth in other boreholes, are attributed to sample disturbance arising from groundwater inflow to the borehole and “heave” of silty sand to sand into the hollow stem augers, and such lower SPT N-values are not considered representative of the relative density of this deposit. In two instances, the split-spoon sampler did not penetrate the entire SPT depth due to refusal conditions (i.e., SPT N-values greater than 100 blows per 0.3 m) on inferred cobble and/or possible boulder obstructions. Therefore, in general, this deposit has a compact to dense relative density.

The measured moisture content of 13 samples of the silty sand to sand deposit ranged from 7% to 24%. The results of grain size analyses testing carried out on 12 samples of the silty to sand are provided in Figures B2 and

B3 in Appendix B. Atterberg limits tests were carried out on three samples of the deposit, which yielded non-plastic test results, as shown on the borehole records in Appendix A.

4.3.4 Silt (ML) to Clayey Silt-Silt (CL-ML) Interlayers

Interlayers of silt and clayey silt-silt were encountered within the silty sand-to-sand deposit in Boreholes 474-22-03 and 474-22-04. In Borehole 22-03, a 2.0 m thick silt interlayer was encountered at Elevation 183.7, immediately underlain by a 1.0 m thick interlayer of clayey silt-silt, while in Borehole 22-04, a 0.2 m thick clayey silt-silt interlayer was encountered at approximately Elevation 179.5 m, and a 0.9 m thick silt interlayer was encountered at approximately Elevation 177.8 m. These encountered interlayers are illustrated on the interpreted stratigraphic profile in Drawing 1, and such interlayers should be anticipated at varying depths/elevations and thicknesses throughout this deposit.

The recorded SPT N-values within the silt interlayers ranged from 15 to 39 blows per 0.3 m of penetration indicating a compact to dense state of compactness. One SPT N-value of 1 blow per 0.3 m of penetration was recorded in the clayey silt-silt interlayer in Borehole 474-22-03, suggesting a very soft consistency; however, limited sample was recovered in the split-spoon, and it is thus interpreted that this interlayer also contains silt soils that were disturbed by groundwater pressures during sampling, suggesting that this SPT N-value may not be representative of the consistency/relative density of this thin interlayer.

The measured moisture content of two samples of the silt to silt-clayey silt was 14% and 23%. The results of grain size analysis testing carried out on two samples of the silt interlayers are provided in Figure B4 in Appendix B.

4.4 Groundwater Conditions

Groundwater levels were measured within the open boreholes upon completion of drilling and within the standpipe piezometer installed in Borehole 474-22-01. The observed groundwater levels are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Groundwater Elevation (m)	Date	Reading
474-22-01	185.0	0.4	184.6	October 13, 2022	Standpipe Piezometer
		0.7	184.3	October 19, 2022	
		-0.5[1]	185.5	May 16, 2023	
474-22-02	187.9	4.6	183.3	October 17, 2022	Open Borehole
474-22-03	187.5	3.2	184.3	July 28, 2022	Open Borehole
474-22-04	185.3	0.9	184.4	July 4, 2022	Open Borehole

Note: The negative groundwater level measured within the standpipe piezometer in Borehole 474-22-01 represents a groundwater level above the existing ground surface and potential artesian conditions.

The groundwater levels at this site will be subject to fluctuations both seasonally and as a result of precipitation events.

4.5 Analytical Testing Results

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

Borehole No.	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
474-22-04	1.5-2.1	0.013	0.13	0.89	8.15	1,124

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. James Sullivan and Mr. Ben Waetcher, under the overall direction of Mr. Kenton Power, P.Eng. This Foundation Investigation Report was prepared by Tibor Berecz, P.Eng. and reviewed by Kenton Power, P.Eng., Senior Geotechnical Engineer. Ms. Lisa Coyne, P.Eng., Geotechnical Engineering Fellow and MTO Principal Foundations Contact conducted an independent technical and quality review of this report.

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https://wsponlinecan.sharepoint.com/sites/ca-ca00122984565/shared documents/06. deliverables/474/3-final/1773612 gwp 4045-17-00 fidr rev0 culvert 474 2024-03-05.docx

PART B

FOUNDATION DESIGN REPORT

Culvert Replacement (Site No. 21X-0474/C0)
Highway 401, Station 10+243 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-0474/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 in the vicinity of Culvert 21X-0474/C0, Highway 401 is to be rehabilitated and widened from the existing four-lane configuration (i.e., two lanes in each direction) 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 this 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 1.5 m of fill on the existing embankment side slopes and nominal regarding in the center median swale. The ultimate configuration will require a further widening of approximately 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-0474/C0 is to be replaced on a new alignment approximately 24 m east of the existing culvert (as measured midpoint-to-midpoint along the Highway 401 centreline), with a skew of approximately 43° as dictated by drainage, fluvial and fisheries requirements. The replacement culvert will be approximately 113 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), 73.2 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 found 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 one) 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 183.33 m at the south (inlet) end to Elevation 183.25 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 or vertical seismic profiling may provide a more favorable 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	0.203
Sa (0.2)	0.351
Sa (0.5)	0.334
Sa (1.0)	0.199
Sa (2.0)	0.0953
Sa (5.0)	0.0254
Sa (10.0)	0.00798
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 containing silt and clayey silt interlayers. 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 found 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/leveling course and installing the replacement culvert, it is recommended that any organic material (i.e., topsoil, peat and/or mixed organic soils), existing fill, and any disturbed materials encountered below the footprint of the box culvert or culvert open footings 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 leveling pad requirements for a pre-cast box culvert should be in accordance with OPSS.PROV 422 (Precast 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 a maximum of 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 leveling pad consisting of OPSS.PROV 1010 (Aggregates) Granular 'A' or fine concrete aggregate meets 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 Level and Factored Axial Geotechnical Resistances

Based on a 350 mm thick concrete bottom slab and the bedding and leveling layer thicknesses recommended above, the founding subgrade level for the replacement culvert will be at approximately Elevation 182.75 m at the south (inlet) end and Elevation 182.67 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/leveling course overlying the native soils at the above-noted elevations, the following factored geotechnical resistances may be used for the design:

- Factored ultimate geotechnical resistance: 400 kPa
- Factored serviceability geotechnical resistance (for 25 mm of settlement): 150 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 inverted elevations as summarized in Section 6.1, the footings should be found at about Elevation

181.9 m to provide a minimum of 1.4 m of soil cover for frost protection. If precast footings are utilized, a minimum 150 mm thick bedding layer and 75 mm thick leveling layer (as discussed in Section 6.5.1) should be placed directly below the underside of the footings.

For 1.2 m wide open footings founded on the properly prepared native soils at Elevation 181.9 m, the following factored geotechnical resistances may be used for the 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, the 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 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 found at or below the elevations given below; the footings may need to be found deeper to achieve a minimum depth of 1.4 m below the 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)	183.0	2	275	175
		3	325	150
North (Outlet)	182.5	2	250	150
		3	300	125

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 $\Phi_{gu} = 0.50$ for “Bearing” and $\Phi_{gs} = 0.80$ 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, the 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/leveling layer	δ'_i = 20°, c'_i = 0 kPa
Between the granular bedding layer and underlying silty sand-to-sand	ϕ' = 32°, c' = 0 kPa
Between cast-in-place culvert and/or retaining wall footings and native silty sand-to-sand	ϕ' = 32°, 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 a 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 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 a concrete cut-off wall, retaining walls or a clay seal. Based on the preliminary GA drawing, it is understood that approximately 5 m long concrete retaining walls and 0.9 m deep cut-off walls are to be constructed at 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/or 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 thinner than the standard natural clay or soil-bentonite layer, thus requiring a shallower excavation into the slope, and is generally 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 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 of 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 a 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 with 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 pressure 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 the 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 fills or select subgrade material is used for embankment construction, the exposed materials will be susceptible to erosion and shallow raveling. To reduce surface water erosion and raveling 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 requirements for remedial works on the slope faces prior to topsoil dressing and seeding.

