

FINAL REPORT

Preliminary Foundation Investigation and Design Report

Replacement of Highway 401/Lake Road Underpass (Structure 21X-0295/B0)

Township of Cramahe, Northumberland County

MTO GWP 4054-17-00; MTO Agreement No. 4016-E-0034-011

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF HIGHWAY 401/LAKE ROAD UNDERPASS
(STRUCTURE 21X-0295/B0)
TOWNSHIP OF CRAMAHE, NORTHUMBERLAND COUNTY
MTO GWP 4054-17-00, AGREEMENT NO. 4016-E-0034-011

1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., now a member of WSP Canada Inc.) has been retained by WSP Canada Inc. (WSP) on behalf of the Ministry of Transportation, Ontario (MTO) to support future procurement to support future procurement-ready design phases of the rehabilitation and widening of Highway 401 from 0.8 km east of Percy Street to 0.4 km west of Christiani Road. The overall project includes the replacement of three bridge structures and four structural culverts.

This report presents the results of the preliminary foundation investigation carried out for the replacement of the Highway 401/Lake Road Underpass (MTO Structure Site No. 21X-0295/B0).

The preliminary foundation engineering services for this project have been delivered under MTO Agreement No. 4016-E-0034-011 as part of MTO GWP 4054-17-00.

2.0 SITE DESCRIPTION

The existing Highway 401/Lake Road underpass is located approximately 6.1 km west of County Road 30 in the Township of Cramahe in Northumberland County, Ontario. The site location is shown on the key plan on Drawing 1. There are no interchange ramps at this location.

At this location, Highway 401 has a four-lane cross-section with two eastbound and two westbound through lanes with wide paved shoulders separated by a concrete median wall. Highway 401 has been constructed in a cut relative to the surrounding natural ground surface, with the highway grade at approximately Elevation 180.5 m to 181 m at the structure site, and the natural ground surface beyond the highway at approximately Elevation 188 m to 189 m on the north side of the highway, and 185 m on the south side of the highway.

Lake Road is an undivided road with a rural cross-section and a single travel lane in each direction that carries traffic over Highway 401. Parapet walls with railing are present along the bridge and steel beam guiderails are present along both side of CR26 beyond the bridge. Lake Road has been constructed on existing glacial till including drumlins, interspersed with sand deposits, with its grade at approximately Elevation 190 m to 186.5 m, declining from north to south across the structure.

The existing underpass was constructed in 1958 under MTO Contract 58-278; it is a three-span structure with perched abutments and piers founded on spread footings. The existing north and south approach embankments are about 10 m and 5 m high relative to the elevation of Highway 401, respectively. Based on visual observation at the time of the investigation, there are no signs of embankment instability or settlement.

The area surrounding the site consists of agricultural land to the north and residential south of Highway 401. Little Lake is located to the southeast of the existing structure. Site photographs showing the general conditions of the site are presented in Appendix D.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on July 19 and 20, 2022 and included advancing two boreholes, (Lake22-01 and Lake22-02) through the travelled lanes of Lake Road north and south of the existing abutments, respectively. The borehole locations are shown on Drawing 1.

The boreholes were advanced with a CME55 truck-mounted drill rig, supplied and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario. Soil samples were obtained using a 50 mm outer diameter split-spoon sampler in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). Soil samples were obtained at vertical sampling intervals of about 0.76 m and 1.5 m.

A monitoring well was installed at Borehole Lake22-02 to observe the stabilised groundwater level at the site. The monitoring well consists of 52 mm outside diameter PVC tube with a 1.5 m long slotted screen. Well installation details are shown on the borehole record for Borehole Lake22-02 provided in Appendix A. The borehole without a monitoring well was 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 the completion of the field work.

The field work was supervised on a full-time basis by members of WSP Golder'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 Golder'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. The laboratory tests were carried out to MTO and/or ASTM Standards, as applicable at WSP Golder's Ottawa laboratory.

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

The borehole locations and elevations were surveyed by WSP Golder using a Trimble R10 GPS unit referenced to the NAD83 CSRS CBNv6-2010.0 MTM Zone 9 geodetic datum. 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 No.	NAD83 CSRS CBNv6-2010.0 MTM Zone 9		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
Lake22-01	4879362.9 (44.04650)	198092.6 (-77.83158)	186.0	9.4 ¹
Lake22-02	4879450.5 (44.04728)	198061.6 (-77.83199)	190.3	9.4 ¹

Notes: ¹ Borehole terminated at refusal within glacial till

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario* the Highway 401/Lake Road underpass lies in the physiographic regions known as the South Slope. 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 the 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 fine sand and silt is found on the surface of the till up to a depth of six or eight feet. The South slope 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.

4.2 General Subsurface Conditions

The subsurface soil, and groundwater conditions encountered in the boreholes including piezometer installation details, water level readings, and the results of in-situ testing from the investigation are shown on the borehole records in Appendix A. The results of the in-situ field tests (i.e., SPT “N”-values) as presented on the borehole records and herein are uncorrected and are based on use of an automatic hammer. The borehole locations and the interpreted stratigraphic profiles projected along the underpass alignment are provided in Drawing 1.

The results of the geotechnical laboratory testing are presented on the borehole records as well as on Figures B1 to B4 in Appendix B. The results of analytical testing for corrosivity parameters are provided in Appendix C.

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.

In general, the subsurface conditions encountered at this site consist of existing pavement structure (asphalt and pavement granular material) and non-cohesive fill associated with the existing Lake Road embankment underlain by a silty sand which is further underlain by glacial till deposits comprising gravelly silty sand and non-plastic silt containing cobbles and boulders up to the termination depth of the boreholes. A more detailed description of the overburden soil deposits encountered during the field investigation is provided in the following sections.

4.2.1 Existing Pavement Structure

An approximately 100 mm thick layer of asphalt pavement was encountered at the ground surface in both boreholes. Approximately 0.7 m of granular material consisting of silty sand and gravel was encountered beneath the asphalt in both boreholes.

4.2.2 Gravelly Sand Fill

Underlying the existing pavement structure at Borehole Lake22-02, non-cohesive fill consisting of gravelly sand containing trace non-plastic fines was encountered. This fill was encountered at a depth of 0.8 m below ground surface and was about 0.7 m thick extending to Elevation 188.8 m.

The Standard Penetration Test (SPT) N-value recorded in the granular fill was 50 blows per 0.3 m of penetration indicating a very dense consistency or loose to very dense state of compactness. The measured moisture content of one sample of the granular fill was 2%. The results of grain size analysis testing carried out on one sample of the granular fill are provided in Figure B1 in Appendix B.

4.2.3 Silty Sand

Underlying the existing pavement structure at Borehole Lake22-01 a silty sand deposit was encountered. This silty sand was encountered at a depth of 0.8 m below ground surface and was about 2.9 m thick extending to Elevation 182.3 m.

The SPT 'N'-values measured within the silty sand layer ranges from 11 to 44 blows per 0.3 m of penetration indicating a compact to very dense state of compactness. The measured moisture content of one sample of the silty sand was 8%. The results of grain size analysis testing carried out on one sample of the silty sand are provided in Figure B2 in Appendix B.

