FINAL REPORT

Foundation Investigation and Design Report

Replacement of Structural Culvert 21X-0472/C0
Highway 401, Station 18+935 Cramahe Township, Northumberland County
MTO GWP 4054-17-00, Agreement No. 4016-E-0034

Submitted to:

Ministry of Transportation Ontario
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FOUNDATION INVESTIGATION REPORT

REPLACEMENT OF STRUCTURAL CULVERT 21X-0472/C0

HIGHWAY 401, STATION 18+935 CRAMAHE TOWNSHIP, NORTHUMBERLAND COUNTY

MTO GWP 4054-17-00, AGREEMENT NO. 4016-E-0034

1.0 INTRODUCTION

WSP Canada Inc. (WSP, formerly Golder Associates Ltd., acquired by 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-0472/C0. The foundation investigation services for this project have been delivered under MTO Agreement No. 4016-E-0034 Assignment #11 as part of GWP 4054-17-00.

2.0 SITE DESCRIPTION

The existing Culvert 21X-0472/C0 is located on Highway 401 approximately 4.5 km west of County Road 30, at about Station 18+935 in Cramahe Township in Northumberland County. The site location is shown on the key plan in Drawing 1. For the purpose of this report, Highway 401 is oriented in a west-east direction with the culvert positioned on a skew to the highway in a northwest-southeast orientation; for simplicity, the culvert is described as being oriented in a north-south direction.

The existing culvert, which was constructed in 1958, consists of an 87.0 m long, 4.3 m span, 2.4 m high (interior dimensions) reinforced concrete box structure that carries creek flow from south to north below all lanes of Highway 401, on an approximately 25° skew to the highway. According to the original contract drawings (Contract No. 58-278, Plan & Profile STA 270+00 to STA 300+00, WP No. 127-57), the existing culvert inlet and outlet were to be constructed at Elevations 169.4 m and 169.0 m respectively. It is understood that the culvert is in good to fair condition but is close to its 75-year design service life.

Based on the Highway 401 Plan and Profile Drawing, Station 270+00 to Station 300+00, WP No. 127-57 dated July and September 1958, the original watercourse meandered at this site with the channel generally located west of the existing culvert (at approximately Station 293+90), and the watercourse was then realigned through the current culvert following its construction (see Figure 1 on the following page and a copy of this drawing in Appendix D following the text of this report). The original watercourse channel was at approximately Elevation 170.2 m (558.5 ft.) at the centreline of Highway 401.

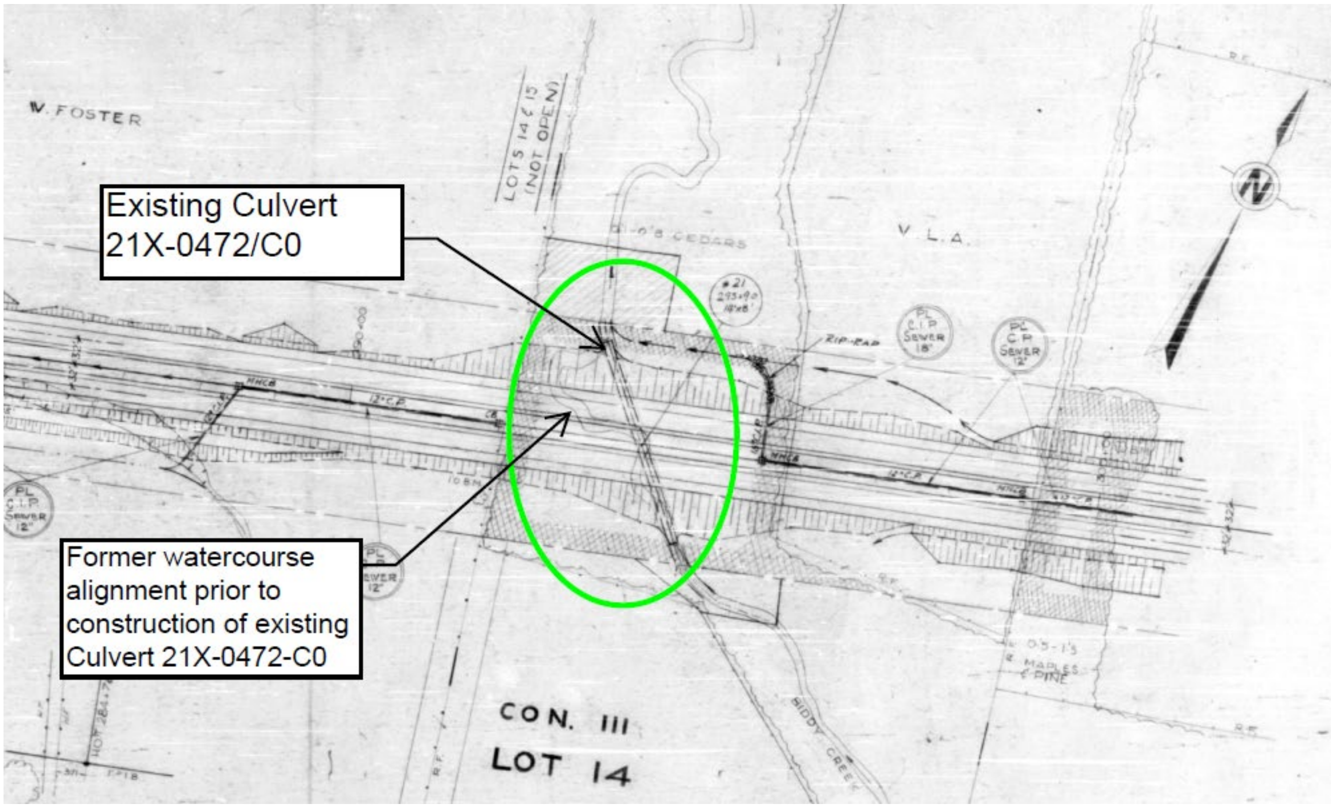

Figure 1: Original watercourse alignment relative to existing Culvert 21X-0472/C0 (from Highway 401 Plan and Profile Drawing, Station 270+00 to Station 300+00, WP No. 127-57 dated July and September 1958 – see Appendix D).

At the culvert location, Highway 401 has an existing four-lane cross-section with paved shoulders separated by a paved median and a tall concrete barrier wall. Steel beam guide rails are located on both outside shoulders of the highway in the vicinity of the culvert. The Highway 401 grade at the site ranges from approximately Elevation 179.7 m (WBL) to 180.0 m (EBL). The EBL and WBL embankments are up to about 9 m high relative to the surrounding ground surface in the watercourse valley which is at approximately Elevation 170.5 m to 171.5 m, with the existing embankment side slope inclined at about 3 horizontals to 1 vertical (3H:1V).

The area immediately surrounding the stream is vegetated with trees, shrubs, and brush both upstream and downstream of the culvert, and the surrounding lands are farmed.

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 or significant settlement.

Site photographs showing the general conditions at the site, along the highway, and at the inlet and outlet, are presented in Appendix D.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out over a four-day period between June 28 and October 17, 2022, and included advancing four boreholes (472-22-01 to 472-22-04) in the general location of the proposed culvert alignment. The borehole locations are shown on Drawing 1.

Boreholes 472-22-01 and 472-22-04, which are located near the north and south culvert ends, respectively, were advanced using a track-mounted Multipower limited access (LAD) drill rig with 165 mm outer diameter hollow stem augers. Boreholes 472-22-02 and 472-22-03, which are located on the Highway 401 platform, were advanced using a truck-mounted CME 55 drill rig with 200 mm 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 driven by an automatic hammer in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). Soil samples were obtained at vertical sampling intervals of about 0.76 m and 1.5 m.

A monitoring well was installed at Borehole 472-22-01 to observe the groundwater level at the site. The monitoring well consists of a 52 mm outside diameter PVC tube with a 1.5 m long slotted screen. Well installation details are shown on the record for Borehole 472-22-01 provided in Appendix A. The boreholes without monitoring well were backfilled with bentonite mixed with soil cuttings, in general accordance with the intent of Ontario Regulation (O.Reg.) 903, as amended. The site conditions were restored following completion of the field work.

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

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

The borehole locations and elevations were surveyed by WSP using a Trimble R10 GPS unit referenced to the NAD83 CSRS CBNv6-2010.0 MTM Zone 9 geodetic datum. The Trimble R10 GPS data have a vertical accuracy of approximately 0.1 m and a horizontal accuracy of approximately 0.5 m in accordance with the requirements of MTO's Guideline for Foundation Engineering Services (Version 3.0). The borehole locations, including northing and easting coordinates, ground surface elevations, and drilled depths are summarized in Table 1.

Table 1: Summary of Borehole Locations

Borehole	NAD83 CSRS CBNv6-2010.0 MTM Zone 9		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude (°))	Easting (m) (Longitude (°))		
472-22-01	4880364.9 (44.055670)	199153.0 (-77.818550)	171.0	6.7
472-22-02	4880342.1 (44.055460)	199177.6 (-77.818240)	179.7	14.3
472-22-03	4880322.2 (44.055290)	199198.7 (-77.817980)	180.0	14.0
472-22-04	4880300.2 (44.055090)	199221.3 (-77.817690)	171.5	9.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The culvert lies at the boundary of the physiographic regions known as the Iroquois Plain and South Slope, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). The Iroquois Plain physiographic region extends around the western part of Lake Ontario, from Niagara River to Trent River. The width of the plain varies from a few hundred meters to approximately 13 km north of the Lake Ontario shoreline, and it extends inland to include a large area in the Trent River valley. 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.

The South Slope region lies between the Oak Ridges Moraine, to the north and the Iroquois Plain to the south. It covers approximately 940 square miles, extending from Niagara Escarpment to the Trent River. The eastern portion of the slope in Northumberland County is thickly covered by large drumlins pointing to the southwest. In Northumberland County, a shallow deposit of fine sand and silt can be found on the surface of the till. The South slope generally lies across the limestones of the Verulam and Lindsay Formations, the grey shales of the Georgian Bay Formation, and the reddish shales of the Queenston Formation.

Based on geological mapping by the Ministry of Northern Development and Mines (MNDM), 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 Shadow Lake Formation.

4.2 Subsurface Conditions

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 results of the geotechnical laboratory are also presented in Appendix B. The results of the in-situ field tests, 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 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.

At the borehole locations, the subsurface conditions generally consist of the existing pavement structure (asphalt and pavement granular material) at boreholes advanced on the highway, or topsoil/peat at boreholes advanced at the culvert ends, underlain by a series of interlayered deposits consisting of a generally compact to dense upper deposit of silty sand to sand, underlain by a till deposit that varies in composition from loose to dense silt and sand to silty sand to stiff to hard sandy clayey silt , underlain by a lower deposit of compact to dense silty sand to sand. A more detailed description of the overburden soil deposits encountered during the field investigation is provided in the following sections.

4.2.1 Topsoil

An approximately 100 mm thick layer of topsoil was encountered at the ground surface (i.e., Elevations 171.0 m and 171.5 m) at Boreholes 472-22-01 and 472-22-04, which were advanced near the proposed north and south culvert ends, respectively.

4.2.2 Pavement Structure and Embankment Fill

An approximately 200 mm and 300 mm thick layer of asphalt was encountered at ground surface (i.e., Elevations 179.7 m and 180.0 m) at Boreholes 472-22-02 and 472-22-03, respectively, which were drilled through the outside shoulders of Highway 401. The pavement structure fill was encountered below the asphalt at Elevations 179.5 m and 179.7 m with thickness of 0.7 m and 0.5 m at Boreholes 472-22-02 and 472-22-03, respectively.

Embankment fill consisting of silty sand, gravelly silty sand to sandy silt, and sand was encountered below the topsoil and pavement structure at Boreholes 472-22-02, to 472-22-04. The top of this layer was encountered at elevations ranging from 171.4 m to 179.2 m. The total thickness of the fill layer ranges from about 1.9 m to 6.7 m in the boreholes. The Standard Penetration Test (SPT) N-values measured within the embankment fill range from 4 blows to 110 blows per 0.3 m of penetration, but more typically about 35 to 54 blows indicating a generally dense to very dense state of compactness.

The measured water contents of four samples of the granular fill ranged from 6% to 8%. The results of grain size distribution testing carried out on four samples of the silty sand to sandy silt fill material are provided in Figure B1 in Appendix B.

4.2.3 Peat

A layer of fibrous peat was encountered below the embankment fill at Borehole 472-22-04. The top of this deposit was encountered at Elevation 169.5 m, and it has a thickness of 100 mm.