6.7.2 Global Stability of Widened Embankment Side Slopes

The existing embankments are up to approximately 3 m to 3.5 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 grade raise). We further understand the existing embankment side slopes will generally be maintained or slightly flattened 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 in 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 185.5 m, approximately at the base of the embankment fill.
- 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 3.5 m to 4 m in total height, and a footing width of approximately 3 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 8 m to 9 m widening is proposed along the outside of both the WBL and EBL embankments resulting in grade raises of about 1.5 m and 0.9 m at the north and south toes of the slope, respectively.

To estimate the magnitude of the settlement as a result of the proposed embankment widening, analyses were carried out on both the north and south toes of the 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 the placement of any new embankment fill material for the widening.

The immediate compression of the native soils was modeled based on typically accepted correlations with the 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 in Figure 1. The groundwater level was assumed to be at approximately Elevation 185 m (i.e., roughly at the 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 474-22-01.

Material	Unit Weight (kN/m^3)	Elastic Modulus (MPa)
Silty Sand to Sand (generally compact to dense)	19	40

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(1)
Westbound Lane				
Replacement Culvert (WBL)	~ 10 mm	~15 mm	~20 mm	~ 1,425:1
Replacement Culvert (EBL)	5 - 10 mm	5 -10 mm	~ 10 mm	~ 2,850:1

Notes:
1. The differential settlement has been 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 corresponding new shoulder following embankment widening.

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

The above estimates do not include compression of the fill itself, which would occur during the 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 twenty 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 474-22-4 are summarized in Section 4.5 and included in Appendix C. The potential for sulphate attack and corrosion are discussed in the following sub-sections; however, it is ultimately up to the designer to determine the appropriate construction materials, including the exposure class, and ensure 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 0.13%, which is in the exposure class of S-3 (Moderate) and is considered Moderate according to Table 7.2 in the MTO Gravity Pipe Design Guidelines (2014) and the use of Type II cement is recommended. Also 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, appropriate.

6.8.2 Potential for Corrosion

The soil has a pH of 8.15 and according to the MTO Gravity Pipe Design Guidelines (2014), pH levels between 5.5 and 8.5 are not considered detrimental to culvert durability. The measured resistivity, R, of 1,124 ohm-cm indicates that the soil corrosiveness is severe (R<2,000), as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014).

6.9 Construction Considerations

6.9.1 Construction Staging and Temporary Protection Systems

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 proceeds. 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. Further, depending on the sheet pile (or soldier pile) tip installation depth, there is a potential risk for artesian conditions, as observed in the standpipe piezometer installed in Borehole 474-22-01 at the time of our May 16, 2023, reading. The contractor should be alerted to the potential for artesian groundwater conditions. A sample Notice to the Contractor is included in Appendix E.

Although the Contractor is responsible for the selection and 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/m^3)	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	22	35	0.27	0.43	3.69
Existing Embankment Fill (loose to dense)	20	32	0.31	0.47	3.25
Silty Sand to Sand containing silt and clayey silt interlayers (generally compact to dense)	19	30	0.33	0.5	3.0

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. The 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

It is anticipated that the creek flow will be maintained within the existing culvert while the replacement culvert is being constructed. Given the permeable subgrade soils encountered at this site and the anticipated depth of the excavations (i.e., extending about 1 m to 2.5 m below the measured groundwater level), a temporary dewatering system potentially in conjunction with a cofferdam/cut-of 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 with consideration of the potential risk for artesian groundwater conditions.

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 of 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 low from a foundation perspective provided the pumping is carried out from properly filtered sumps/well points. As such, the Foundation fill-in 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 leveling 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 native subgrade soils at this site, as inferred to be present based on instances of split-spoon refusal in Borehole 474-22-03. The extent and depth of the obstructions may vary beyond and between the borehole locations. A sample Notice to the Contractor is included in Appendix E.

7.0 CLOSURE

This Foundation Design Report was prepared by Tibor Berecz, P.Eng. and reviewed by Kenton Power, P.Eng., Senior Geotechnical Engineer. Ms. Lisa Coyne, P.Eng., Geotechnical Engineering Fellow and MTO Principal Foundations Contact conducted an independent technical and quality review of this report.

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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.PROV 803	Construction Specification for Vegetative Cover
OPSS.PROV 804	Construction Specification for Temporary Erosion Control
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 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 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 the inlet to mitigate potential scour under the culvert(s). ▪ Difficult to compact backfill materials to the 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.

DRAWINGS

DRAWING 1

BOREHOLE LOCATIONS AND SOIL STRATA

MINISTRY OF TRANSPORTATION, ONTARIO

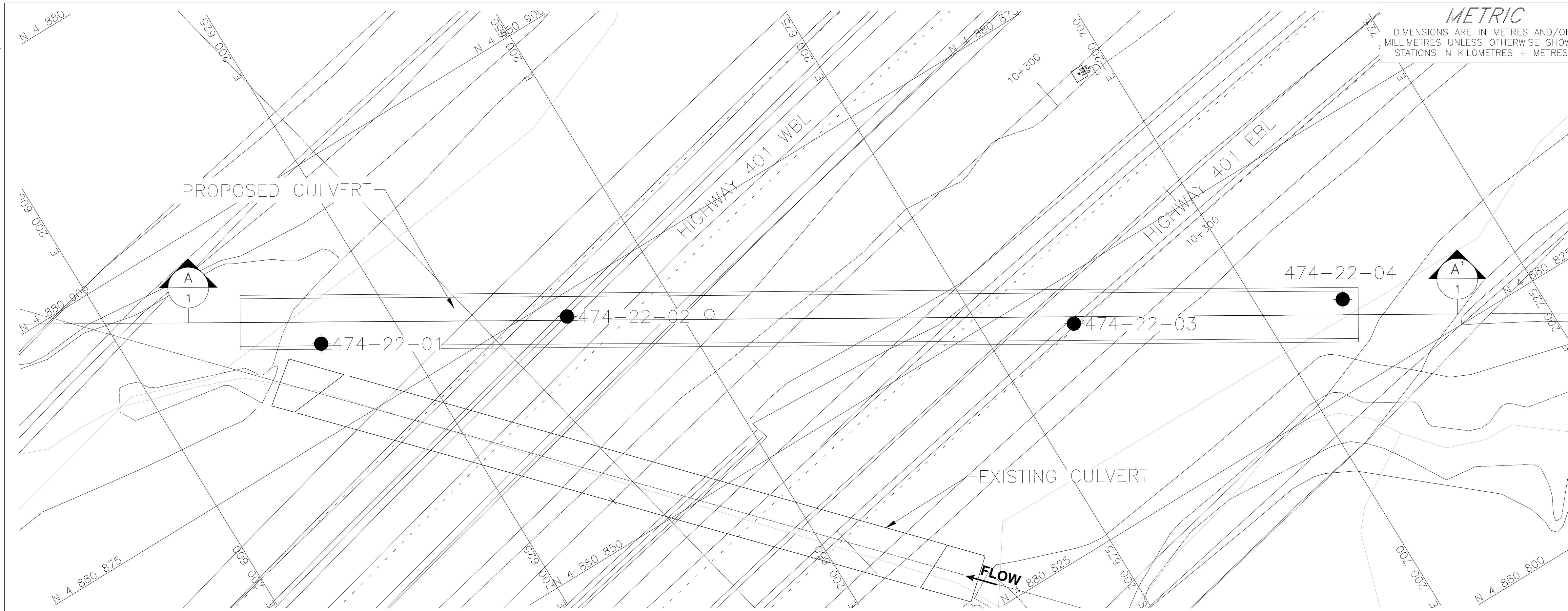
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.4054-17-00