4.2.4 Silty Sand to Non-Plastic Silt and Sand Till

Glacial till was encountered at a depth of 3.7 m (Elevation 182.3 m) at Borehole Lake22-01 and 1.5 m (Elevation 188.8 m) at Borehole Lake22-02. The till deposit varies in composition from silty sand, trace to some gravel to gravelly silty sand, and it grades with depth in Borehole Lake22-02 to non-plastic silt and sand containing some gravel. A granite boulder was encountered at a depth of 8.5 m (Elevation 177.5 m) within the till and required rotary coring techniques to penetrate to advance the borehole; a 30 cm granite boulder sample was recovered. Boreholes Lake22-01 and Lake 22-02 were terminated in the till deposit after penetrating it for 5.7 m and 7.9 m, respectively.

Frequent split-spoon sampler refusal and auger grinding was observed in both boreholes. This coupled with cobbles in the drill cuttings and the cored boulder confirm the presence of cobbles and boulder within the glacial till deposit. The SPT 'N' values measured in the glacial till ranged from 19 blows to greater than 100 blows per 0.3 m, but more commonly 56 to 70 blows per 0.3 m penetration suggesting a very dense state of compactness.

The water content measured on samples of the silty sand till ranged from 4% to 7% and one sample of non-plastic silt and sand till was 9%. The results of grain size distribution testing carried out on three samples of the silty sand till and one sample of non-plastic silt and sand are presented in Figures B3 and B4, respectively, in Appendix B. The results of Atterberg limits testing completed on one sample of the fines portion of the non-plastic silt and sand till do not plot on a Plasticity Chart; this Atterberg limit test result indicates a non-plastic silt (ML).

4.3 Groundwater Conditions

A monitoring well was installed at Borehole Lake22-02 to measure the stabilized groundwater level at the site. The groundwater levels measured in the monitoring well are presented in Table 2.

It is expected that the groundwater levels will be subject to fluctuations both seasonally and as a result of precipitation events.

Table 2: Summary of Groundwater Conditions

Borehole	Screened Interval	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Groundwater Elevation (m)	Date
Lake22-02	Glacial Till	190.3	6.9	183.4	July 21, 2022

4.4 Results of Chemical Analyses to Assess Potential for Steel Corrosion and Sulphate Attack

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: Steel Corrosion and Sulphate Attack, Chemical Analysis

Borehole	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
Lake22-01	0.8 – 1.4	0.007	<0.01	0.23	9.32	4,348

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Ben Waechter, EIT at WSP Golder and reviewed by Kenton Power, P.Eng., a senior geotechnical engineer with WSP Golder. Lisa Coyne, P.Eng., a Fellow and MTO Designated Foundations Contact for WSP Golder, conducted an independent technical and quality review of this report.

Signature Page


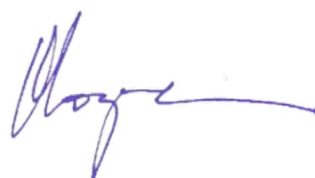
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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
REPLACEMENT OF HIGHWAY 401/LAKE ROAD UNDERPASS
(STRUCTURE 21X-0295/B0)
TOWNSHIP OF CRAMAHE, NORTHUMBERLAND COUNTY
MTO GWP 4054-17-00, AGREEMENT NO. 4016-E-0034-011**

6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation recommendations for planning and preliminary design of the Highway 401/Lake Road underpass (MTO Structure Site No. 21X-0295/B0). The recommendations are based on interpretation of the factual data obtained from the boreholes advanced as part of the current preliminary investigation and the current preliminary replacement plan provided.

The Preliminary Foundation Design Report (Part B of this report) including the discussion and preliminary recommendations are intended for the use of MTO and their designers for planning and preliminary design and shall not be relied upon for any other purpose or by any other parties, including the future construction contractor or design-build proponents. Contractors undertaking the work must make their own interpretation based on the factual data presented in the Preliminary 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 concept and preliminary design of the project and for which special provisions may be required in the future contract documents. Those requiring information on aspects of detail design and construction must make their own interpretation of the information provided and supplement as necessary, as such interpretation may affect detail design, equipment selection, proposed construction methods, scheduling and the like.

6.2 Project Understanding

Based on the preliminary General Arrangement (GA) drawings provided by WSP (dated February 2023), the proposed Highway 401/Lake Road underpass replacement will be maintained along the existing Lake Road alignment. The existing three-span structure is proposed to be replaced with a three-span structure with a total length of approximately 87 m. The proposed south abutment will be located approximately 12 m north of the existing abutment, immediately south of the existing south pier, while the proposed north abutment is planned to be located approximately 20 m north of the existing abutment. The future south pier is planned to be located at approximately the current Highway 401 centreline, and the future north pier is to be located north of the existing highway platform and existing north pier. The replacement underpass may be constructed with an interim two-span configuration, in which the interim north abutment is located at the future north pier location, between the existing north pier and north abutment. Retained soil system walls are shown in the preliminary GA drawing in front of the interim and final abutment locations. The interim and final configurations are shown on the profile on the Borehole Location and Soil Strata drawing (Drawing 1).

Highway 401 will be widened from four lanes to eight lanes in the ultimate configuration. The current elevation of the Highway 401 eastbound and westbound lanes at the limits of the underpass structure are approximately 180.6 m and 179.6 m, respectively. The Lake Road grade will be raised by less than approximately 1 m at the north and south approach embankments. If an interim two-span structure configuration is adopted with the interim north abutment located at the future north pier, up to approximately 6 m of fill will be required to be placed in the wedge-shaped zone atop the existing cut slope on the north side of Highway 401 to support the Lake Road grade immediately north of the interim north abutment.

6.3 Foundation Options

Based on the proposed three-span configuration as well as potential consideration of an interim two-span configuration, the surface topography (sloping downward from north to south) and the subsurface conditions encountered at the site, both shallow and deep foundation options have been considered for support of the new abutments and piers. The preliminary recommendations provided herein will be subject to change, based on future investigations and testing in detail design.

A summary of the general advantages and disadvantages associated with each option is provided below and a comparison of the alternative foundation options based on advantages, disadvantages, relative costs and risks is provided in Table 9 following the text of this report.