4.2.4 Upper Silty Sand (SM) to Sand (SP/SW)

An upper silty sand to sand deposit was encountered below the embankment fill at Boreholes 472-22-02 and 472-22-03 and below the peat at Borehole 472-22-04. The top of this layer was encountered at elevations ranging from 169.4 m to 172.8 m. The total thickness of this layer ranges from about 1.0 m to 1.9 m. The SPT N-values within the silty sand to sand layer ranged from 10 blows to 35 blows per 0.3 m of penetration, indicating a compact to dense state of compactness.

The measured water content of one tested sample of the silty sand to sand was 18%. The results of grain size distribution testing carried out on two samples of this material are provided in Figure B2 in Appendix B.

4.2.5 Non-Cohesive Till

A non-cohesive till deposit was encountered below the topsoil at Borehole 472-22-01 and below the silty sand to sand layer at Borehole 472-22-02; the top of this layer was encountered at Elevations 170.9 m and 171.0 m in these boreholes, and the total thickness of this till layer is 1.8 m and 3.3 m. This glacial till is described as silt and sand to silty sand to sandy silt containing trace amounts of clay and gravel, as well as cobbles and boulders. A layer of gravelly sand till containing some silt and cobbles and boulders was also encountered below cohesive till at Borehole 472-22-03. The top of this layer was encountered at Elevation 166.9 m, and the borehole was terminated upon reaching target depth after penetrating this layer for about 0.9 m.

The SPT N-values within this till layer ranged from 4 blows to 30 blows per 0.3 m of penetration, indicating a loose to dense state of compactness. The SPT N-value in the gravelly sand till was 72 blows per 0.3 m of penetration, representing a very dense condition.

The measured water contents of three tested samples of non-cohesive till ranged from 11% to 14%. The results of grain size distribution testing carried out on three samples of the silt and sand to silty sand till material are provided in Figure B3 in Appendix B. The results of Atterberg limits testing completed on a single sample of the silt and sand to silty sand till indicate a liquid limit of 13%, plastic limit of 11% and plasticity index of 2. The Atterberg Limits test results are provided on Figure B4 in Appendix B and indicate that the fines portion of this till is a silt of low plasticity (ML).

4.2.6 Cohesive Till

A deposit of sandy clayey silt till was encountered below the upper silt and sand at Boreholes 472-22-03 and 472-22-04; this cohesive till may represent a gradation from the non-cohesive till as described above. The cohesive till deposit contains varying amounts of gravel and cobbles and boulders. The top of this cohesive till layer was encountered at Elevations 168.4 m and 170.9 m, with a total thickness of about 2.4 m to more than 6.7 m; Borehole 472-22-04 was terminated in this layer at an Elevation of 161.7 m.

The SPT N-values within this till layer ranged from 11 blows to 80 blows per 0.3 m of penetration, but typically about 18 blows to 35 blows indicating a generally very stiff to hard consistency.

The measured water contents of four samples of this sandy clayey silt till ranged from 10% to 20%. The results of grain size distribution testing carried out on six samples of this till material are provided in Figure B5 in Appendix B. The results of Atterberg limits testing completed on five samples of the cohesive till indicate liquid limits ranging from 14% to 17%, plastic limits ranging from 10% to 12% and plasticity indices ranging from 4 to 5. The Atterberg Limits test results are provided on a plasticity chart on Figure B6 in Appendix B and confirm the cohesive portion of the till is a clayey silt-silt of low plasticity (CL-ML).

4.2.7 Lower Silty Sand (SM) to Sand (SP/SW)

A lower deposit of silty sand to sand with varying amounts of gravel was encountered below the non-cohesive till at Boreholes 472-22-01 and 472-22-02. The top of this layer was encountered at Elevations 167.7 m and 169.2 m. Boreholes 472-22-01 and 472-22-02 were both terminated in this layer at Elevations 164.3 and 165.4 m upon reaching the target depth for the boreholes.

The SPT N-values within the silty sand to sand layer ranged from 9 blows to 44 blows per 0.3 m of penetration, but more typically 23 blows to 44 blows indicating a generally compact to dense state of compactness.

The measured water contents of two tested samples of the silty sand to sand were 15% and 17%. The results of grain size distribution testing carried out on one sample of this material is provided in Figure B2 in Appendix B. The results of Atterberg limits testing completed on a single sample of the silty sand to sand indicate that the fines portion of this material is non-plastic.

4.3 Groundwater Conditions

A standpipe piezometer was installed at Borehole 472-22-01 to measure the groundwater level at the site. The groundwater level recorded in the piezometer is shown on the borehole record in Appendix A and is summarized in Table 2. The measured water levels indicate artesian conditions are present at the site.

Table 2: Summary of Groundwater Conditions

Borehole	Screened Interval	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Groundwater Elevation (m)	Date
472-22-01	Sand	171.0	-0.4[1]	171.4	December 14, 2022
			-0.9[1]	171.9	May 16, 2023

Note:
1. Negative readings indicate height above existing grade i.e., artesian groundwater conditions.

A higher water level (at Elevation 175.3 m) was measured in the open Borehole 472-22-02 (which was drilled through the Highway 401 embankment) upon completion of drilling; this may not represent the stabilized water level at this location, but it does indicate that water-bearing soils and artesian conditions should be expected at the site.

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

4.4 Analytical Laboratory Testing Results

One soil sample was submitted to Eurofins for chemical testing/analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The test results are provided in Appendix C and are summarized in Table 3.

Table 3: Summary of Analytical Test Results for Steel Corrosion and Sulphate Attack Parameters

Borehole	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (m^S/cm)	pH	Resistivity (ohm-cm)
472-22-04	0.8-1.4	0.014	0.06	0.55	8.15	1,818

5.0 CLOSURE

This Foundation Investigation Report was prepared by Kinjal Gajjar, a geotechnical consultant and reviewed by Kenton Power, P.Eng., a senior geotechnical engineer. Lisa Coyne, P.Eng., a Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP, conducted an independent technical and quality review of this report.

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https://golderassociates.sharepoint.com/sites/11407g/wo11 colborne to brighten/3. reporting/5-culvert 472/3-final/gwp 4054-17-00 final fidr rev0 21x-0472c0 2024-03-11 (1773612).docx

PART B

FOUNDATION DESIGN REPORT

REPLACEMENT OF STRUCTURAL CULVERT 21X-0472/C0
HIGHWAY 401, STATION 18+943 CRAMAHE TOWNSHIP, NORTHUMBERLAND COUNTY
MTO GWP 4054-17-00, AGREEMENT NO. 4016-E-0034

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for the detailed design of replacement of Culvert 21X-0472/C0. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced as part of the current investigation and the design information in the General Arrangement drawing provided by WSP.

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

6.2 Project Understanding

It is understood that Highway 401 is to be rehabilitated and widened from the existing four-lane configuration to a proposed interim six-lane configuration and ultimate eight-lane configuration (i.e., interim three lanes then ultimate four lanes in each direction). The existing grade on Highway 401 will be maintained at this site. This interim configuration will require approximately 5 m to 6 m of embankment widening to the outside for both the WBL and EBL. This interim widening will requirement placement of up to approximately 2 m (vertical thickness) of fill atop the existing embankment side slope on the north side of the WBL embankment, and up to approximately 4 m of fill on the existing embankment side slope on EBL, with nominal regrading in the centre 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 (i.e., a total thickness of up to approximately 3 m on the north side and 5 m on the south side of the highway embankment).

The proposed culvert is to be installed along a new alignment approximately 8 m east of the existing culvert at Highway 401 Station 18+944. Based on the Preliminary General Arrangement drawing dated August 2022, the replacement culvert will be approximately 104 m long to accommodate the ultimate eight-lane highway configuration. Based on the Highway 401 right-of-way limits and topography, concrete headwalls and retaining walls will be 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, with temporary extensions as required to accommodate the embankment widening. 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.

Once construction of the new culvert is complete, the existing concrete culvert can be decommissioned by removal or by abandoning in place via grouting up the culvert. Based on the construction staging including construction of the replacement culvert on a new alignment, it is anticipated that abandonment by grouting in place will be the preferred solution, although it may also be feasible to remove a portion of the existing culvert during the second stage of construction. However, it is noted that culvert construction staging, timing for decommissioning of the existing culvert and maintenance of flow will need to be confirmed as part of the future detail design.

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 11 following the text of this report.

- Multiple pipe culverts would likely be required to provide a similar flow-through capacity compared to an open-footing or closed-bottom box culvert option. Further, if constructed from steel, pipe culverts will likely have a shorter design life compared to concrete structures.
- A closed-bottom concrete box culvert can be formed of pre-cast segments that can be placed more expeditiously compared to a cast-in-place option, offering schedule advantages with respect to construction/traffic staging and dewatering. Concrete boxes can typically be founded at a shallower level compared to open footing culverts, reducing excavation and dewatering requirements compared to the open footing 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 and would most likely be 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 GA drawing, it is understood that a precast reinforced concrete box culvert has been selected as the preferred replacement type. The culvert will have a 4.8 m span and 2.4 m inside height based on hydraulic requirements, with the invert varying from approximately Elevation 169.1 m at the south (inlet) end to Elevation 168.7 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 CHBDC have been used for design.

For seismic design, the consequence factor, ψ, and resistance factor, ϕ_{gu}, should be taken as unity, and the geotechnical resistance factor shall be as specified in Table 6.3. as per Section 6.14.4 of CHBDC (2019).

6.4.2 Seismic Design

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

6.4.2.1 Seismic Site Classification

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

6.4.2.2 Spectral Response Values

In accordance with Section 4.4.3.1 of the CHDBC and based on the location of the proposed structure, the Class D peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca) are provided in Table 4.

Table 4: Site Class D Spectral Values for Subject Site

Parameter	2% Probability of Exceedance in 50 Years (2,475-year return period) (g)
PGA	0.204
Sa (0.2)	0.351
Sa (0.5)	0.334
Sa (1.0)	0.199
Sa (2.0)	0.0951
Sa (5.0)	0.0254
Sa (10.0)	0.00796
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 site is underlain by a deposit of compact to dense silty sand to sand, underlain by a loose to dense silt and sand till and compact to dense/ stiff to hard sandy clayey silt to silt and sand till, underlain by a compact to dense silty sand to sand. 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). Footings constructed at this site would require a minimum embedment depth of 1.4 m below final finished grade for frost protection purposes. However, if a box culvert is constructed it is not necessary to ensure that the full length of the replacement culvert is founded below the frost depth, as box culverts are tolerant of small magnitudes of movement related to freeze-thaw cycles.

6.5 Culvert Foundation Design Recommendations

6.5.1 Culvert Subgrade Preparation

Prior to placing the bedding/levelling course for box culvert sections or the concrete working slab for an open footing, 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 foundation footprint be sub-excavated and replaced with Ontario Provincial Standard Specification, Provincial Oriented (OPSS.PROV) 1010 Granular A or Granular B Type II fill; Granular B Type II fill is recommended for placement in wet conditions.

6.5.2 Box Culvert Bedding and Levelling Layer Requirements

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

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

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

6.5.3 Box Culvert Founding Level and Factored Axial Geotechnical Resistances

Based on a 650 mm thick concrete bottom slab and the bedding and levelling layer thicknesses recommended above, the founding subgrade level for the replacement culvert will be at approximately Elevation 168.2 m at the south (inlet) end and Elevation 167.8 m at the north (outlet) end. For the proposed box culvert within an overall footprint width of 5.8 m (exterior dimension) founded on the properly prepared granular bedding/levelling course overlying the native soils at the above-noted elevations, the following factored geotechnical resistances may be used for design:

- Factored ultimate geotechnical resistance: 450 kPa
- Factored serviceability geotechnical resistance (for 25 mm of settlement): 250 kPa

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

6.5.4 Open Footing Culvert Founding Level and Factored Axial Geotechnical Resistances

Strip footings should be placed on the properly prepared native subgrade soils below the frost penetration depth. Based on the invert elevations as summarized in Section 6.3 (Elevation 169.1 m to Elevation 168.7 m from south to north), the footings should be founded at about Elevation 167.7 m to 167.3 m to provide a minimum 1.4 m of soil cover for frost protection. If precast footings are utilized, it is recommended that a minimum 150 mm thick bedding layer and 75 mm thick levelling layer (as discussed in Section 6.5.2) be incorporated directly below the underside of the footings to facilitate their placement.

For 1.2 m wide footings founded on the properly prepared native soils at Elevation 167.6 m to 167.3 m, the following factored geotechnical resistances may be used for design:

- Factored ultimate geotechnical resistance: 225 kPa
- Factored serviceability geotechnical resistance (for 25 mm of settlement): >225 kPa

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

6.5.5 Retaining Wall Founding Level and Factored Axial Geotechnical Resistances

Retaining walls are required at the upstream and downstream ends of the replacement culvert 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 5 m relative to the ground surface in front of the wall.