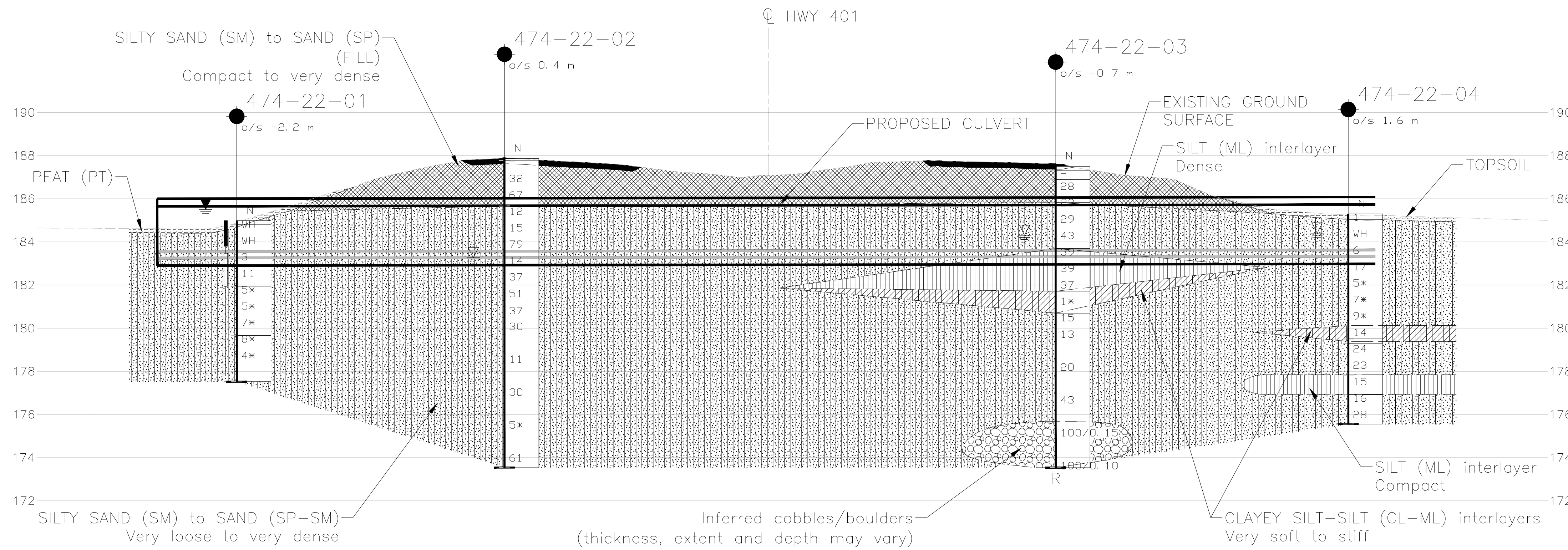
HIGHWAY 401 WIDENING
CULVERT 21X-0474/C0
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

wsp



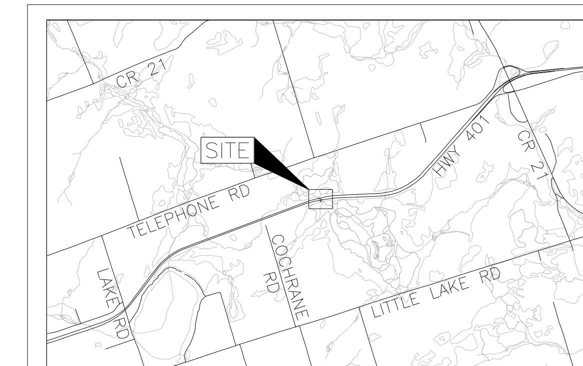
PLAN
SCALE
5 0 5 10 m



PROFILE

VERTICAL SCALE
2.5 0 2.5 5 m

HORIZONTAL SCALE
5 0 5 10 m



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- ● Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R REFUSAL
- Seal
- Piezometer
- WL in piezometer, measured on May 16, 2023.
- WL upon completion of drilling

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Mar. 07, 2024 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER K.T.C. POWER 100130652 Mar. 07, 2024 PROVINCE OF ONTARIO

BOREHOLE CO-ORDINATES NAD 83/MTM ZONE 9			
No.	ELEVATION	NORTHING	EASTING
474-22-01	185.0	4880883.5	200617.5
474-22-02	187.9	4880872.8	200640.1
474-22-03	187.5	4880845.5	200683.3
474-22-04	185.3	4880833.4	200707.7

Structural Site Location Latitude: 44.060540° Longitude: -77.800380°

NOTES

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

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

REFERENCE

Base plans provided in digital format by WSP, drawing file no. Mainline-8Lane proposed alignment for Culvert Sections_ACAD (updated - April 12 2022).dwg, received APR. 14, 2022 and drawing file no. 17M-01712-11-306-001GA.dwg, received AUG. 4, 2022.

NO.	DATE	BY	REVISION

Geocres No. 31C04-002

HWY. 401	PROJECT NO. 1773612		DIST. EASTERN
SUBM'D. BW	CHKD. BW	DATE: 03/07/2024	SITE: 21-0474/C0
DRAWN: ZS/DD	CHKD. KCP	APPD. LCC	DWG. 1

PLOT DATE: March 7, 2024
FILENAME: S:\Clients\MTO\Hwy_401_Colborne_to_Brighton\99_PROJ\1773612_Parsons_CulvertReplacement\40_PROD\0004_Geotech_Culvert_4\1773612-0004-BD-0002.dwg

PHOTOGRAPHS

PHOTOGRAPHS 1 TO 6

Photograph 1: Highway 401 Westbound Lane, Facing East

Photograph 2: Highway 401 EBL & Existing Culvert Inlet (South End), Facing West

Photograph 3: Existing Culvert Outlet (North End), Facing North

Photograph 4: Existing Culvert Outlet (North End), Facing East

Photograph 5: Highway 401 Westbound Lane, North Embankment Side Slope, Facing East