- **Strip footings:** Shallow foundations founded on the dense to very dense glacial till below the Highway 401 grade are feasible for both the abutments and piers. Subject to further investigation in detail design to confirm the shallow subsoil conditions, this option may be most practicable at the piers as “100-blow” till is present at relatively shallow depth. At the abutments, use of this option in a “closed” structure configuration would require significant excavation and associated protection systems to extend below the natural ground surface to reach the very dense till. As Highway 401 is constructed in a cut at this location, competent native soils exist above the Highway 401 grade; if a longer, “open” structure configuration were adopted, it would be possible to perch abutment footings above Highway 401 grade within these competent native soils. Strip footings are likely to require a wider excavation than the footprint required for drilled shafts (caissons); if working space considerations are critical during construction staging on Highway 401, then a deep foundation option may be preferred. If an interim two-span structure is constructed, in which the interim north abutment becomes the future north pier, then shallow foundations may not be technically preferred given the lateral loading conditions associated with temporary soil retention together with the significant grade difference from the north to south sides of the site.
- **Driven piles:** Steel H-piles or tube piles driven into the “100-blow” glacial till are feasible at the abutments and would allow the pile caps to be perched within the approach embankments, thus minimizing excavation and temporary protection system requirements compared with shallow foundations. At the new piers, the very shallow depth to “100-blow” soil poses a risk to the piles being misaligned and pre-drilling would be required; in addition, based on the short pile length and effective stress conditions along such piles, the factored geotechnical resistances will be relatively low, and therefore driven piles are not recommended at the piers. Pre-drilling is also likely to be required at the abutments owing to the relatively shallow depth to “100-blow” till, but such an operation may be necessary for installation of corrugated steel pipe (CSP) liners if an integral abutment structure were adopted (if technically feasible) or otherwise to prevent the piles from transferring lateral forces to retained soil system (RSS) walls if such walls are constructed in front of the abutments. .
- **Drilled shafts (caissons):** Drilled shafts are feasible at this site and would be relatively short given the shallow depth to “100-blow” till. This foundation type offers higher axial and lateral geotechnical resistances in a narrower footprint as compared with shallow foundations and is an excellent alternative to a strip footings for support of the abutments or piers. At the piers in particular, including the interim north abutment/future north pier, drilled shafts would permit elimination of a below-grade pile cap for support of the structural columns. The use of temporary liners and/or polymer slurry will be required for support of the caisson sidewalls as well as to minimize disturbance of soils at the caisson base during construction, and

tremie concrete methods will be required based on the groundwater conditions in sand and non-cohesive till deposits.

From a geotechnical/foundation perspective given the shallow depth to “100-blow” till, and given the significant grade difference along the structure together with the potential for an interim two-span structure prior to construction of the ultimate three-span structure, drilled shaft (caisson) foundations are preferred at the piers, and drilled shafts or pre-drilled piles are preferred at the abutments.

If and where retaining walls are required adjacent to the abutments, concrete walls supported on shallow foundations are feasible; deep foundations will not likely be required. In addition, RSS walls are geotechnically feasible with negligible risk of settlement, given the shallow depth to “100-blow” till below the Highway 401 grade.

6.4 General Foundation Design Context

6.4.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code CAN/CSA S6-19* (CHBDC 2019) and its *Commentary*, the bridge structures and foundation system may be classified as having medium traffic volumes and their performance as having potential impacts on other transportation corridors, resulting in a “typical consequence level” associated with exceeding limit states design.

Based on the preliminary level of foundation investigation completed to date at this location (see Part A of this report) in comparison to the degree of site understanding, the level of confidence for design of the bridge foundation elements and approach embankments has generally been assessed as a “typical degree of site and prediction model understanding”. Accordingly, the ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor, Ψ and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , for a typical degree of site understanding, from Tables 6.1 and 6.2 of CHBDC 2019 have been used at this stage of preliminary design. During detail design, additional investigation and testing may increase the site understanding and modify the geotechnical resistance factors as appropriate.

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

6.4.2 Seismic Design

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

6.4.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the energy-corrected average standard penetration resistance, \bar{N}_{60} , below the founding level, the site may be classified as Site Class C in accordance with Clause 4.4.3.2 and Table 4.1 of CHBDC (2019), in the absence of any geophysical testing.

6.4.2.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHDBC and based on the location of the proposed structure, the Class C 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 C Spectral Values for Subject Site

Parameter	2% Probability of Exceedance in 50 years (2,475-year) (g)
PGA	0.149
Sa(0.2)	0.305
Sa(0.5)	0.2
Sa(1.0)	0.11
Sa(2.0)	0.0524
Sa(5.0)	0.014
Sa(10.0)	0.00481
PGV [m/s]	0.131

The fundamental period of the replacement structures has yet to be confirmed and may depend on the final design of the superstructure. In consideration of the structure's "Other" importance category and the site-specific seismic hazard values given in Table 4, in accordance Table 4.10 of the CHDBC the bridge would fall in Seismic Performance Category 2 regardless of fundamental period of the structure.

Geophysics testing such as MASW or VSP may provide a more favourable average shear wave velocity, and hence seismic site class for the SPC assessment during detailed design.

6.4.2.3 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". Lateral spreading and flow slide often accompany liquefaction along rivers and other shorelines.

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

6.4.3 Frost Protection

Strip footings and/or pile caps should be founded at a minimum depth of 1.4 m below the lowest surrounding final grade, including any distance measured perpendicular to a sloping ground surface if applicable, to provide adequate protection against frost penetration (as interpreted from OPSD 3090.101).

6.5 Shallow Foundations

Strip footings founded on the native dense to very dense glacial till below the Highway 401 grade may be used for support of the abutments and piers. As Highway 401 is constructed in a cut at this location, competent native soils exist above the Highway 401 grade; if a longer, “open” structure configuration were adopted, it would be possible to perch abutment footings above Highway 401 grade within these competent native soils. Table 5 summarizes the founding level and factored geotechnical resistances that may be used for preliminary design assuming a 3 m or 5 m wide footing.

Table 5: Preliminary Factored Geotechnical Resistances for Shallow Foundations

Foundation Element	Founding Stratum	Maximum (Highest) Founding Elevation (m)	Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance ¹ (kPa)
Abutments or piers below Highway 401 grade	Dense to very dense till	179.5 to 178.0 (1.4 m below adjacent grade)	3	500	500
			5	650	450

Notes:

1. For 25 mm of settlement.

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width and founding elevation and as such, the geotechnical resistances must be reviewed and revised if the founding elevation/depth or footing width varies from that specified. For preliminary design, interpolation or extrapolation from the above values may be applied. In general, for larger footing sizes, higher factored ultimate and lower factored serviceability geotechnical resistances would apply. The preliminary factored geotechnical resistances should also be re-evaluated to incorporate further data that may be available at the detailed design stage.

The factored ultimate geotechnical resistances provided above are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footings, eccentricity and inclination of the load should be taken into account in accordance with CHBDC 2019.

6.6 Driven Steel H-Pile or Tube Piles

Steel HP 310x110 piles or 324 mm outer diameter closed ended tube piles (assuming a minimum wall thickness of 9.5 mm) driven into the “100-blow” glacial till are considered feasible for the abutment foundations; however, some pre-drilling is expected to be required to permit the pile to penetrate the 100-blow soil. If integral abutments are feasible from a structural perspective, the piles would need to be a minimum of 5 m long, with corrugated steel pipe (CSP) liners; CSPs may also be required for non-integral abutments if adopted in conjunction with a perched pile configuration.

The depth to “100-blow” soil is interpreted to be within 2 m below the Highway 401 grade at the piers, which poses a higher risk of piles being misaligned and which would require pre-drilling for placement of piles. Hence, driven piles are not recommended at the piers.