Based on the borehole results, the retaining wall footings should be founded at or below the elevations given in Table 5 to extend below existing fill or loose material; the footings may need to be founded deeper to achieve a minimum depth of 1.4 m below lowest surrounding grade to provide adequate protection against frost penetration, depending on the regrading associated with the new watercourse channelization. Table 5 provides the factored ultimate and serviceability geotechnical resistances to be used for design assuming that the retaining walls footings are founded at least 1.4 m below lowest surrounding grade.

Table 5: Factored Ultimate and Serviceability Geotechnical Resistances for Retaining Wall Foundations

Retaining Wall Area	Highest * Founding Elevation (m)	Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa)
South (Inlet)	168.5	2	275	>275
		3	325	325
North (Outlet)	169.0	2	275	>275
		3	325	325

* This represents the highest founding elevation; footings may need to be founded deeper to provide a minimum depth of 1.4 m below lowest surrounding grade for frost protection purposes.

The factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with 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:

Table 6: Interface Friction Angles and Shear Strengths

Interface	Interface Strength
Between pre-cast concrete and underlying granular bedding/levelling layer	$\delta'_i = 20°$, $c'_i = 0$ kPa
Between the granular bedding layer and underlying silt and sand to silty sand till	$\phi' = 32°$, $c' = 0$ kPa
Between cast-in-place retaining wall footings and native silty sand to sand or clayey silt till	$\phi' = 32°$, $c' = 0$ 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 and compacted to not less than 98% of the material's SPMDD in accordance with OPSS.PROV 501 (Compacting). The fill should also be placed concurrently on both sides of the culvert, ensuring that the backfill depth on one side does not exceed the other side by more than 400 mm as per OPSS.PROV 422 (Precast Reinforced Concrete Box Culverts). Embankment restoration after completion of the culvert replacement should be carried out in accordance with OPSS.PROV 206.

6.5.8 Culvert Erosion and Scour Protection

To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles which could lead to the formation of sinkholes), consideration should be given to the use of a concrete cut-off wall and/or clay seal. Based on the GA drawing, it is understood that concrete 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 addition to the cut-off walls, the clay material should meet the requirements of OPSS.PROV 1205 (Clay Seal), and the seal should be a minimum of 1 m thick, whether constructed of natural clay or soil-bentonite mix. Alternatively, a geosynthetic clay liner (GCL) may be incorporated, and this is generally considered the preferred alternative as it is much thinner (only a few millimeters thick) than the standard natural clay or soil-bentonite layer, thus requiring a shallower excavation into the slope, and is much easier to install. The clay seal or GCL should extend a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and from a depth of 1 m below the scour level up to a minimum vertical height on the embankment side slopes equivalent to the high-water level. If a GCL is utilized, the GCL should be constructed within the embankment slope to allow for a minimum 0.3 m thick granular (embankment) fill cover to be placed over the GCL to provide protection from the requisite overlying erosion protection material. Rip-rap/rock fill slope protection material should be placed on the granular cover layer and not directly on the GCL.

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

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

6.6 Lateral Earth Pressures

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

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

- Select, free draining, non-frost susceptible granular fill meeting the requirements of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ (Type I or II) should be used as backfill behind the culvert walls and associated headwalls and retaining walls, as well as on top of the culvert for a minimum thickness of 300 mm in a similar configuration to that shown in OPSD 803.010 (Backfill and Cover for Concrete Culverts).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with the Section 6.12.3 and Figure 6.8 of CHBDC (2019). Hand-operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, the granular fill should be placed in a zone with the width equal to at least 1.4 m behind the back of the wall (see Figure C6.31(a) of the Commentary to CHBDC (2019)). For unrestrained walls, the fill should be placed within the wedge-shaped zone defined by a line drawn flatter than 1H:1V extending up and back from the rear face of the footing (see Figure C6.31(b) of the Commentary to CHBDC). However, where side slopes inclined at 3H:1V or flatter are required for open-cut excavations extending below the groundwater level, in accordance Ontario Regulation 213, Ontario Occupational Health and Safety Act (OHSA) for Construction Projects (as amended), consideration could be given to backfilling the full open-cut excavation area with OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ (Type I or II) in order to

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

The parameters and lateral earth pressure coefficients in Table 7 may be used in the design of culvert walls, headwalls and retaining walls. The lateral earth pressure coefficients provided in Table 7 have been developed for flat (i.e., non-sloping) ground above/behind the culvert walls, as well as for a 2H:1V slope condition for unrestrained walls as applicable for the retaining walls at the ends of the replacement culvert. If the inclination of the slope above the wall differs, revised lateral earth pressures parameters will need to be calculated in accordance with CHBDC Clause C6.12.1, Figures C6.28 (active earth pressure) and C6.29 (passive earth pressure), and Clause C6.12.2.2 (at-rest earth pressure).

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

If the wall allows lateral yielding (i.e., unrestrained structure), active earth pressures should be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.12 of the Commentary to CHBDC (2019).

Table 7: Lateral Earth Pressure Coefficients

Wall Movement Condition	Restrained Wall	Unrestrained Wall			
Fill Material	Existing or New Embankment Fill Behind Granular Backfill, Φ'=32°	Granular A and B Type II Φ'=36°		Granular B Type I Φ'=32°	
Unit Weight (kN/m^3)	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 materials meeting the specifications of OPSS.PROV 1010 Granular A or Granular B Type I or Type II or Select Subgrade Material. Fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading).

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

6.7.2 Global Stability of Widened Embankment Including Retaining Walls

The existing Highway 401 eastbound and westbound embankments are up to approximately 8.5 m to 9.0 m in height relative to the surrounding ground surface. Based on the GA drawing, it is understood that the existing embankment heights at the 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. Retaining walls up to approximately 5 m in height will be required adjacent to the north and south ends of the culverts.

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

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

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

- The soil stratigraphy was based on Profile A-A' shown in Drawing 1 following the text of this report, with the footing founding level per Section 6.5.5. of this report.
- The groundwater level was assumed to be at Elevation 172 m, at or above the ground surface beyond the embankment footprint.
- A seismic horizontal loading of 0.102g, equal to one-half of the site-specific PGA value (0.5 of 0.204 g Site Class C) was used for seismic analysis (see Section 6.4.2.2. of this report).
- The retaining walls were assumed to have an average total height of approximately 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 E1 and E2 in Appendix E. 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 9 m to 10 m widening is proposed along the outside of the WBL and EBL embankments resulting in placement of up to approximately 3 m to 5 m of fill (vertical thickness) on the existing embankment side slopes.

To estimate the magnitude of the settlement as a result of the proposed embankment widening, analyses were carried out near the proposed new crest and the existing toe of the slope, where the highest-grade raise is anticipated to occur. The settlement analysis discussed below assumes that all organics within the footprint of the widened embankments will be sub-excavated and replaced prior to placement of any new embankment fill material for the widening.

The immediate compression of the native soil deposits was modelled based on typically accepted correlations with the obtained SPT ‘N’ values as presented in Bowles (1984) and by Kulhawy and Mayne (1990) together with engineering judgment based on experience in similar subsurface conditions. The unit weight and associated stiffness (moduli) are summarized in Table 8. The groundwater level was assumed to be at approximately Elevation 172.0 m (i.e., roughly at or above ground surface beyond the embankment toe).

Table 8:Unit Weight and Stiffness of Founding Soil Strata

Material	Unit Weight (kN/m^3)	Elastic Modulus (MPa)
Upper silty sand to sand – Generally compact	20	40 to 60
Silt and sand to silty sand till – Generally compact	20	40 to 60
Sandy clayey silt till – Very stiff to hard	21	60 to 90

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 less than approximately 25 mm at the existing and proposed crest of the embankment on the north side of the WBL and south side of the EBL. The noted magnitude of settlement is expected to be elastic and to occur during and immediately following 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 construction of the embankment depending on the type of material used. The magnitude of compression of granular fill, SSM or non-cohesive earth fill may range from 0.5% to 1% of the height of the embankment, assuming the embankment fill is placed and compacted in accordance OPSS.PROV 501 as outlined above in this case, settlement of the granular/non-cohesive fill itself is expected to occur essentially during embankment construction. Cohesive earth fill materials are not preferred for embankment construction as they may exhibit some additional settlement over time depending on their gradation, plasticity, and field compaction effort. Although not anticipated for this project, 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 post-construction settlement and differential settlement criteria in Table 9 are considered acceptable for settlements to occur within twenty years post-paving for the bridge approach embankments at this site.

Table 9: Post-Construction and Differential Settlement Criteria

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 during construction staging, or the proposed replacement culvert.

6.8 Corrosion Assessment and Protection

The analytical results for the soil samples submitted for testing are summarized in Section 4.4 and the analytical laboratory test reports are 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 A23.1-19 Section 4.1.1 "Durability Requirements" are followed when designing concrete elements, as applicable.

6.8.1 Potential for Sulphate Attack

The analytical test results summarized in Table 3 of this report were compared to CSA Standard, CAN/CSA-A23.1-19 Table 3 and Table 7.2 of MTO's Gravity Pipe Design Guidelines (2014), for potential sulphate attack on concrete. The sulphate concentrations measured in one tested sample was 0.01% and is below the exposure class of S-3 (Moderate). Therefore, based on the soil sample tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

6.8.2 Potential for Corrosion

The test results indicate a pH value of 8.2 and a resistivity of 1,818 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not detrimental to concrete durability. The resistivity indicates that the soil corrosiveness is Severe (2,000 ohm-cm > R), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014), and appropriate corrosion protection should be applied to the foundation element / materials. Further, given that the foundations are located adjacent to the highway and may be exposed to de-icing salt, consideration should be given to selection of a “C” type exposure class as defined by CSA A23.1 Table 1.

These recommendations are provided as guidance only; the designer should take the results of the laboratory testing into consideration for selecting and specifying appropriate materials and corrosion susceptibility for design service of the structure foundations and determine the appropriate exposure class and ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

6.9 Construction Considerations

6.9.1 Construction Staging and Temporary Roadway Protection

The temporary excavations for the culvert replacement will extend through the existing granular embankment fill and into the native subgrade soils. The granular fill and near-surface loose 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 above the water table (or following dewatering to below the base of the excavation) 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).

It is understood that two lanes of traffic will be maintained in each direction during construction, and that the lanes will initially be shifted toward the median to construct the outside portions of the culvert and the required embankment widening, after which traffic will be shifted to the outside to construct the central portion of the new replacement culvert. Temporary protection systems are expected to be required along Highway 401 between the stages. The protection systems could consist of either driven sheet piling or soldier piles and lagging where H-piles would be placed in pre-augered holes or driven to a suitable depth, with horizontal lagging installed as the excavation proceed. Support to the system could be in the form of struts, Wales, rakers, or anchors. Based on the encountered subgrade soil conditions and anticipated excavation requirements, a sheet pile shoring system would be considered more practical and more cost effective. The installation of sheet piles could potentially be impeded by the presence of cobbles and/or boulder obstructions, or the presence of woody vegetation below the existing embankment fill, but the risks are anticipated to be relatively low.

Although the contractor is responsible for the selection and detailed design of the temporary protection/dewatering systems, the parameters in Table 10 are provided to enable the detail designers to develop a conceptual design and assess the approximate construction costs for the protection systems.

Table 10: Static Lateral Earth Pressure Coefficients

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 (dense to very dense)	20	32	0.31	0.47	3.25
Silty Sand to Sand (compact too dense)	19	30	0.33	0.5	3.0
Silt and Sand Till (loose to dense)	19	32	0.31	0.47	3.25
Sandy Clayey Silt Till (Very stiff to hard)	22	34	0.28	0.44	3.53

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 to account for the fact that a large strain would be required for mobilization of the full passive resistance.

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

6.9.2 Control of Groundwater and Surface Water

It is anticipated that the creek flow will be maintained within the existing culvert while the replacement culvert is being constructed. However, given the permeable subgrade soils encountered at this site and the anticipated depth of the excavations (i.e., extending about 2 m below the measured groundwater level), a temporary dewatering system likely in conjunction with a cofferdam/cut-of system surrounding the culvert excavation is anticipated to be required to maintain a dry and stable subgrade.

Given the permeable nature of the silty sand to sand deposit at this site and the high groundwater table, an active dewatering system is expected to be required. The 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; this drawdown may be affected by the presence of cohesive till as encountered in the boreholes over the south portion of the culvert, and at these locations the dewatering system (or cut-off plus groundwater control) should draw the water level down to the surface of the cohesive till. The extent/depth of dewatering requirements shall be reviewed by the contractor, based on their proposed construction methods/ procedures.