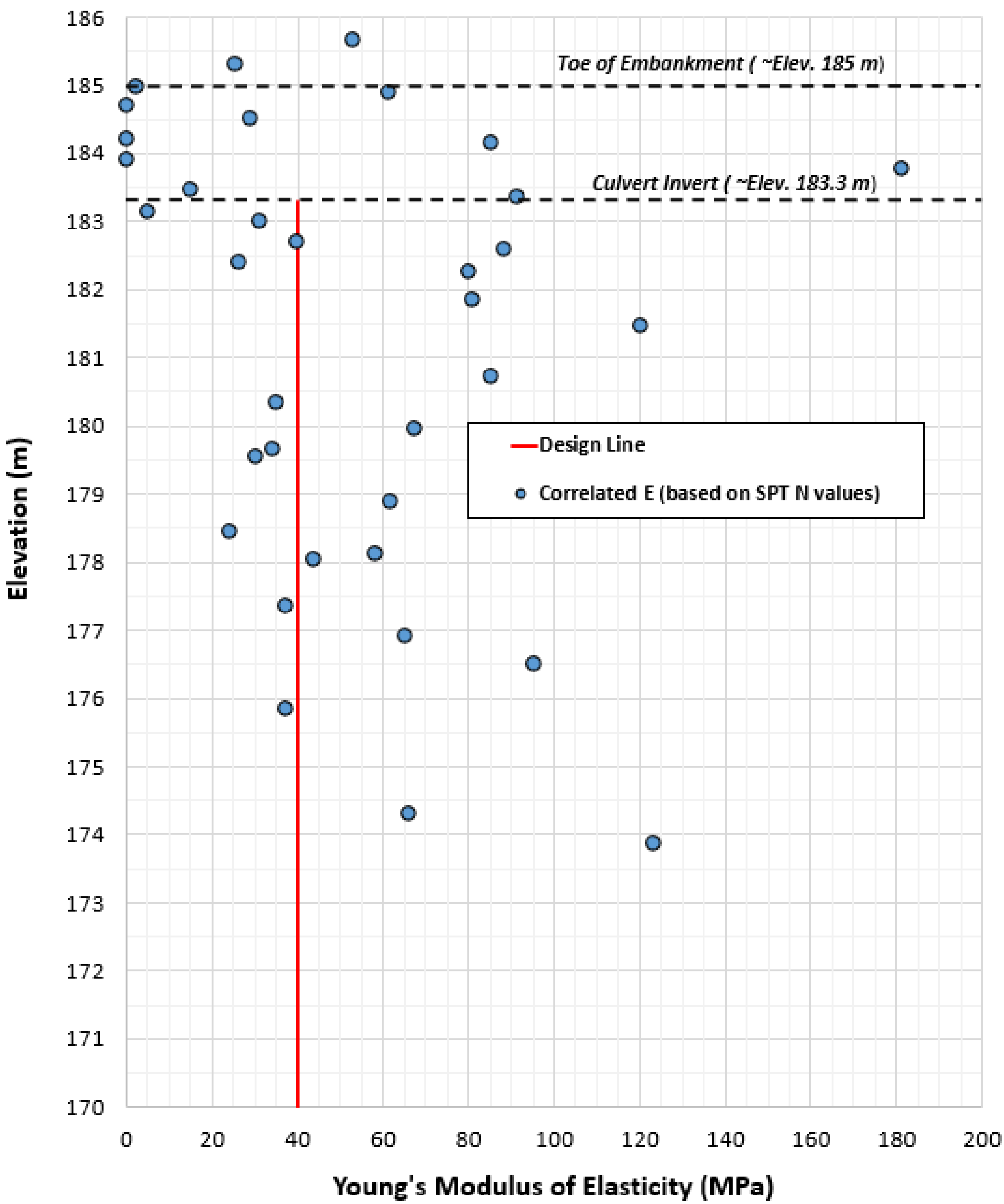
Photograph 6: Highway 401 Eastbound Lane, South Embankment Side Slope, Facing East

FIGURES

Figure 1 Settlement Parameters – Elastic Modulus

Settlement Parameters - Modulus of Elasticity

Figure 1



Date: June 2023

Project No: 1773612 WO#11

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 (ϕ = 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 474-22-01

SHEET 1 OF 1 **METRIC**

PROJECT 1773612
G.W.P. 4054-17-00
LOCATION N 4880883.5; E 200617.5 MTM NAD ZONE 9 (LAT. 44.060540; LONG. -77.800380)
ORIGINATED BY JS
DIST Eastern HWY 401
BOREHOLE TYPE LAD Multipower Track Mounted, 83 mm I.D. Hollow Stem Augers
COMPILED BY ZS
DATUM GEODETIC
DATE June 27 & 28, 2022
CHECKED BY KCP

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	Water Content (%)	Remarks & Grain Size Distribution (%) GR SA SI CL
185.0	GROUND SURFACE					
0.0	Fibrous PEAT (PT), some sand					
0.2	SILTY SAND (SM) Very loose to compact Brown Wet	1	SS	WH		
		2	SS	WH		
		3A / 3B	SS	3		
		4	SS	11	~18	0 67 (33)
182.0 / 3.0	SAND (SP), trace silt Loose Brown Wet	5	SS	5*		
		6	SS	5*	~20	0 94 (6)
		7	SS	7*		
179.7 / 5.3	SILTY SAND (SM) Loose Brown Wet	8	SS	8*	~21	0 73 (27)
		9	SS	4*		
177.5 / 7.5	END OF BOREHOLE					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (184, 183, 182, 181, 180, 179, 178); 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

NOTES:

* SPT N values impacted by sample disturbance due to groundwater conditions and heave within hollow stem augers.

1. Between 0.6 m and 1.5 m of heaving occured prior to sampling for each of samples 5 to 9.

2. Borehole terminated due to excessive heave inside augers.

3. Water level measured in standpipe piezometer:

Date	Depth(m)	Elev.(m)
Oct.12/22	0.4	184.6
Oct.19/22	0.7	184.3
May16/23	-0.5	185.5

4. The negative groundwater level measure represents a groundwater level above the existing ground surface and potential artesian conditions.

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

Foundation Design

RECORD OF BOREHOLE No 474-22-02

SHEET 1 OF 2

METRIC

PROJECT 1773612

G.W.P. 4054-17-00

DIST Eastern HWY 401

DATUM GEODETIC

LOCATION N 4880872.8; E 200640.1 MTM NAD ZONE 9 (LAT. 44.060450; LONG. -77.800100)

BOREHOLE TYPE CME 55 Truck Mounted, 108 mm I.D. Hollow Stem Augers

DATE October 17, 2022

ORIGINATED BY BW

COMPILED BY ZS

CHECKED BY KCP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
187.9	GROUND SURFACE											
0.0	ASPHALT (75 mm)											
0.1	Gravelly SILTY SAND (SM) (FILL) Dense to very dense Brown Moist	1	AS	-								
		2	SS	32		187	o					
		3	SS	67		186						
185.7 / 2.2	SILTY SAND (SM) to gravelly SILTY SAND Loose to very dense Brown Moist to wet	4	SS	12		185						
		5	SS	15			o					
		6	SS	79		184	o	NP	14	49	30	7
					▽							
		7	SS	14		183						
		8	SS	37		182						
		9	SS	51		181						
		10	SS	37			o	NP	0	86	10	4
		11	SS	30		180						
						179						
		12	SS	11		178						
		13	SS	30		177						
						176						

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

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

RECORD OF BOREHOLE No 474-22-02 SHEET 2 OF 2 METRIC

PROJECT 1773612
G.W.P. 4054-17-00
LOCATION N 4880872.8; E 200640.1 MTM NAD ZONE 9 (LAT. 44.060450; LONG. -77.800100)
ORIGINATED BY BW
DIST Eastern HWY 401
BOREHOLE TYPE CME 55 Truck Mounted, 108 mm I.D. Hollow Stem Augers
COMPILED BY ZS
DATUM GEODETIC
DATE October 17, 2022
CHECKED BY KCP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---											
	SILTY SAND (SM) to gravelly SILTY SAND Loose to very dense Brown Moist to wet	14	SS	5*			○	NP	20	45	23	6
						175						
						174						
173.6		15	SS	61			○		29	45	20	6
14.3	END OF BOREHOLE											

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
REMARKS & GRAIN SIZE DISTRIBUTION (%)

NOTES:

1. Water level measured at a depth of 4.6 m (Elev. 183.3 m) upon completion of drilling.

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

Foundation Design

RECORD OF BOREHOLE No 474-22-03 SHEET 1 OF 2 METRIC

PROJECT 1773612

G.W.P. 4054-17-00

LOCATION N 4880845.5; E 200683.3 MTM NAD ZONE 9 (LAT. 44.060210; LONG. -77.799550)