For the installation of the steel H-piles or steel pipe piles, consideration must be given to the presence of cobble and boulders within the native soils as auger grinding, split spoon refusal, and cobbles in the soil cuttings was

frequently observed within these deposits. In this regards, steel H-piles are preferred over steel pipe piles as pile pipes are considered to pose a higher risk of experiencing refusal on boulders or being deflected away from the vertical/batter orientation during installation due to their large end area. It is recommended that piles be reinforced at the tip with driving shoes and/or flange plates in according with OPSD 3000.100 (Steel H-Pile Driving Shoe) or OPSD 3001.100 (Steel Tube Pile Drive Shoe) Type II, as appropriate, to reduce the potential for damage to the piles during driving. In very dense strata containing cobbles and/or boulders, as encountered at this site, driving shoes (such as Titus Standard 'H' Bearing Pile Points) are preferred over flange plates.

Table 6 summarizes the factored geotechnical resistances that may be used for preliminary design of HP 310x110 or 324 mm diameter tube piles; it is noted that the factored ultimate geotechnical resistance is relatively low due to the short pile length and associated effective stress along the pile shaft and at the tip. Higher capacities could be achieved for longer piles; however, it is expected that deeper pre-drilling would be required to be able to drive longer/deeper piles at the abutments.

Table 6: Preliminary Factored Geotechnical Resistances for Pile Foundations

Foundation Element	Approximate Pile Cap Elevation (m)	Maximum (Highest) Pile Tip Elevation (m)	Approximate Minimum Pile Length (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ¹ (kN)
North Abutment	186.5	181.0	5.5	1,000	>1,000
South Abutment	183.0	177.5	5.5	1,000	>1,000

Notes:

1. The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance

Pile installation should be carried out in accordance with OPSS.PROV 903 with High-Strain Dynamic testing specified on at least 10% of the piles or two piles at each foundation element (whichever is greater) in each stage of construction.

For integral abutment design, based on the subsurface conditions in the vicinity of the abutments, corrugated steel piles (CSPs) backfilled with loose sand are recommended to be installed consistent with the MTO Structural Office Report SO-96-01 titled "Integral Abutment Bridges".

6.7 Drilled Shafts (Caissons)

Caissons founded within the very dense ("100-blow") glacial till are feasible for supporting the abutments and piers. The following geotechnical resistances may be used for preliminary design based on geotechnical resistance factors for a typical degree of site understanding; these values may be refined based on the results of further investigations and testing in detail design, including deeper boreholes that extend further into the 100-blow soil.

Table 7: Preliminary Factored Geotechnical Resistances for Drilled Shafts (Caissons)

Foundation Element	Maximum (Highest) Caisson Base Elevation (m)	Caisson Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ¹ (kN)
North Abutment	173.0	1.2 1.5	11,000 15,000	>11,000 >15,000
North Pier (Interim North abutment) and South Pier	173.0	1.2 1.5	5,000 8,000	>5,000 >8,000
South Abutment	174.0	1.2 1.5	8,000 11,000	>8000 >11,000

Notes:

1. The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance.

Caisson installation must be in accordance with OPSS.PROV 903. Where caisson foundations are adopted for support of any of the foundation elements, a temporary liner or use of polymer slurry is required to support the soils during construction, to reduce disturbance and loss of ground in the water-bearing cohesionless soils. It will also be necessary to maintain a sufficient head of water and/or polymer slurry to prevent basal heave. Given that the above drilled shaft capacities have a significant end-bearing component, the performance of the drilled shafts in compression will depend to a large degree upon the final cleaning and verification of the condition of the base of the drilled shaft. Following cleaning to remove all loose cuttings, the base should be inspected by a qualified geotechnical engineer using a shaft inspection device (SID) or given the use polymer slurry, a shaft quantitative inspection device (SQUID). Should the inspection indicate that loosened material is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected.

6.8 Approach Embankments Including RSS Walls

As detailed in Section 6.2 it is proposed to replace the existing three-span structure with a three-span structure with a total length of approximately 87 m. The final configuration will have the new south abutment to be constructed 12 m north of the existing, while the new north abutment will be located 20 m north of the existing abutment. The new south pier is to be located at the current Highway 401 centreline, while the new north pier is to be located to north of the existing highway platform. It is understood that the new structure may be constructed in stages first with an interim two-span configuration followed by third span following the highway widening. Retained soil system walls are shown in the preliminary GA drawing in front of the interim and final abutment locations. The interim and final configurations are shown on Drawing 1.

The foundation engineering parameters for the major soil types encountered on the north and south side of Highway 401 are summarized in Table 8.

Table 8: Preliminary Engineering Design Parameters

Stratigraphic Unit	γ' (kN/m ³)	ϕ' (°)	E (MPa)
Existing compact sand and gravel fill	21	32	--
Compact to dense silty sand	17	34	50-100
Dense to very dense till	20	35	150

6.8.1 Subgrade Preparation for Embankment Grade Raise and Founding Level for RSS Walls

As the structure replacement will remain on the existing Lake Road alignment, the grade raise of less than 1 m will result in placement of less than 1 m of fill atop the existing embankment side slopes. If an interim two-span structure configuration is adopted with the interim north abutment located at the future north pier, up to approximately 6 m of fill will be required to be placed in the wedge-shaped zone atop the existing cut slope on the north side of Highway 401. Existing topsoil should be stripped from the embankment side slopes and foreslope cut, and the new fill benched into the existing in accordance with OPSD 208.010. In addition, where embankment side slopes are greater than 8 m in height, a 2 m wide mid-height bench should be incorporated to reduce erosion and promote surficial stability.

Where RSS walls are adopted in front of the interim and final abutments, the founding levels for the facing panels and reinforced soil mass should be in accordance with MTO's *RSS Design Guidelines*. No subexcavation is expected to be required for founding of the RSS walls adjacent to the abutments based on the competent soils encountered in the boreholes in the preliminary investigation, although this must be confirmed through further investigation during detail design.

6.8.2 Global Stability of Conventional Embankments and RSS Walls

Minimum target Factors of Safety of 1.3 and 1.5 are considered appropriate for global stability of approach embankment slopes or RSS walls for temporary (short-term) and permanent (long-term) conditions, respectively, as per Table 6.2 of CHBDC (2019) and MERO (2020) using a typical degree of site understanding.

The approach embankments with side slopes maintained no steeper than 2H:1V will have a factor of safety of greater than 1.3 and 1.5 in short-term and long-term conditions, respectively.

It is estimated that RSS walls will be up to approximately 5 m high adjacent to the south abutment, approximately 7 m high adjacent to the interim north abutment, and approximately 10 m high adjacent to the ultimate north abutment location. For the dense to very dense till founding soils, the width of the reinforced soil zone should be at least two-thirds of the height of the RSS wall to obtain a factor of safety for global stability of greater than 1.5 in permanent (long-term, effective stress) conditions. This will require further evaluation during detail design to consider any sloping ground in front of the toe of the wall during the interim conditions, which may necessitate longer reinforcing strips to achieve the required minimum factor of safety for global stability. Assessment of the internal stability of the RSS wall by the proprietary designers may also result in longer reinforcement.