An Environmental Activity Section Registry (EASR) is not required for the temporary surface water diversion through an existing culvert. However, where 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) will be required, depending on the groundwater conditions at the time of construction and estimated pumping volumes. If an interlocking sheet pile cut-off system or other form of cofferdam/cut-off is not implemented, it is estimated that the dewatering rate will exceed 400 m^3/day; therefore, it is recommended that a draft PTTW be obtained for this project to accommodate the construction of this culvert among the other components of the contract. The Contractor should 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 confirm their dewatering estimate and discharge plan.

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

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

6.9.4 Obstructions

The contractor should be alerted to the potential presence of cobble and boulder obstructions within the fill material, and within the glacially derived native soils at the site. A sample Notice to the Contractor is included in Appendix F.

7.0 CLOSURE

This Foundation Design Report was prepared by Kinjal Gajjar, a geotechnical consultant and reviewed by Kenton Power, P.Eng., a senior geotechnical engineer. Lisa Coyne, P.Eng., a Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP, conducted an independent technical and quality review of this report.

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Provincial Engineering Memorandum #20201, Material Engineering and Research Office (MERO), March 23, 2020.

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Ontario Provisional Standard Drawing:

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 Provincial Standard Specifications and Special Provisions:

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 517	Dewatering
SP 517F01	Dewatering
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 Regulations

Ontario Regulation 213	Construction Projects (as amended)
Ontario Regulation 903	Wells (as amended)

Table 11: Comparison of Alternatives Culvert Types

Foundation Option	Advantages	Disadvantages	Risk / 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.
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.

DRAWING

Drawing 1 – Borehole Location and Soil Strata

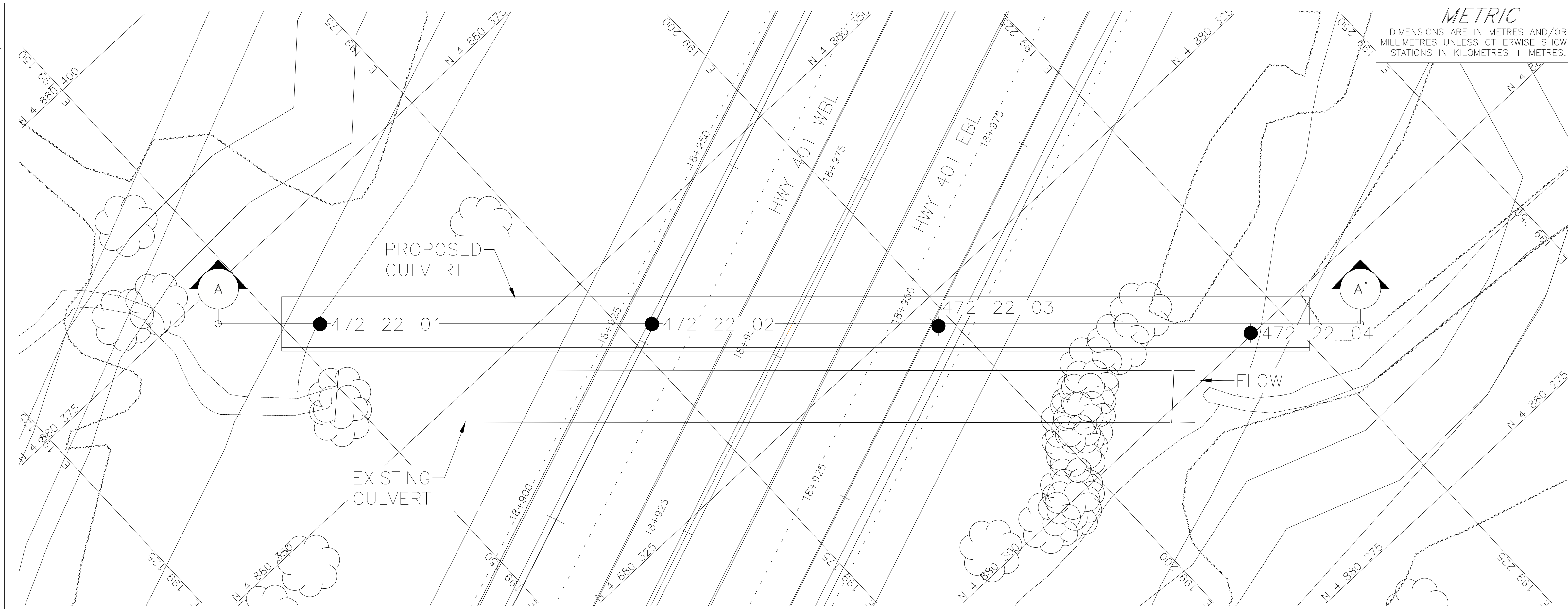
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4054-17-00

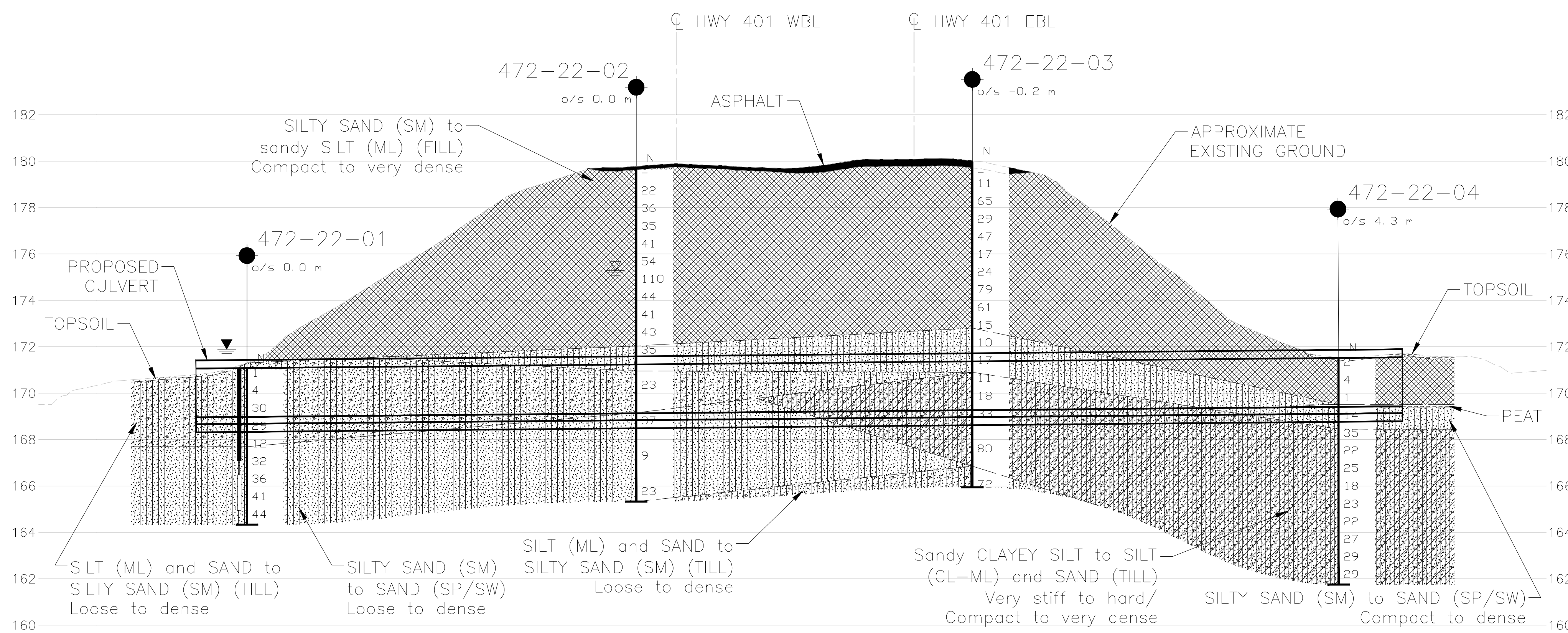
SHEET

HIGHWAY 401 WIDENING
REPLACEMENT OF CULVERT 21X-0472/C0
BOREHOLE LOCATIONS AND SOIL STRATA

WSP



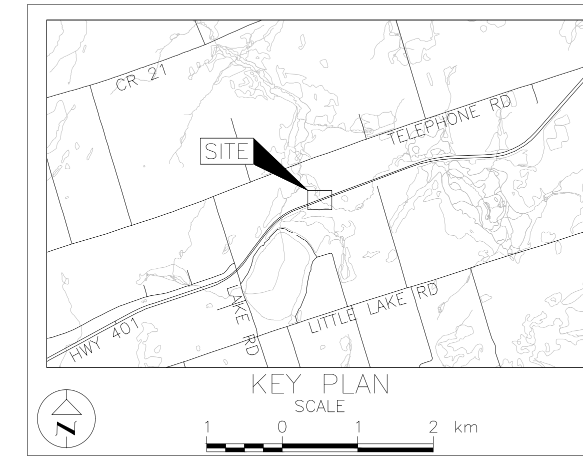
PLAN
SCALE
5 0 5 10 m



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
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on May 16, 2023
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES NAD 83 (CSRS)/MTM ZONE 9

No.	ELEVATION	NORTHING	EASTING
472-22-01	171.0	4880364.9	199153.0
472-22-02	179.7	4880342.1	199177.6
472-22-03	180.0	4880322.2	199198.7
472-22-04	171.5	4880300.2	199221.3

Structural Site Location Latitude: 44.05548 Longitude: -77.81836

NOTES

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

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

REFERENCE

Base plans provided in digital format by WSP, drawing file no. Mainline-8Lane proposed alignment for Culvert Sections_ACAD (updated – April 12 2022).dwg, received APR. 14, 2022.

NO.	DATE	BY	REVISION

Georcs No. 31C04-005

HWY. 401	PROJECT NO. 1773612		DIST. EASTERN
SUBM'D. KG	CHKD. KG	DATE: 04/17/2024	SITE: 21X-0472/C0
DRAWN: ZS	CHKD. KCP	APPD. LCC	DWG. 1

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
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

* 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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

Fresh (W1): no visible sign of rock material weathering.

Slightly Weathered (W2): discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

Moderately Weathered (W3): less than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Highly Weathered (W4): more than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Completely Weathered (W5): 100% of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

Residual Soil (W6): all rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90^{o} angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

AXJ Axial Joint
BD Bedding
BC Broken Core
CC Continuous Core
CL Closed
CO Contact
CU Curved
CT Coated
FLT Fault
FOL Foliation
FR Fracture
GO Gouge
IN Infilled
IR Irregular
JN Joint
KV Karstic Void
K Slickensided
LC Lost Core
MB Mechanical Break
PL Planar
PO Polished
RO Rough
SA Slightly Altered
SH Shear
SM Smooth
SR Slightly Rough
SY Stylolite
UN Undulating
VN Vein
VR Very Rough

ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250

Foundation Design

RECORD OF BOREHOLE No 472-22-01 SHEET 1 OF 1 METRIC

PROJECT 1773612
G.W.P. 4054-17-00 LOCATION N 4880364.9; E 199153.0 MTM NAD ZONE 9 (LAT. 44.055670; LONG. -77.818550) ORIGINATED BY JS
DIST Eastern HWY 401 BOREHOLE TYPE LAD Multipower Track Mounted, 165 mm OD Hollow Stem Augers COMPILED BY ZS/KG
DATUM GEODETIC DATE June 28, 2022 CHECKED BY KCP/LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
171.0	GROUND SURFACE						
0.0	TOPSOIL						
0.1	Brown Moist SILT (ML) and sand, trace gravel, contains cobbles and boulders (TILL) Loose Grey-brown Wet	1	SS	1			
		2	SS	4	170	~20	4 42 (54)
169.5 1.5	SILTY SAND to sandy SILT (SM/ML), some clay, some gravel, contains cobbles and boulders (TILL) Compact to dense Grey-brown Moist	3	SS	30	169		
		4	SS	29		~14	15 31 45 9
		5	SS	12	168		
167.7 3.4	Gravelly SAND (SW), trace silt Compact to dense Brown Wet	6	SS	32	167		
		7	SS	36	166	~14	22 73 (5)
165.8 5.2	SAND (SP), some silt Dense Brown, mottled Wet	8	SS	41	165		
		9	SS	44			
164.3 6.7	END OF BOREHOLE						

NOTES:

1. Artesian flow out of hollow stem augers was observed at 6.7 m depth during drilling. Drilling was halted and well installed. Groundwater level was at 0.4 m above ground surface (Elev. 171.4 m) immediately after well installation