ORIGINATED BY JS

DIST Eastern HWY 401

BOREHOLE TYPE CME 55 Truck Mounted, 108 mm I.D. Hollow Stem Augers

COMPILED BY ZS

DATUM GEODETIC

DATE July 28, 2022

CHECKED BY KCP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
187.5	GROUND SURFACE						
0.0	ASPHALT (175 mm)						
0.2	SAND (SP) and gravel, trace silt (FILL) Brown Moist	1	AS	-		o	44 48 (8)
186.9 / 0.6	SILTY SAND (SM), some gravel (FILL) Compact Brown to grey Moist	2	SS	28			
185.8 / 1.7	SILTY SAND (SM), trace gravel Compact to dense Brown Moist	3	SS	23			
		4	SS	29		o	8 49 (43)
		5	SS	43	∇		
183.7 / 3.8	SILT (ML) Dense Grey Moist	6	SS	39			
		7	SS	39		o	0 3 87 10
		8	SS	37			
181.7 / 5.8	CLAYEY SILT-SILT (CL-ML), trace to some sand Firm Brown to grey Wet	9	SS	1*			
180.7 / 6.8	SILTY SAND (SM), trace gravel Compact to dense Brown Wet	10	SS	15			
		11	SS	13			
		12	SS	20			
		13	SS	43		o	3 56 (41)
	- Hard augering below 11.6 m depth (Elev. 175.9 m), contains cobbles and boulders						

ELEVATION SCALE: 187, 186, 185, 184, 183, 182, 181, 180, 179, 178, 177, 176

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/m^3

Continued Next Page

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

Foundation Design

RECORD OF BOREHOLE No 474-22-03

SHEET 2 OF 2 **METRIC**

PROJECT 1773612			
G.W.P. 4054-17-00	LOCATION N 4880845.5; E 200683.3 MTM NAD ZONE 9 (LAT. 44.060210; LONG. -77.799550)	ORIGINATED BY JS	
DIST Eastern HWY 401	BOREHOLE TYPE CME 55 Truck Mounted, 108 mm I.D. Hollow Stem Augers	COMPILED BY ZS	
DATUM GEODETIC	DATE July 28, 2022	CHECKED BY KCP	

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	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
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY SAND (SM), trace gravel Compact to dense Brown Wet		14	SS	100/0.15		175				
							174				
173.5			15	SS	100/0.10						
14.0	END OF BOREHOLE SPLIT-SPOON REFUSAL NOTES: 1. Water level measured at a depth of 3.2 m (Elev. 184.3 m) upon completion of drilling										

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

GTA-MTO 001 S:\CLIENTS\MTO\HWY_401_COLBORNE_TO_BRIGHTON\02_DATA\GINT\HWY_401_COLBORNE_TO_BRIGHTON.GPJ GAL-GTA.GDT 3/8/24

RECORD OF BOREHOLE No 474-22-04 SHEET 1 OF 1 METRIC

PROJECT 1773612
G.W.P. 4054-17-00
LOCATION N 4880833.4; E 200707.7 MTM NAD ZONE 9 (LAT. 44.060100; LONG. -77.799250)
ORIGINATED BY JS
DIST Eastern HWY 401
BOREHOLE TYPE LAD Multipower Track Mounted, 83 mm I.D. Hollow Stem Augers
COMPILED BY ZS
DATUM GEODETIC
DATE July 4, 2022
CHECKED BY KCP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
185.3	GROUND SURFACE										
0.0 / 185.1 / 0.3	TOPSOIL - SILTY SAND (SM), some organics, Dark brown, Moist	1	SS	1		185					
	SILTY SAND (SM), Loose to compact, Brown to grey, Moist to wet	2	SS	WH	▽						
		3	SS	6		184	o	1	54	(45)	
		4	SS	17		183					
		5	SS	5*		182					
		6	SS	7*			o	0	81	(19)	
		7	SS	9*		181					
		8	SS	14		180					
179.5	CLAYEY SILT-SILT (CL-ML), trace sand, Brown, Wet										
6.0	SILTY SAND (SM), Compact, Brown, Wet	9	SS	24		179					
		10	SS	23		178					
177.8 / 7.5	SILT (ML) and sand, Compact, Brown, Wet	11	SS	15		177	o	0	44	(56)	
176.9 / 8.4	Gravelly SILTY SAND (SM), Compact, Brown, Wet	12	SS	16		176					
		13	SS	28			o	26	45	(29)	
175.6 / 9.8	END OF BOREHOLE										

NOTES:

1. Water level measured at a depth of 0.9 m (Elev. 184.4 m) upon completion of drilling.

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; 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

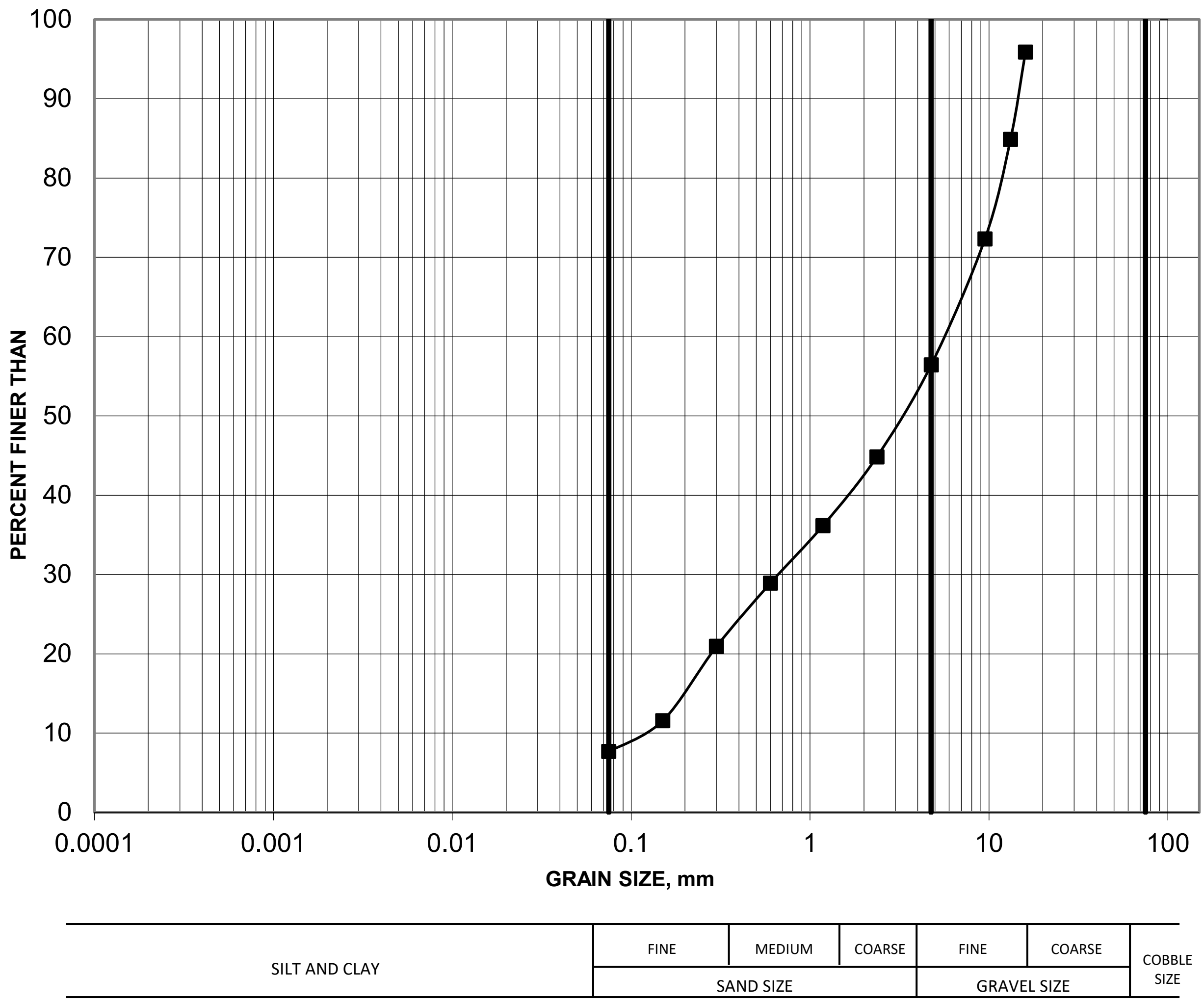
+3, ×3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