6.8.3 Embankment Settlement

The target settlement performance criteria for design of approach embankments are outlined in MTO's "Embankment Settlement Criteria for Design", dated July 2, 2010. In general, new embankments approaching structural elements such as bridge abutments are to be designed such that total settlement and rate of differential settlement do not exceed 25 mm, over a 20-year period following completion of construction.

Based on the native dense to very dense cohesionless soils encountered in the boreholes and the nominal grade raise at the approaches, post-construction settlements are anticipated to be negligible at the approach embankments.

6.9 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete and/or steel of foundations buried in the soil. The long-term performance and durability of the foundations are directly related to their corrosion resistance. Generally, the corrosivity potential to a structure can be assessed based on the soil resistivity / electrical conductivity, hydrogen ion concentration (pH), and salts (chloride and sulphate) concentrations. 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.

6.9.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-19 Table 3 for potential sulphate attack on concrete. The sulphate concentrations measured in one tested sample was <0.01% (i.e., below the lower Method Reporting Limit for the procedure carried out). 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.

6.9.2 Potential for Corrosion

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results indicate a pH value of 9.3 and a resistivity of 4,348 ohm-cm. As per Gravity Pipe Design Guidelines (MTO, 2014), the measured resistivity at this site indicates a low potential for corrosion while in general, while a pH reading that is highly alkaline, as measured at this site, it is indicative of an increased potential for corrosion which should be considered in the design.

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.

6.10 Construction Considerations

6.10.1 Subgrade Preparation and Approach Embankment Construction

Prior to construction of the nominal grade raise and associated widening of the approach embankments, it is recommended that existing vegetation and topsoil be stripped from the existing embankment side slopes and replaced with OPSS.PROV 1010 Select Subgrade Material (SSM), Granular A or Granular B soils, such that the permanent embankment side slopes are maintained no steeper than 2H:1V.

To reduce surface water erosion on the widened embankment side slopes, establishment of topsoil and vegetative cover as per OPSS.PROV 803 should be carried out as soon as possible after completion of the embankment grade raise and widening.

6.10.2 Excavations and Temporary Protection Systems

Excavations will be required for the placement of the new lanes and northern abutment. Temporary excavations will be required for the construction of the new centre/north pier, as well as for removals of existing foundations where applicable.

All temporary excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended. The existing fill layers are classified as Type 3 soils. The native dense silty sand and glacial till deposits are classified as Type 2 soils, and the very dense glacial till are classified as Type 1 soils. Any soils impacted by groundwater or observed to be wet should be classified as Type 4 soils unless appropriate groundwater control is in place. Temporary excavations (i.e., those open for a relatively short time period) within Type 1 and Type 2 soils should be made with side slopes no steeper than 1H:1V, starting at a depth of 1.2 m. For Type 3 soils, the excavation should be made with side slopes no steeper than 1H:1V from the bottom of the trench to the surface. For Type 4 soils, the side walls should be slope at 3H:1V from the bottom of the trench.

At this stage, it is anticipated that Lake Road will be closed during the structure replacement, and therefore there will be sufficient space for open-cut excavations at the abutment and may be sufficient space for open-cut excavations at the new northern pier. However, temporary protection systems are likely to be required at the southern pier and to facilitate the extent of removals that may be required for the existing north and south piers adjacent to the existing highway shoulders. Where required, temporary protection systems must be designed and constructed in accordance with OPSS.PROV 539. For excavations adjacent to Highway 401 live lanes, the lateral movement of the temporary protection systems must meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent utilities can tolerate this magnitude of deformation. For excavations located further from Highway 401 where there is no risk of impact to existing structure foundations, utilities or travelled lanes, Performance Level 3 may be appropriate.

6.10.3 Groundwater Control

The highest groundwater level measured during the foundation investigation was at about Elevation 183.4 m in the monitoring well installed in Borehole Lake22-02 located on the north side of the highway. This water level is about 6.9 m below the Lake Road grade and 2.8 m below the Highway 401 grade, although higher water levels may occur seasonally and following periods of precipitation and snow melt.

At this preliminary stage it is anticipated that temporary excavations will be maintained above the groundwater table at the site; if localized “perched” water is encountered, it is anticipated that any groundwater seepage into the foundation excavations can be adequately controlled by ditching and pumping from filtered sumps within or adjacent to the excavations. Based on the groundwater level and proposed construction works, it is anticipated that neither registration on MECP’s Environmental Activity and Sector Registry (EASR) nor a Permit to Take Water (PTTW) will be required for construction at this site.

Surface water must always be directed away from excavations and must be properly diverted / controlled such that the integrity of any foundation subgrade is maintained.

6.10.4 Obstructions during Pile / Caisson Installation

During pile installation through the glacially derived soils, and in particular the “100-blow” till at this site, there is a risk of encountering cobbles and boulders, as indicated by auger grinding, soil cuttings, and cored boulders during drilling. Although pre-drilling will be required if H-piles or tube piles are adopted at the foundations, some nominal

pile driving would still occur and it is recommended that steel H-piles or tube piles be reinforced and protected from damage with appropriate driving shoes as per OPSD 3000.100 (Steel H-Pile Driving Shoe) or 3001.100 (Steel Tube Driving Shoe) or equivalent. Equipment and procedures for pre-drilling for H-piles or tube piles and for caissons are expected to be capable of penetrating and/or removing obstructions as may be required.

6.11 Recommendations for Additional Work

The preliminary foundation recommendations provided in this report are based on the subsurface information from two boreholes advanced near the proposed north and south abutments. Additional subsurface investigation is recommended to be carried out during detail design to confirm the subsurface soil and groundwater conditions at the piers as well as for any retaining walls that may be incorporated adjacent to the abutments. Boreholes should be advanced into the “100-blow” glacial till which was encountered below approximately Elevation 185 m near the north abutment, and Elevation 180 m near the south abutment, and it will be necessary to extend the boreholes more than 3 m into “100-blow” soil to obtain information for longer/deeper piles or caissons that extend sufficiently below the Highway 401 cut grade. The foundation types, sizes and geotechnical resistances should be reassessed and revised as necessary and the need for dewatering reassessed at that time.

Additional foundation investigation and design should meet the general requirements outlined in the latest version of the *Guideline for MTO Foundation Engineering Services*. It is recommended that the existing standpipe piezometer (installed in Boreholes Lake22-02) be maintained operational to allow for continued monitoring of the groundwater level during detail design and up to construction, at which time the piezometer will need to be decommissioned in accordance with Ontario Regulation 903 (as amended).

It may be beneficial, depending on the proposed replacement plan, to carry out Multi-Channel Analysis of Surface Wave (MASW) ground profiling or vertical seismic profiling (VSP) from within new boreholes to assess the average shear wave velocity of the 30 m of soil/bedrock beneath at the proposed abutment/pier foundation locations. Geophysics testing such as MASW or VSP may provide a more favourable average shear wave velocity, for the Seismic Performance Category assessment during detailed design.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ben Waechter, EIT at WSP Golder and reviewed by Kenton Power, P.Eng., a senior geotechnical engineer with WSP Golder. Lisa Coyne, P.Eng., a Fellow and MTO Designated Foundations Contact for WSP Golder, conducted an independent technical and quality review of this report.