2. Water level measured in standpipe piezometer.

Date	Depth(m)	Elev.(m)
Dec.14/22	-0.4	171.4
May 16/23	-0.9	171.9

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

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

Foundation Design

RECORD OF BOREHOLE No 472-22-02

SHEET 1 OF 2

METRIC

PROJECT	1773612		
G.W.P.	4054-17-00	LOCATION	N 4880342.1; E 199177.6 MTM NAD ZONE 9 (LAT. 44.055460; LONG. -77.818240)
DIST	Eastern	HWY	401
BOREHOLE TYPE	CME 55 Truck Mounted, 200 mm OD Hollow Stem Augers		
DATUM	GEODETIC	DATE	October 18, 2022
ORIGINATED BY	BW	COMPILED BY	ZS/KG
CHECKED BY	KCP/LCC		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
179.7	GROUND SURFACE
0.0	ASPHALT (150 mm)
0.2	Gravelly SILTY SAND to SAND and GRAVEL (SM) (PAVEMENT STRUCTURE) (FILL) Brown Moist
178.8 / 0.9	SILTY SAND (SM), some gravel, trace clay (FILL) Dense Brown Moist
177.6 / 2.1	Gravelly SILTY SAND (SM) to sandy SILT (ML), some gravel, trace clay (FILL) Dense to very dense Brown Moist
172.1 / 7.6	SAND (SP-SM), some gravel, trace silt, trace clay Dense Brown Moist
171.0 / 8.7	SILT (ML) and sand, trace gravel, some clay, contains cobbles and boulders (TILL) Compact Brown to light brown Moist
169.2 / 10.5	SILTY SAND to SAND (SW/SM), trace gravel Loose to dense Brown Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	AS	-				
2	SS	22				
3	SS	36	13	48	29	10
4	SS	35				
5	SS	41				
6	SS	54	13	28	51	8
7	SS	110				
8	SS	44				
9	SS	41				
10	SS	43				
11	SS	35	12	75	9	4
12	SS	23	5	37	42	16
13	SS	37				

GROUND WATER CONDITIONS: ∇ (at approx. elev. 175.3)

ELEVATION SCALE: 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No 472-22-02 SHEET 2 OF 2 METRIC

PROJECT 1773612

G.W.P. 4054-17-00 LOCATION N 4880342.1; E 199177.6 MTM NAD ZONE 9 (LAT. 44.055460; LONG. -77.818240) ORIGINATED BY BW

DIST Eastern HWY 401 BOREHOLE TYPE CME 55 Truck Mounted, 200 mm OD Hollow Stem Augers COMPILED BY ZS/KG

DATUM GEODETIC DATE October 18, 2022 CHECKED BY KCP/LCC

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 to SAND (SW/SM), trace gravel Loose to dense Brown Wet		14	SS	9		167		o (w ≈ 15)		
							166				
			15	SS	23						
165.4 14.3	END OF BOREHOLE NOTES: 1. Water level measured in open borehole at a depth of 4.4 m (Elev. 175.3 m) upon completion of drilling.										

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

RECORD OF BOREHOLE No 472-22-03

SHEET 1 OF 2 **METRIC**

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

LOCATION N 4880322.2; E 199198.7 MTM NAD ZONE 9 (LAT. 44.055290; LONG. -77.817980)
BOREHOLE TYPE CME 55 Truck Mounted, 200 mm OD Hollow Stem Augers
DATE July 24, 2022

ORIGINATED BY JS
COMPILED BY ZS/KG
CHECKED BY KCP/LCC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
180.0	GROUND SURFACE
0.0	ASPHALT (300 mm)
179.7 / 0.3	Gravelly SAND (SW) (PAVEMENT STRUCTURE) (FILL) Brown Moist
179.2 / 0.8	SILTY SAND (SM), some clay, some gravel (FILL) Compact to very dense Brown Wet to moist
174.8 / 5.2	SILTY SAND (SM), trace gravel, some clay, organics (FILL) Very dense Brown Moist
172.8 / 7.2	SAND (SW), some gravel, trace silt, contains organics and woody debris Dark brown, with black staining Moist
172.4 / 7.6	SILTY SAND (SM), some gravel Compact Dark brown to brown Moist to wet
170.9 / 9.1	Sandy CLAYEY SILT to SILT (CL-ML) and sand, trace gravel, contains cobbles and boulders (TILL) Compact to very dense Grey w~PL

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS	-	
2	SS	11	
3	SS	65	
4	SS	29	12 44 (44)
5	SS	47	
6	SS	17	
7	SS	24	
8	SS	79	
9	SS	61	10 46 (44)
10	SS	15	
11	SS	10	20 56 (24)
12	SS	17	
13	SS	11	12 31 38 19
14	SS	18	2 46 (52)
15	SS	33	0 22 59 19

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (20 40 60 80 100) — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 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 Next Page

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

RECORD OF BOREHOLE No 472-22-03 SHEET 2 OF 2 METRIC

PROJECT 1773612
G.W.P. 4054-17-00
LOCATION N 4880322.2; E 199198.7 MTM NAD ZONE 9 (LAT. 44.055290; LONG. -77.817980)
ORIGINATED BY JS
DIST Eastern HWY 401
BOREHOLE TYPE CME 55 Truck Mounted, 200 mm OD Hollow Stem Augers
COMPILED BY ZS/KG
DATUM GEODETIC
DATE July 24, 2022
CHECKED BY KCP/LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Sandy CLAYEY SILT to SILT (CL-ML) and sand, trace gravel, contains cobbles and boulders (TILL) Compact to very dense Grey w~PL	16	SS	80		167	
166.9 13.1	Gravelly SAND (SW), some silt, contains cobbles and boulders (TILL) Very dense Grey Wet	17	SS	72		166	
166.0 14.0	END OF BOREHOLE						

NOTE:

1. Water was added to the hollow stem augers once drilling extended below approximately 12 m depth (Elev. 168.0 m) to assist in washing out cuttings.

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: 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

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

RECORD OF BOREHOLE No 472-22-04

SHEET 1 OF 1 **METRIC**

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

LOCATION N 4880300.2; E 199221.3 MTM NAD ZONE 9 (LAT. 44.055090; LONG. -77.817690)
BOREHOLE TYPE LAD Multipower Track Mounted, 165 mm OD Hollow Stem Augers
DATE July 6, 2022

ORIGINATED BY JS
COMPILED BY ZS/KG
CHECKED BY KCP/LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
171.5	GROUND SURFACE					
0.0	TOPSOIL					
0.1	Dark brown, Moist. SILTY SAND (SM), some gravel, trace to some clay (FILL). Very loose. Light brown. Moist to wet	1	SS	2		
		2	SS	4		
170.6 / 0.9	SAND (SP), some silt, some gravel, trace clay, contains organics (FILL). Loose. Dark brown. Moist to wet	3	SS	1		
169.5	Fibrous PEAT (PT), contains woody debris					
2.1	SAND (SW), some gravel, trace to some silt, contains organics. Compact. Dark drown. wet	4	SS	14		
168.5 / 3.1	Sandy CLAYEY SILT to SILT (CL-ML) and sand, trace gravel, contains cobbles and boulders (TILL). Very stiff to hard/ dense to compact. Grey. Moist to wet	5	SS	35		
		6	SS	22	plotted	9 27 (64)
		7	SS	25		
		8	SS	18	plotted	4 38 (58)
		9	SS	23		
		10	SS	22	plotted	1 39 (60)
		11	SS	27		
		12	SS	29		
		13	SS	29		
161.8 / 9.8	END OF BOREHOLE					

NOTE:

1. Borehole was advanced using a head of water within the hollow stem augers below 6.1 m depth to control ground/sample disturbance.

Elevation scale: 171, 170, 169, 168, 167, 166, 165, 164, 163, 162

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
WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

$+^3, \times^3$: 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 3/15/24

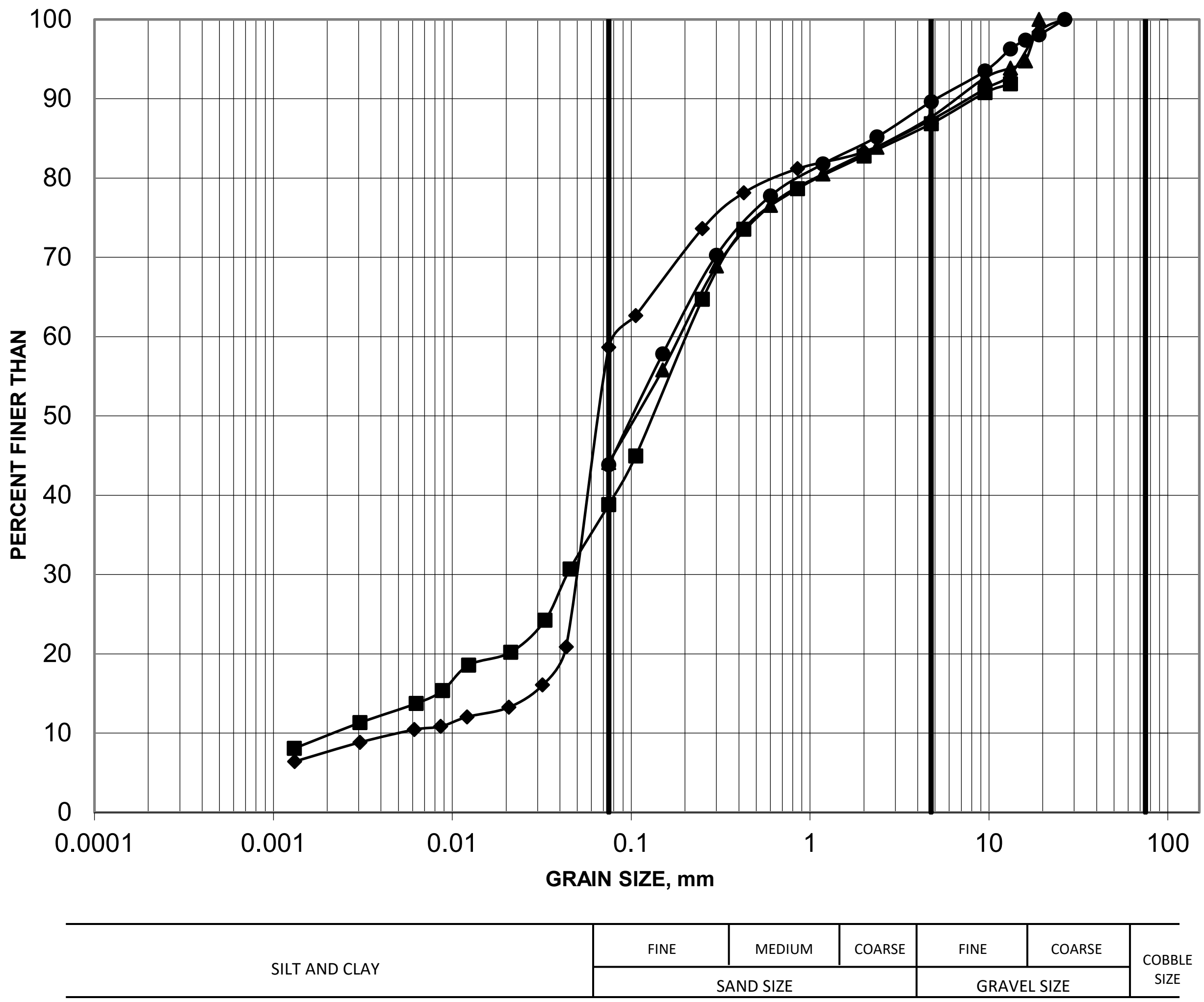
APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY SAND (SM) TO SAND SILT (ML) FILL



Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
■ 472-22-02	3	1.52-2.13	13	48	29	10
◆ 472-22-02	6	3.81-4.42	13	28	51	8
▲ 472-22-03	4	2.29-2.90	12	44	44	
● 472-22-03	9	6.10-6.71	10	46	44	