SAND (SP) FILL



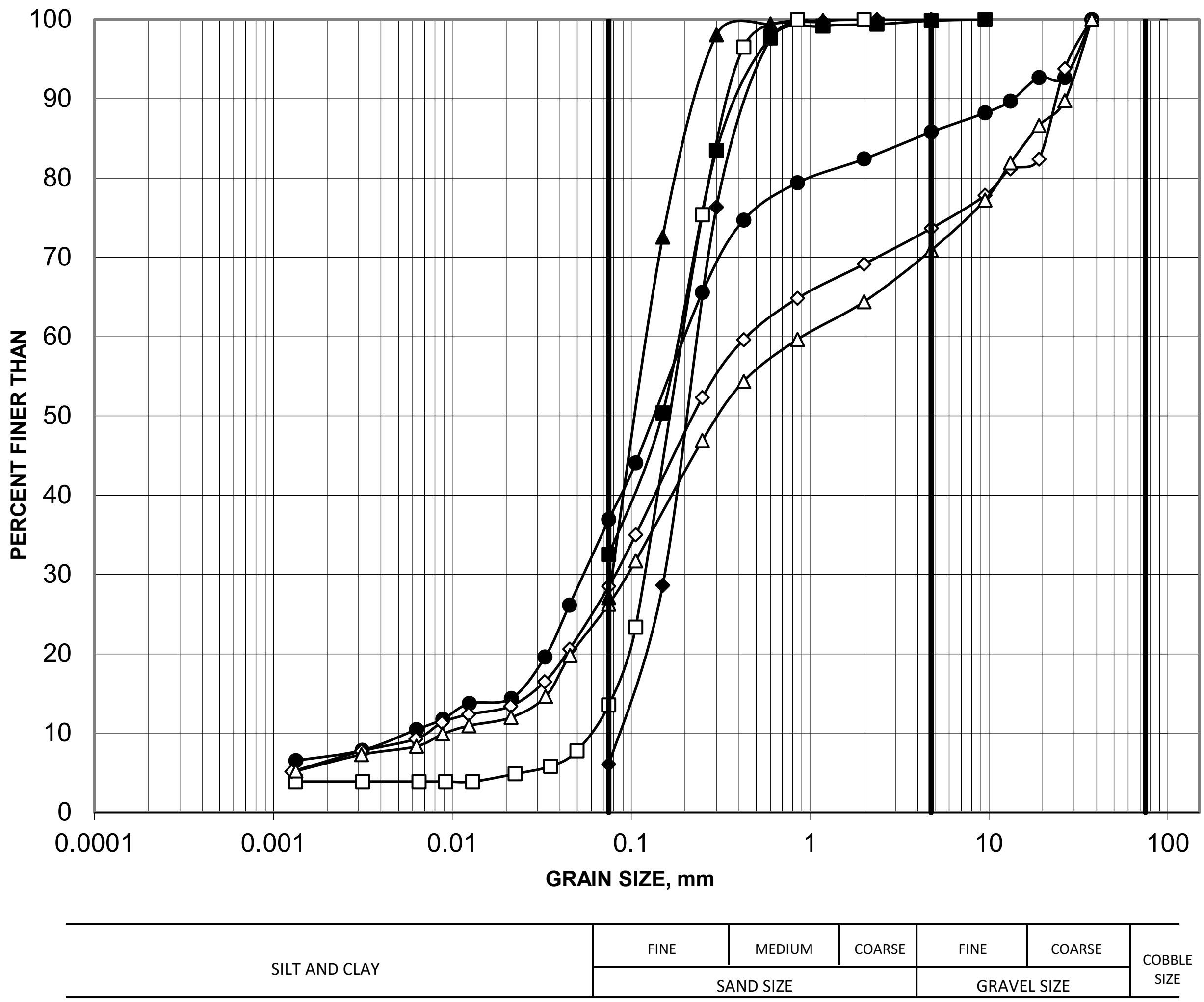
			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
474-22-03	1	0.18-0.46	44	48	8	

Project: 1773612_WO 11

Created by: KG
Checked by: KCP

GRAIN SIZE DISTRIBUTION	FIGURE B2

SILTY SAND (SM) to SAND (SP-SM)



Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
474-22-01	4	2.29-2.90	0	67	33	
474-22-01	6	3.81-4.42	0	94	6	
474-22-01	8	5.33-5.94	0	73	27	
474-22-02	6	3.81-4.42	14	49	30	7
474-22-02	10	6.86-7.47	0	86	10	4
474-22-02	14	12.19-12.80	26	45	23	6
474-22-02	15	13.72-14.33	29	45	20	6

Project: 1773612_WO 11

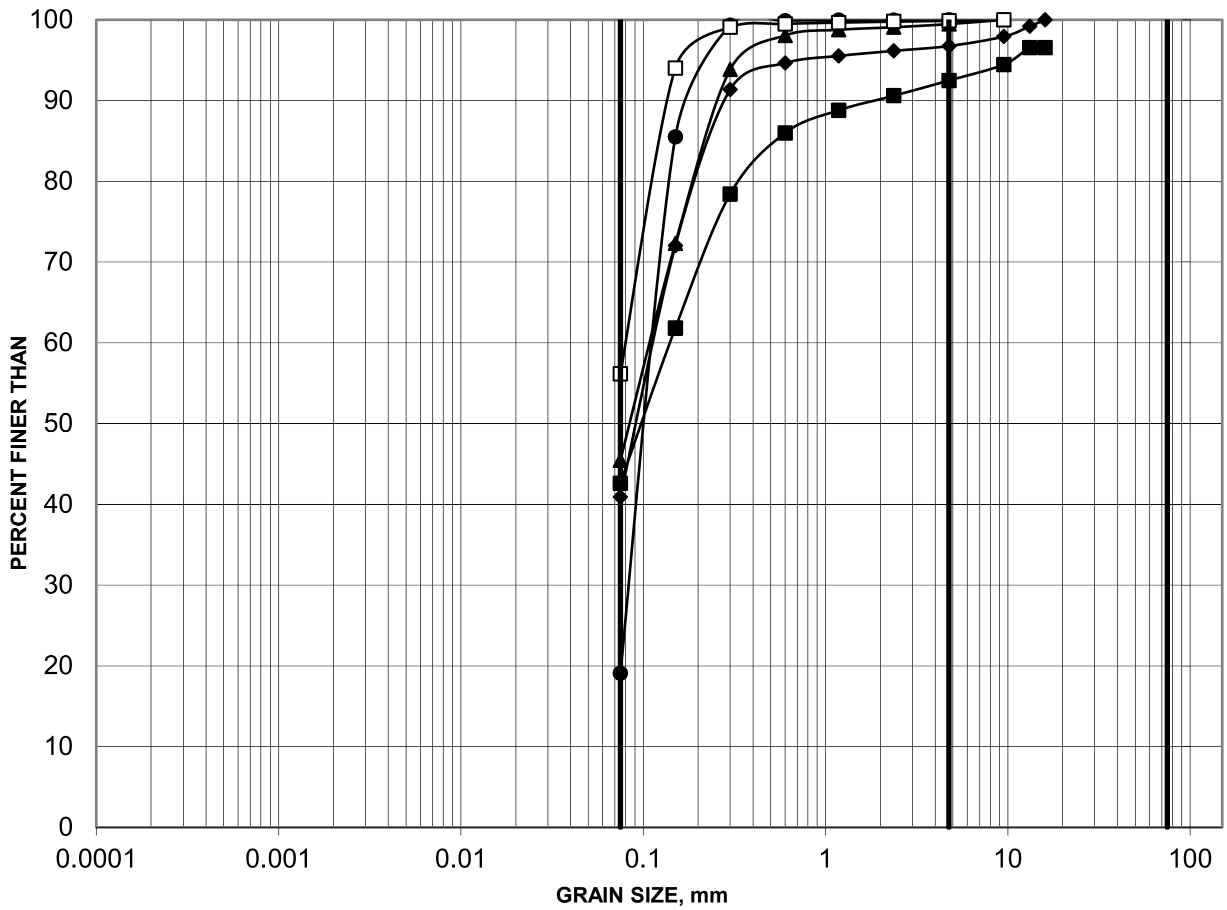
GOLDER
MEMBER OF WSP

Created by: KG
Checked by: KCP

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab/7-Culvert 474/Figures/

GRAIN SIZE DISTRIBUTION	FIGURE B3

SILTY SAND (SM) to SAND (SP-SM)