Signature Page


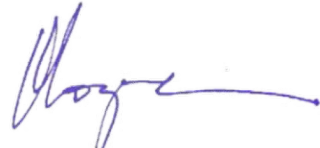
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BW/KCP/LCC/ljv

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REFERENCES

- Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.
- Canadian Geotechnical Society. 2006. *Canadian Foundation Engineering Manual (CFEM)*, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
- Chapman, L.J. and Putnam, D. F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.
- Kulhawy, F.H. and Mayne, P.W. 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.
- National Resources Canada, 2020. Seismic Hazard Tool. <https://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php>
- Terzaghi, K.V., 1955. Evaluation of Coefficient of Subgrade Reaction. *Getechnique*, 5(4): 297-326.
- Terzaghi, K. and Peck, R.B., 1967. *Soil Mechanics in Engineering Practice*, 2nd Edition, John Wiley and Sons, New York.
- Unified Facilities Criteria, U.S. Navy. 1986. *NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures*. Alexandria, Virginia.

ASTM International

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

Canadian Standards Association (CSA):

CAN/CSA-S6-19, 2019. *Canadian Highway Bridge Design Code (CHBDC)* and *Commentary on*. CSA Group.

CSA A23.1-19/A23.2-19, 2019. Concrete materials and methods of concrete construction / Test methods and standard practices for concrete.

Ministry of Transportation Ontario

Gravity Pipe Design Guidelines, Circular Culverts and Storm Sewers, April 2014.

MTO Foundations Guideline, Embankment Settlement Criteria for Design, July 2010.

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

Guideline for MTO Foundation Engineering Services, Version 3, dated April 2022.

Ontario Provisional Standard Drawing:

OPSD 208.010 Benching of Earth Slopes

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 Specification:

OPSS.PROV 206 Construction Specification for Grading

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 539 Temporary Protection Systems

OPSS.PROV 803 Construction Specification for Vegetative Cover

OPSS.PROV 902 Construction Specification for Excavating and Backfilling - Structures

OPSS.PROV 903 Construction Specification for Deep Foundations

OPSS.PROV 1010 Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material

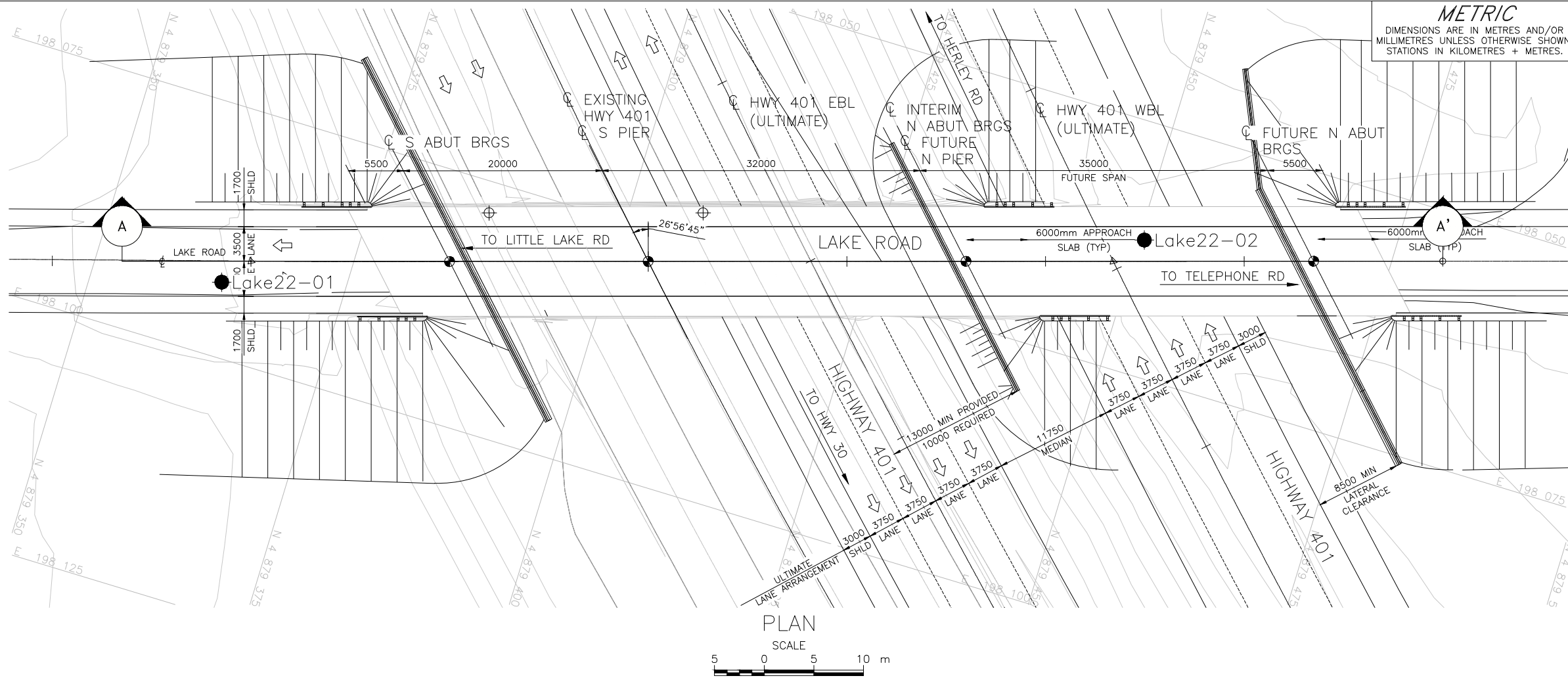
Ontario Provincial Regulations

O.Reg. 213 Construction Projects (as amended)

O.Reg. 903 Wells (as amended)