Project: 1773612_WO 11

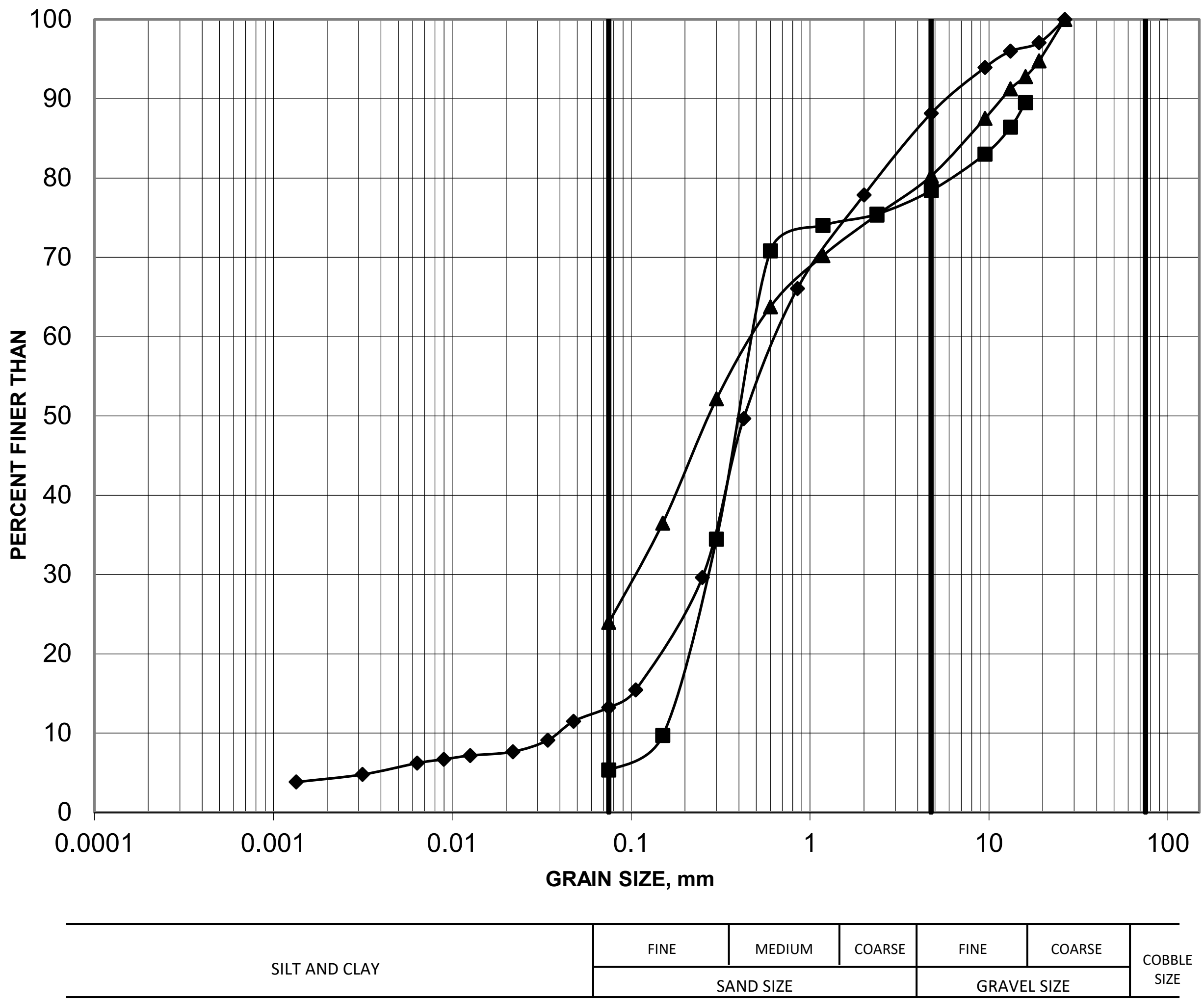
GOLDER
MEMBER OF WSP

Created by: KG
Checked by: KCP

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab/5-Culvert 472/Figures/

GRAIN SIZE DISTRIBUTION	FIGURE B2

SILTY SAND (SM) TO SAND (SP/SW)



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■ 472-22-01	7	4.57-5.18	22	73	5	
◆ 472-22-02	11	7.62-8.23	12	75	9	4
▲ 472-22-03	11	7.62-8.23	20	56	24	

Project: 1773612_WO 11

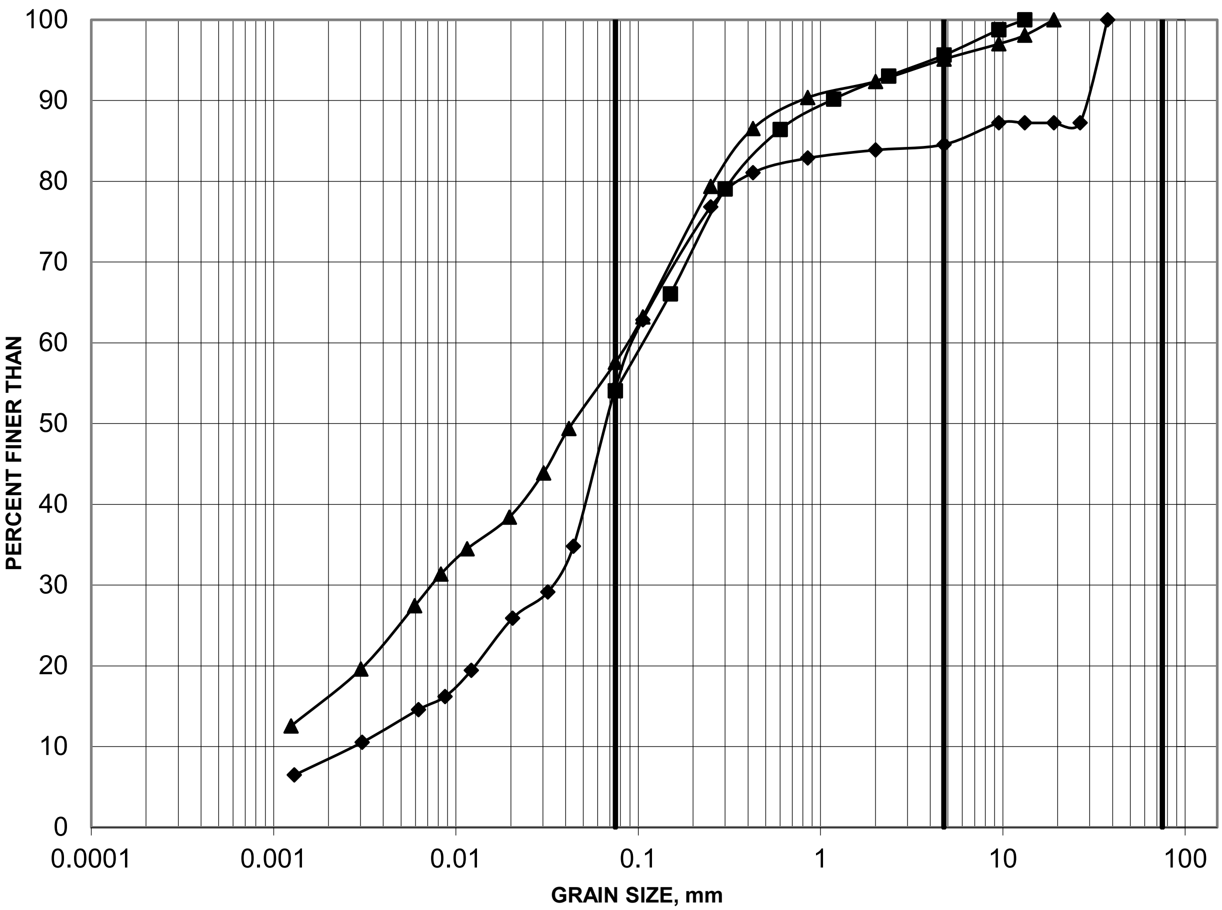
GOLDER
MEMBER OF WSP

Created by: KG
Checked by: KCP

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab/5-Culvert 472/Figures/

GRAIN SIZE DISTRIBUTION	FIGURE B3

SILT (ML) AND SAND TO SILTY SAND (SM) TILL



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

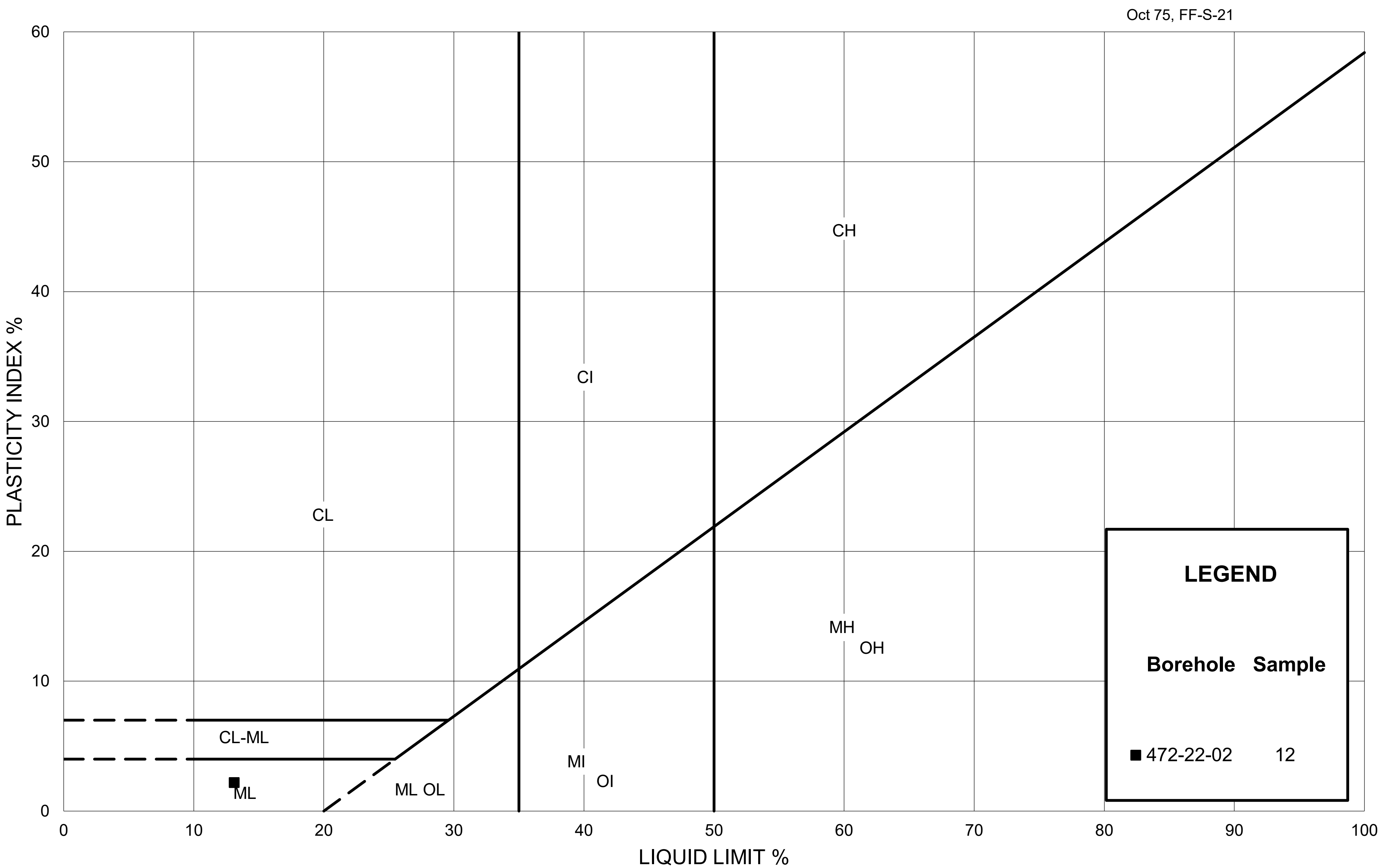
			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■ 472-22-01	2	0.76-1.37	4	42	54	
◆ 472-22-01	4	2.29-2.90	15	31	45	9
▲ 472-22-02	12	9.14-9.75	5	37	42	16