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■ 474-22-03	4	2.29-2.90	8	49	43	
◆ 474-22-03	13	10.67-11.28	3	56	41	
▲ 474-22-04	3	1.52-2.13	1	54	45	
● 474-22-04	6	3.81-4.42	0	81	19	
□ 474-22-04	11	7.62-8.23	0	44	56	

Project: 1773612_WO 11

GOLDER
MEMBER OF WSP

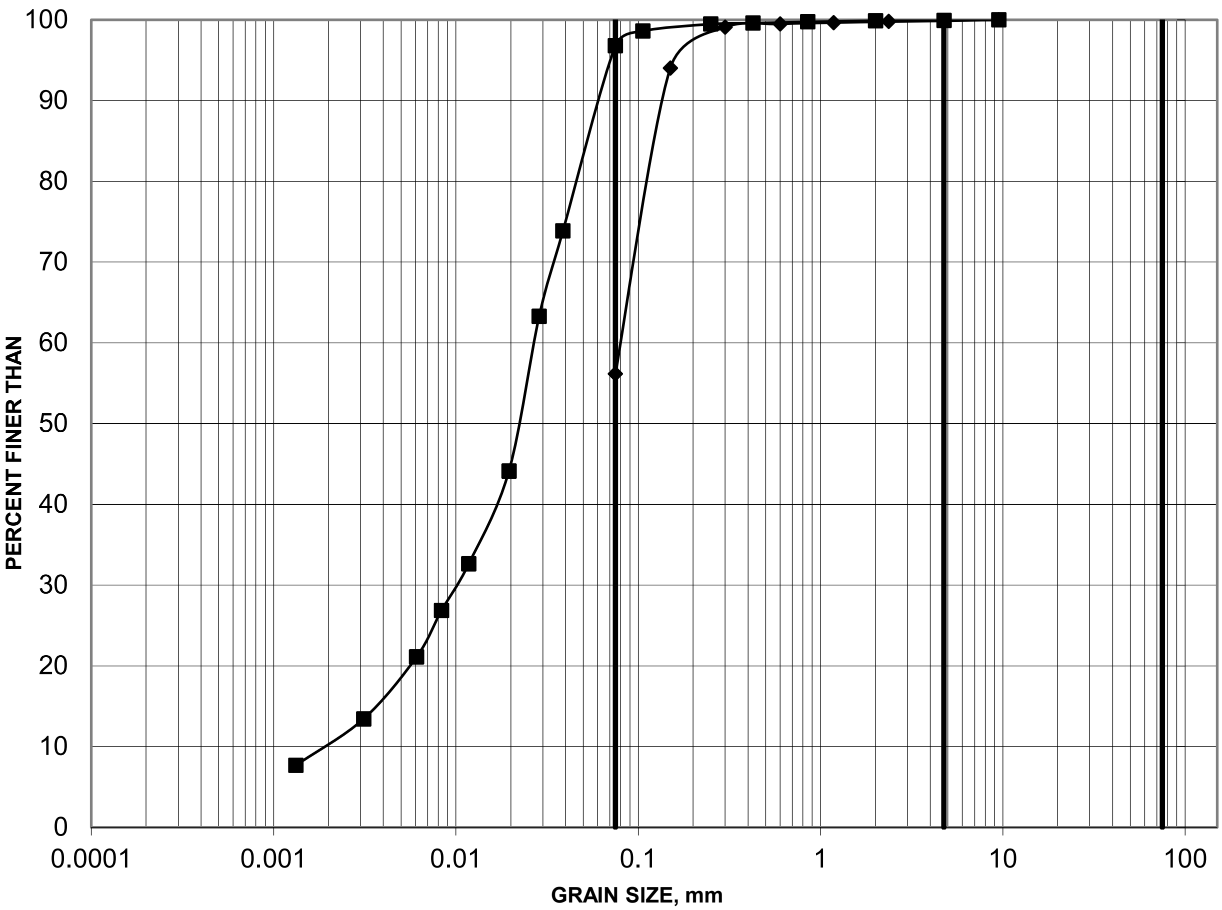
Created by:	KG
Checked by:	KCP

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab/7-Culvert 474/Figures/

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILT (ML)



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

	Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
■	474-22-03	7	4.57-5.18	0	3	87	10
◆	474-22-04	11	7.62-8.23	0	44	56	

Project: 1773612_WO 11

GOLDER
MEMBER OF WSP

Created by:	KG
Checked by:	KCP

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab/7-Culvert 474/Figures/

APPENDIX C

Analytical Laboratory Test Results

Certificate of Analysis

eurofins | Environment Testing

Client:	Golder Associates Ltd (Ottawa) 1931 Robertson Road, Ottawa, Ontario .	Report Number:	1985544
		Date Submitted:	2022-09-07
		Date Reported:	2022-09-15
		Project:	1773612-W011
		COC #:	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:

Emma-Dawn Ferguson 2022.09.15 12:07:37 -04'00'

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			8.86	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

Certificate of Analysis

eurofins | Environment Testing

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

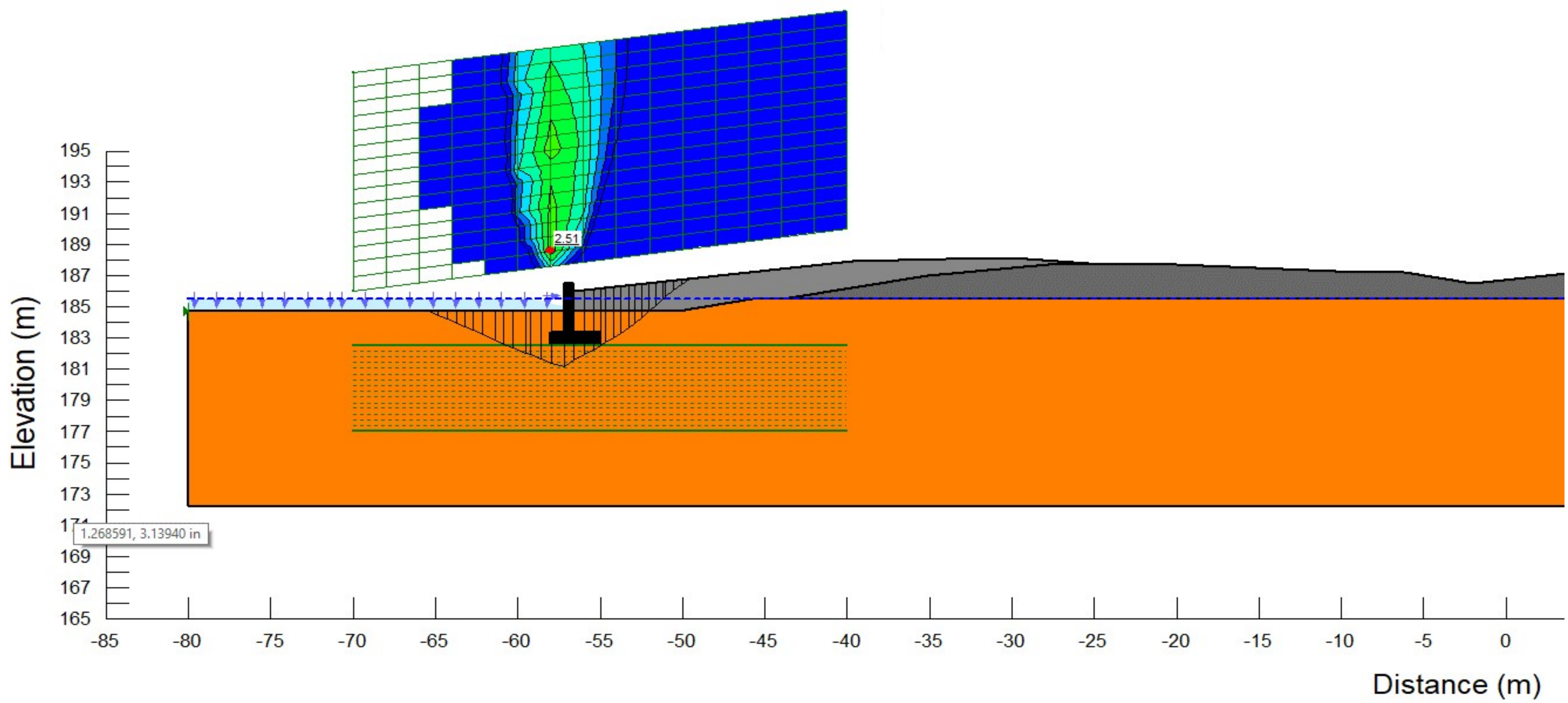
MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired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