Table 9: Comparison of Foundation Alternatives

Option	Feasibility	Advantages	Disadvantages	Risks/Consequences	Relative Costs
Shallow strip footing	<ul style="list-style-type: none">Feasible but not preferred at this site given the structure grade (significant downward slope from north to south) and the interim two-span to ultimate three-span configuration	<ul style="list-style-type: none">Conventional construction methodsVery dense soil present at relatively shallow depth, requiring limited depth of excavation at piers and offering relatively high axial geotechnical resistances	<ul style="list-style-type: none">Deeper excavations, with associated protection systems and dewatering, required at abutmentsGenerally larger footprint required at piersMay require larger/deeper footprint, incorporation of keys or dowels to address lateral loading requirements associated with grade along the structure and the interim/final structure configuration	<ul style="list-style-type: none">Limited or negligible risk of post-construction settlementSome constructability and staging challenges associated with footprint of pier excavations relative to traffic staging, and depth of excavation at abutments	<ul style="list-style-type: none">In general, lower cost than deep foundations, although this is partially offset by more significant protection system and dewatering costs
Steel H-piles or tube piles installed in pre-drilled holes extended into "100-blow" till	<ul style="list-style-type: none">Feasible and a technically preferred solution for support of abutments.Not recommended at piers due to presence of "100-blow" soil at shallow depth below Highway 400 grade; pre-drilling would be required and for this reason, drilled shafts (caissons) would be preferable	<ul style="list-style-type: none">Permits the abutment pile caps to be maintained higher, with lesser excavation and associated protection systems and dewatering compared with a shallow foundation option Negligible post-construction settlement with piles founded in 100-blow till	<ul style="list-style-type: none">Would require pre-drilling to permit abutment piles to extend to sufficient depth into "100-blow" till, introducing a second operation and equipment typeGeotechnical resistances are low given relatively short pile length, although these can be increased by pre-drilling deeper	<ul style="list-style-type: none">Negligible risk of post-construction settlementLow risk of penetrating through the till given pre-drilling requirement, and associated relatively low risk of damage due to cobbles and boulders	<ul style="list-style-type: none">Higher cost than shallow foundations, although partially offset by reduced protection system and dewatering costs at abutmentsDriven piles can be less expensive than drilled shafts, but the requirement for pre-drilling at the abutments would increase the cost of pile foundations
Drilled shafts (caissons)	<ul style="list-style-type: none">Feasible and preferred for piers given shallow depth to "100-blow" tillFeasible and a technically preferred solution at abutments, as an alternative to pre-drilled H-piles or tube piles	<ul style="list-style-type: none">Permits the abutment pile caps to be maintained higher, with lesser excavation and associated protection systems and dewatering compared with a shallow foundation optionRelatively short caissons required given shallow depth to "100-blow" tillAffords relatively high geotechnical resistances with fewer deep foundation elements compared with smaller pre-drilled piles; higher capacities can be achieved with deeper drillingAllows for elimination of below-grade pile caps at the piers, affording a smaller final below-grade footprint	<ul style="list-style-type: none">Temporary liners and/or polymer slurry required for construction to mitigate against soil disturbance and base heave, and to permit inspection and cleaning of the baseConcrete would have to be placed by tremie methods below the water /slurry level, although this is a standard construction technique	<ul style="list-style-type: none">Relatively low risks associated with caisson construction in these soil conditions under OPSS.PROV 903 or Ministry's special provision for higher complexity caissonsNegligible risk of post-construction settlement provided caissons bases are properly cleaned and inspected via SID or SQUID	<ul style="list-style-type: none">Generally much higher cost than shallow foundations, although this is partially offset by reduced protection system and dewatering costs, and may also be offset by elimination of below-grade pile cap at piersGenerally higher cost than driven steel H-piles, although this is offset by requirement for pre-drilling for piles at this site, and also be offset by elimination of below-grade pile cap at piers

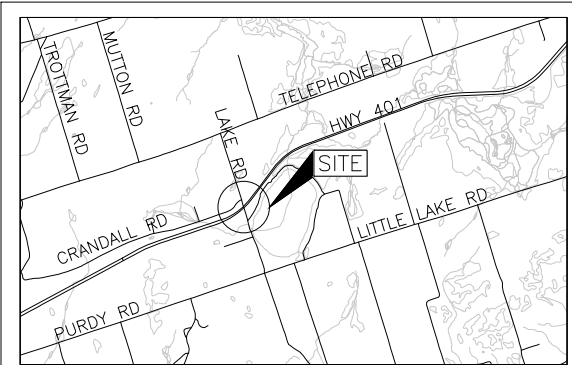


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.4054-17-00

REPLACEMENT OF HIGHWAY 401
UNDERPASS AT LAKE ROAD

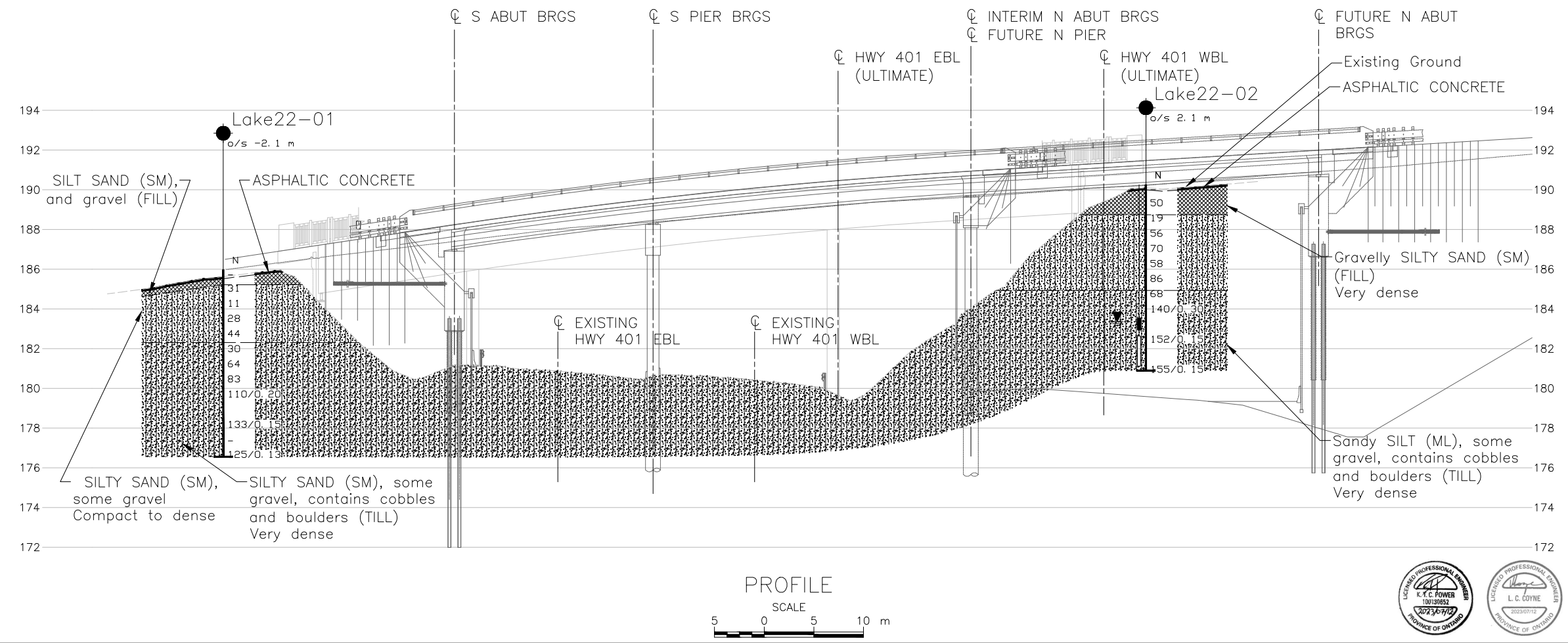
BOREHOLE LOCATIONS AND SOIL STRATA



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 July 21, 2022.