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/5-Culvert 472/Figures/



Ministry of Transportation

Ontario

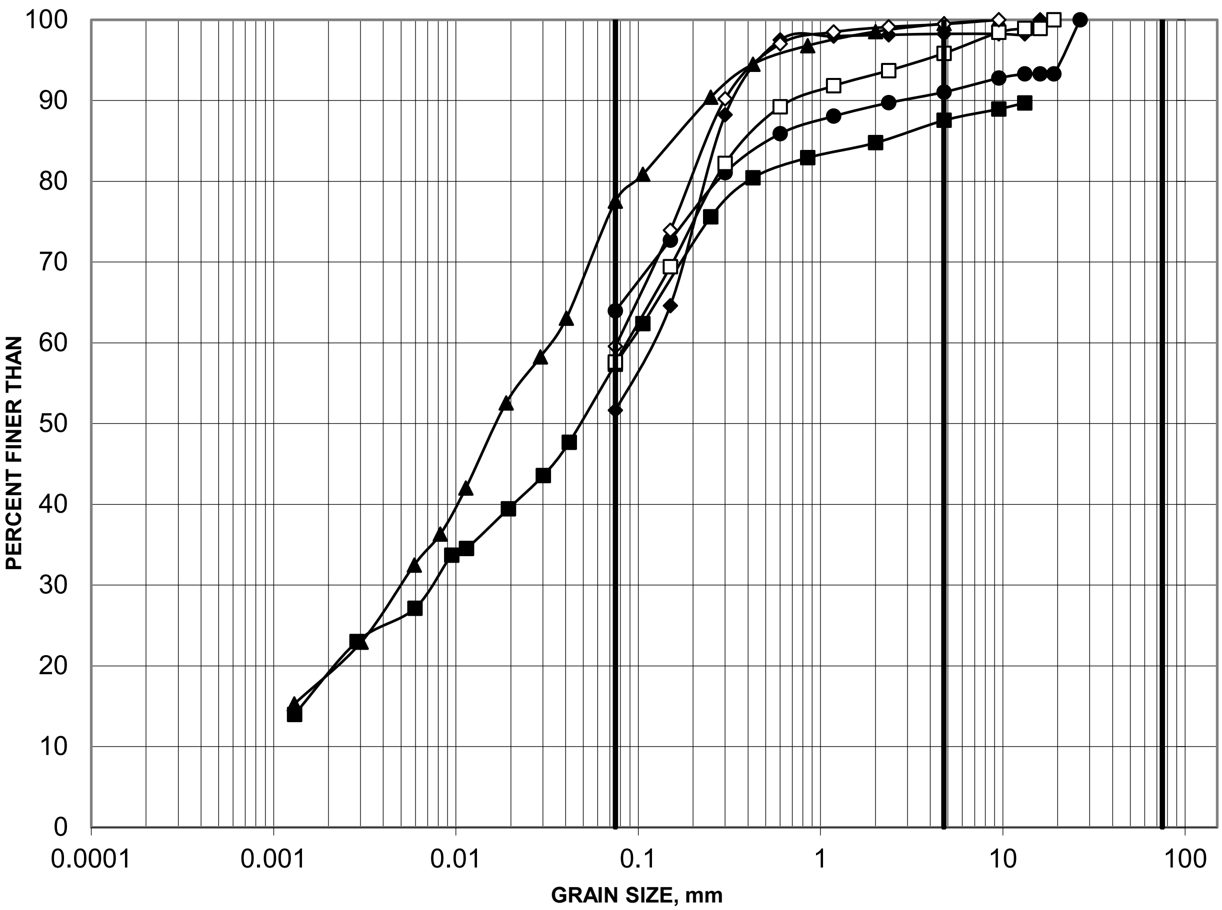
PLASTICITY CHART

SILT (ML) AND SAND TO SILTY SAND (SM) TILL

Figure:	B4
Project:	1773612_W011
Created By: KG	Checked By: KCP

GRAIN SIZE DISTRIBUTION	FIGURE B5

SANDY CLAYEY SILT (CL-ML) TILL



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

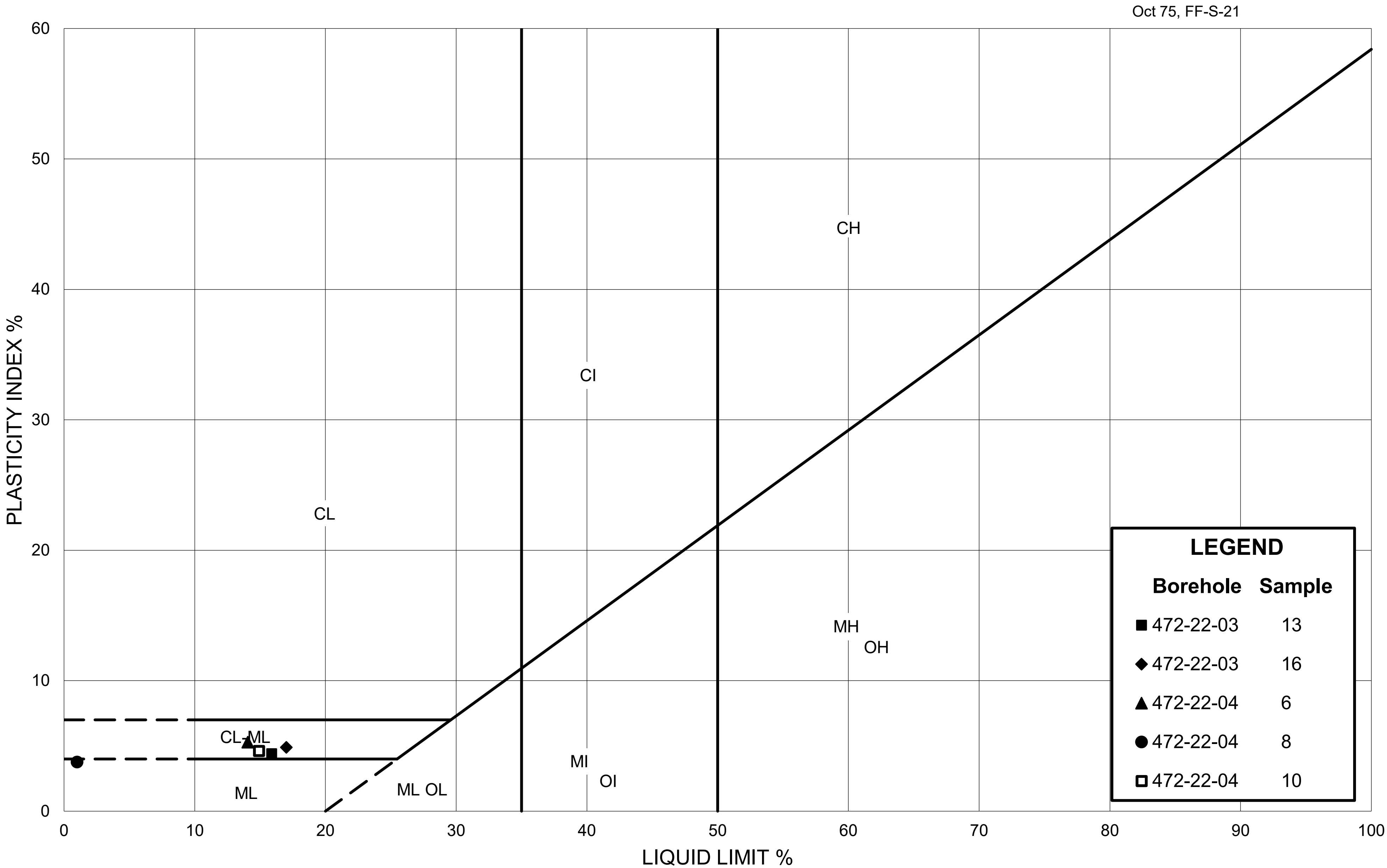
Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
■ 472-22-03	13	9.14-9.75	12	31	38	19
◆ 472-22-03	14	9.91-10.52	2	46	52	
▲ 472-22-03	15	10.67-11.28	0	22	59	19
● 472-22-04	6	3.81-4.42	9	27	64	
□ 472-22-04	8	5.33-5.94	4	38	58	
◇ 472-22-04	10	6.86-7.47	1	39	60	

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/5-Culvert 472/Figures/



Ministry of Transportation

Ontario

PLASTICITY CHART

SANDY CLAYEY SILT (CL-ML) TILL

Figure:	B6
Project:	1773612_W011
Created By: KG	Checked By: KCP

APPENDIX C

Analytical Laboratory Test Results

Certificate of Analysis

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

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
.
Attention: Mr. Kenton Power
PO#:
Invoice to: Golder Associates Ltd

Report Number: 1985544
Date Submitted: 2022-09-07
Date Reported: 2022-09-15
Project: 1773612-W011
COC #: 899907

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

				Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1649740 Soil 2022-07-06 472-22-04 Sa2/2.5-4.5'	1649741 Soil 2022-07-27 473-22-03 Sa2/2.5-4.5'	1649742 Soil 2022-07-04 474-22-04 Sa3/5-7'
Group	**Analyte**	**MRL**	**Units**	**Guideline**			
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
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

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

Site Photographs and 1958 Drawings

Photograph 1: Existing Culvert Location Looking Downstream

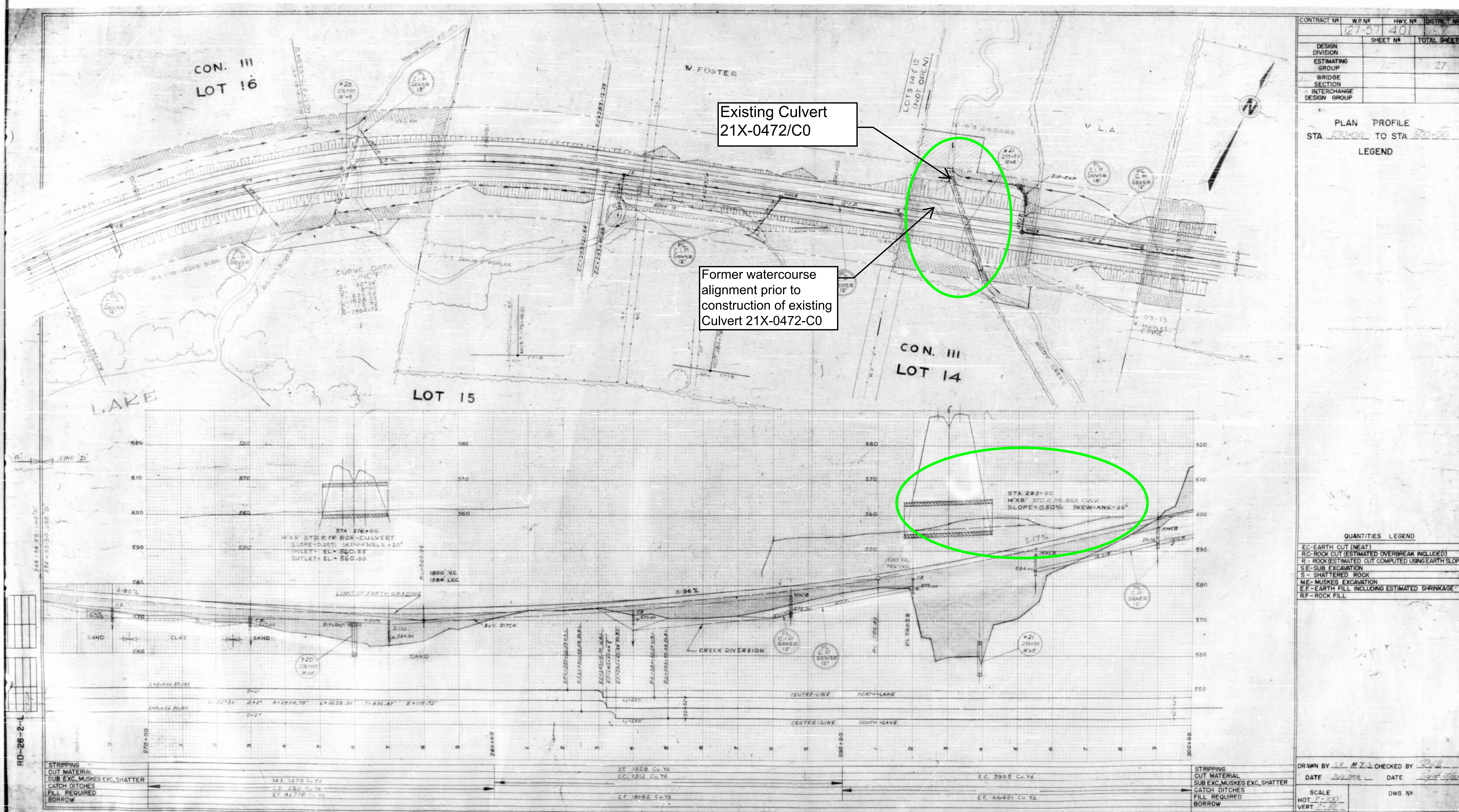
Photograph 2: Looking Northwest from South Side of Highway 401 Eastbound Lanes