APPENDIX D

Global Stability Analysis

Name: 1-Proposed Side Slope CV474 with RW - Static North Embankment
Analysis Type: Morgenstern-Price
Groundwater Elev.185.5 m
Direction of movement: Right to Left
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 (Generlly Compact to Dense)	Mohr-Coulomb	19	30	1



wsp

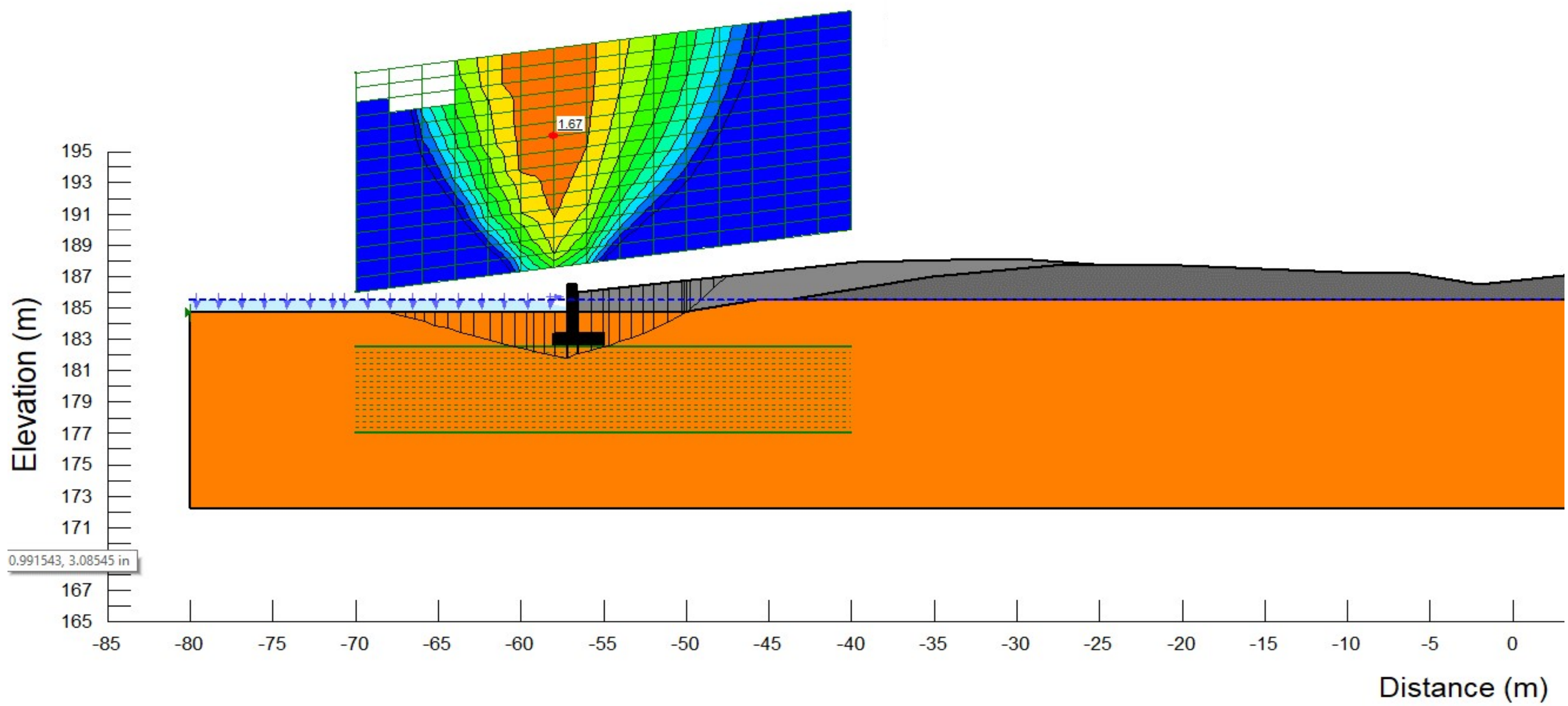
Replacement of Highway 401 Culvert - Structure Site No. 21X-0474/C0
Cramahe Township, Northumberland County, Ontario
MTO GWP 4054-17-00; Agreement No. 4016-E-0034-11
Global Stability - North Embankment Widening and Retaining Wall - Static Analysis

Project No:	1773612
Drawn:	BW
Date:	March 6, 2024
Checked:	KCP
Review:	LCC

FIGURE D1

Name: 2-Proposed Side Slope CV474 with RW - Siesmic North Embankment
Analysis Type: Morgenstern-Price
Groundwater Elev.185.5 m
Direction of movement: Right to Left
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 (Generlly Compact to Dense)	Mohr-Coulomb	19	30	1



Replacement of Highway 401 Culvert - Structure Site No. 21X-0474/C0
Cramahe Township, Northumberland County, Ontario
MTO GWP 4054-17-00; Agreement No. 4016-E-0034-11
Global Stability - North Embankment Widening and Retaining Wall - Seismic Analysis

Project No:	1773612
Drawn:	BW
Date:	March 6, 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="8">TEMPORARY FLOW PASSAGE SYSTEMS</th></tr>
<tr><th colspan="8">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">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></td></tr>
</table>

<table>
<tr><th colspan="4">DEWATERING SYSTEMS</th></tr>
<tr><th>Site Name / Station Reference</th><th>Preconstruction Survey Distance (m) (Note 4)</th><th>Minimum Lowered Groundwater Depth Below Base of Excavation or Work Area (m) (Note 5)</th><th>Design Engineer Requirements (Note 2)</th></tr>
<tr><td>Culvert 21X-0474/C0</td><td>N/A</td><td>1 m</td><td>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 native subgrade soils as inferred to be present within Borehole 474-22-03. The extent and depth of obstructions may vary beyond and between the borehole locations.

The Contractor is also alerted to the potential for artesian groundwater conditions as observed in the standpipe piezometer installed in Borehole 474-22-01.

Consideration of the presence of these obstructions and potential artesian conditions must be made in selection of appropriate equipment and procedures for temporary works and/or construction/installation of the culvert foundation, as may be required.

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