Photograph 3: Highway 401 Eastbound Lane, Looking Eastward

Photograph 4: Location of Borehole 472-22-01 on North Side of Highway 401

Photograph 5: Existing North Embankment Side Slope Looking Eastbound

Photograph 6: Existing North Toe of Side Slope Looking East

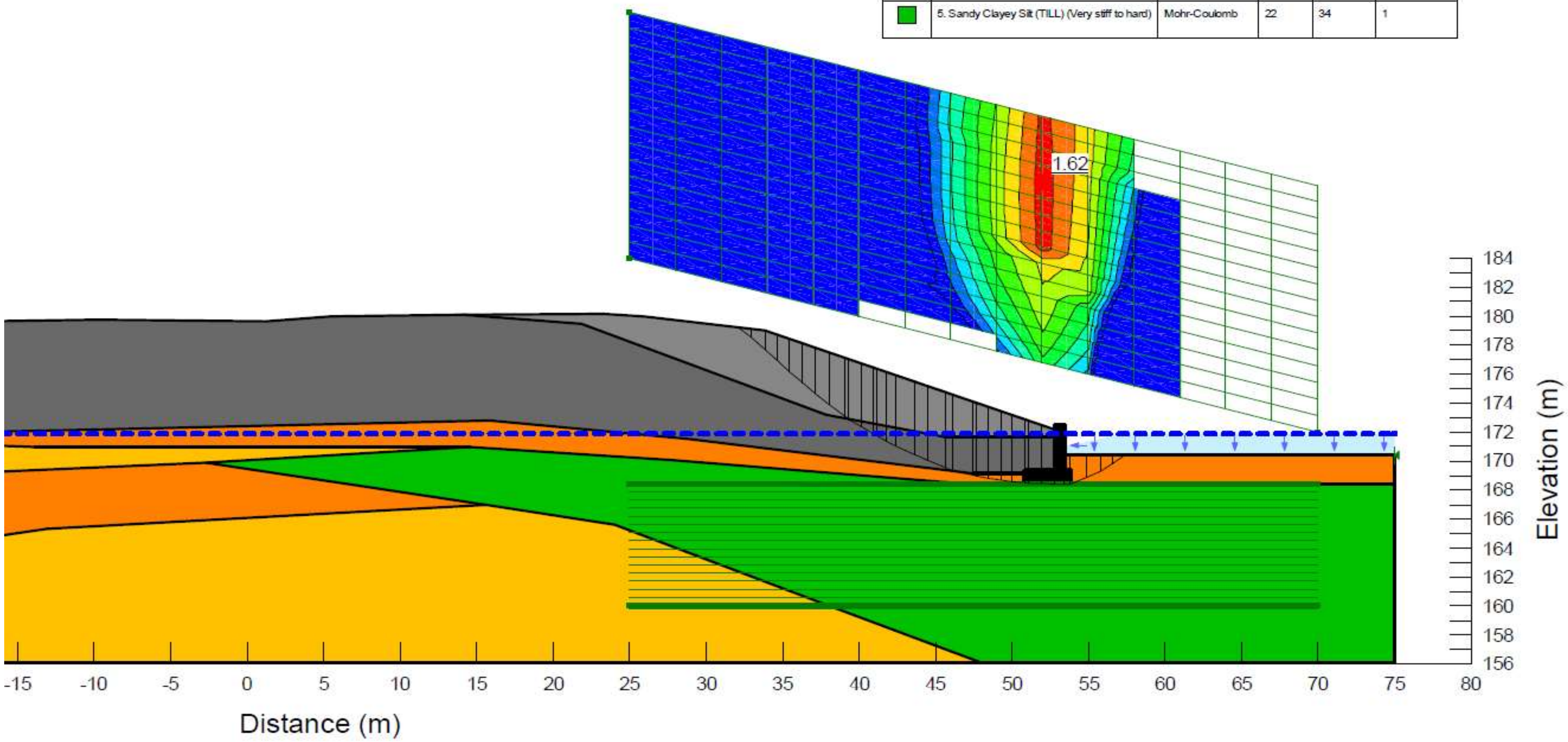

Existing Culvert
21X-0472/C0
Former watercourse alignment prior to construction of existing Culvert 21X-0472-C0
CONTRACT Nº
W.P Nº
127-57
HWY. Nº
401
DISTRICT Nº
SHEET Nº
TOTAL SHEETS
DESIGN DIVISION
ESTIMATING GROUP
BRIDGE SECTION
INTERCHANGE DESIGN GROUP
PLAN PROFILE
STA 270+00 TO STA 300+00
LEGEND
CON. III
LOT 16
LOT 15
CON. III
LOT 14
LOTS 14 & 15 (NOT OPEN)
W. FOSTER
V. L. A.
LAKE
CURVE DATA
STA. 293+90
14'X8' STD. R. FR. BOX CULV.
SLOPE = 0.50% SKEW-ANG.= 25°
STA 276+00
14'X8' STD. R. FR. BOX-CULVERT
SLOPE = 0.25% SKEW-ANGLE = 20°
INLET - EL.= 560.55
OUTLET - EL.= 560.00
LIMIT OF EARTH GRADING
CREEK DIVERSION
CENTRE-LINE NORTH-LANE
CENTRE-LINE SOUTH-LANE
SAND
CLAY
QUANTITIES LEGEND
E.C.- EARTH CUT (NEAT)
R.C.- ROCK CUT (ESTIMATED OVERBREAK INCLUDED)
R.- ROCK (ESTIMATED CUT COMPUTED USING EARTH SLOPES)
S.E.- SUB EXCAVATION
S.- SHATTERED ROCK
M.E.- MUSKEG EXCAVATION
E.F.- EARTH FILL INCLUDING ESTIMATED SHRINKAGE
R.F.- ROCK FILL
STRIPPING
CUT MATERIAL
SUB EXC., MUSKEG EXC., SHATTER
CATCH DITCHES
FILL REQUIRED
BORROW
DRAWN BY
CHECKED BY
DATE
SCALE
HOR. 1"=100'
VERT. 1"=10'
DWG Nº
RD-28-2-L

APPENDIX E

Global Stability Analyses

Name: 1-Proposed Side Slope CV472 with RW - Static South Embankment
Analysis Type: Morgenstern-Price
Groundwater Elev.171.9 m
Direction of movement: Left to Right
Horz Seismic Coef.: 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Friction Angle (°)	Piezometric Surface
	Retaining Wall	High Strength	30		
	1. New Granular A or B Type 1 or Type 2 FILL (Compacted)	Mohr-Coulomb	22	35	1
	2. Existing Granular Fill (Loose to dense)	Mohr-Coulomb	20	32	1
	3. Silty Sand to Sand (Generlly Compact to Dense)	Mohr-Coulomb	19	30	1
	4. Silt and Sand (TILL) (Loose to Dense)	Mohr-Coulomb	19	32	1
	5. Sandy Clayey Silt (TILL) (Very stiff to hard)	Mohr-Coulomb	22	34	1



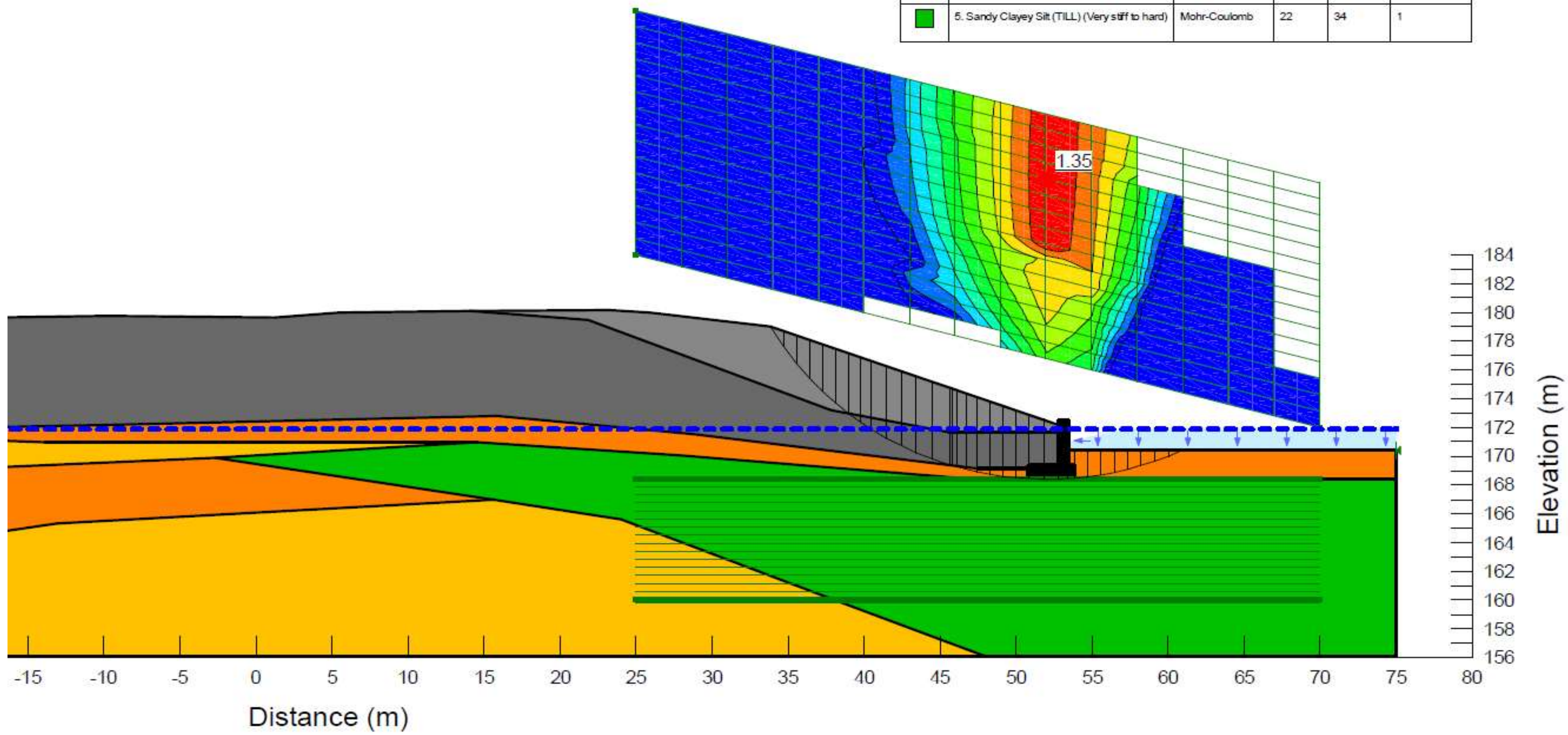
Replacement of Highway 401 Culvert Site 21X-0472/C0
Cramahe Township, Northumberland County, Ontario
MTO GWP 4054-17-00; Agreement NO. 4016-E-0034 Assignment No. 11
Global Stability - South Embankment Widening and Retaining Wall - Static Analysis

Project No:	1773612
Drawn:	BW
Date:	March 7, 2024
Checked:	KCP
Review:	LCC

FIGURE E1

Name: 2-Proposed Side Slope CV472 with RW - Siesmic South Embankment
Analysis Type: Morgenstern-Price
Groundwater Elev.171.9 m
Direction of movement: Left to Right
Horz Seismic Coef.: 0.074

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
	4. Silt and Sand (TILL) (Loose to Dense)	Mohr-Coulomb	19	32	1
	5. Sandy Clayey Silt (TILL) (Very stiff to hard)	Mohr-Coulomb	22	34	1



wsp

Replacement of Highway 401 Culvert Site 21X-0472/C0
Cramahe Township, Northumberland County, Ontario
MTO GWP 4054-17-00; Agreement NO. 4016-E-0034 Assignment No. 11
Global Stability - South Embankment Widening and Retaining Wall - Seismic Analysis

Project No:	1773612
Drawn:	BW
Date:	March 7, 2024
Checked:	KCP
Review:	LCC

FIGURE E2

APPENDIX F

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><td colspan="9">TEMPORARY FLOW PASSAGE SYSTEMS</td></tr>
<tr><td colspan="9">Source of Return Period Flow Estimates:</td></tr>
<tr><td rowspan="2">Site Name / Station Reference</td><td rowspan="2">Minimum Return Period (Years)</td><td colspan="4">Return Period Flow Estimates (m^3/s) (Note 1)</td><td rowspan="2">Design Engineer Requirements (Note 2)</td><td rowspan="2">Fish Passage Required (Note 3)</td></tr>
<tr><td>2 Year</td><td>5 Year</td><td>10 Year</td><td>25 Year</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td colspan="9">DEWATERING SYSTEMS</td></tr>
<tr><td>Site Name / Station Reference</td><td>Preconstruction Survey Distance (m) (Note 4)</td><td colspan="4">Minimum Lowered Groundwater Depth Below Base of Excavation or Work Area (m) (Note 5)</td><td colspan="2">Design Engineer Requirements (Note 2)</td></tr>
<tr><td>Culvert 21X-0472/C0</td><td>N/A</td><td colspan="4">1 m</td><td colspan="2">Yes</td></tr>
</table>

Notes:

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

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

NOTES TO DESIGNER:

Designer Fill-Ins for Table 1:

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

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

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

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

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

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

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

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

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

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

WARRANT: Always with these tender items.

Existing Subsurface Conditions

Notice to Contractor

The Contactor is alerted to the potential for cobble and/or boulder obstructions within the fill materials. Non-cohesive till (silt and sand to silty sand till as well as gravelly sand) and cohesive till (sandy clayey silt) deposits were encountered below the fill materials in all drilled boreholes, and glacial tills inherently contain cobbles and boulders. The extent and depth of obstructions and of gravelly zones may vary between and beyond the borehole locations. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for temporary works and/or construction of the replacement culvert, as applicable.

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