BOREHOLE CO-ORDINATES NAD 83 (CSRS)/MTM ZONE 9			
No.	ELEVATION	NORTHING	EASTING
Lake22-01	186.0	4879362.9	198092.6
Lake22-02	190.3	4879450.5	198061.6

Structural Site Location Latitude: 44.04650 Longitude: -77.83158

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

Geocres No. 31C-322			
HWY. 401	PROJECT NO. 1773612		DIST. EASTERN
SUBM'D. BW	CHKD. KCP	DATE: 7/11/2023	SITE: 21X-0295/B0
DRAWN: ZS	CHKD. LCC	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	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
		2.00 to 4.75	(10) to (4)
SAND	Coarse	0.425 to 2.00	(40) to (10)
	Medium	0.075 to 0.425	(200) to (40)
	Fine		
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>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

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

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

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Cone Penetration Test (CPT)

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

Dynamic Cone Penetration Resistance (DCPT); N_d:

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

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

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness¹

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

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 grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

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.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index = $(w_L - w_P)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_P) / I_P$
I_C	consistency index = $(w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	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 = $\tan \delta$
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

Notes: 1
2

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

PROJECT		RECORD OF BOREHOLE No Lake22-01										SHEET 1 OF 1		METRIC								
G.W.P.		4054-17-00		LOCATION		N 4879362.9; E 198092.6 MTM NAD ZONE 9 (LAT. 44.046500; LONG. -77.831580)						ORIGINATED BY		BW								
DIST		Eastern HWY 401		BOREHOLE TYPE		Power Auger, 200 mm Dia. (Hollow Stem)						COMPILED BY		TR								
DATUM		GEODETIC		DATE		July 19, 2022						CHECKED BY		KCP								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)				
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PROJECT		RECORD OF BOREHOLE					No Lake22-02		SHEET 1 OF 1		METRIC					
G.W.P.		4054-17-00		LOCATION		N 4879450.5; E 198061.6 MTM NAD ZONE 9 (LAT. 44.047280; LONG. -77.831990)					ORIGINATED BY		BW			
DIST		Eastern HWY		401		BOREHOLE TYPE		CME 55 Truck Mounted					COMPILED BY		TR	
DATUM		GEODETIC		DATE		July 19, 2022					CHECKED BY		KCP			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
190.3	GROUND SURFACE							20	40	60	80	100				
0.0	ASPHALT (100 mm)															
0.1	Silty sand (SM), and gravel (PAVEMENT STRUCTURE) (FILL)		1	AS	-											
189.5	Brown Moist															
0.8	Gravelly SILTY SAND (SM) (FILL)		2	SS	50											31 57 (12)
	Very dense Brown to grey Moist															
188.8																
1.5	Gravelly SILTY SAND (SM), contains cobbles and boulders (TILL)		3	SS	19											
	Compact to very dense Brown to white Moist															
			4	SS	56											
			5	SS	70											27 52 (21)
			6	SS	58											
			7	SS	86											
185.0																
5.3	Sandy SILT (ML), some gravel, contains cobbles and boulders (TILL)		8	SS	68											11 35 (54) NP
	Very dense Brown Moist to wet															
	- Cobbles and boulders from 6.3 m to 9.4 m		9	SS	40/0.36											
			10	SS	52/0.15											
180.9			11	SS	55/0.15											
9.4	END OF BOREHOLE Auger Refusal															
	NOTES: 1. Water level in screen measured at a depth of 6.9 m (Elev. 183.4 m) on July 21, 2022.															

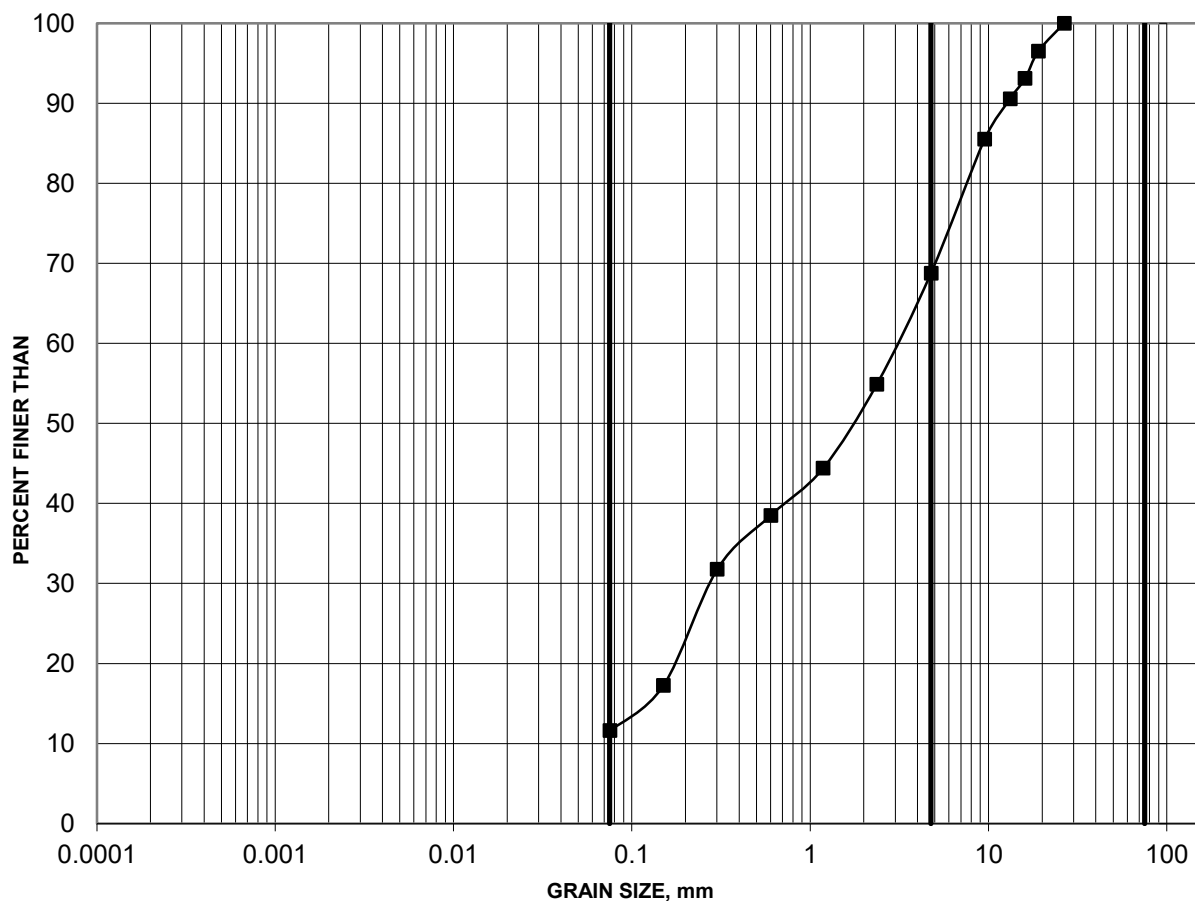
APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B1

Gravelly SAND (SW-SM) (FILL)



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

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■ L-22-02	2	0.76-1.37	31	57	12	

Project: 1773612-WO11



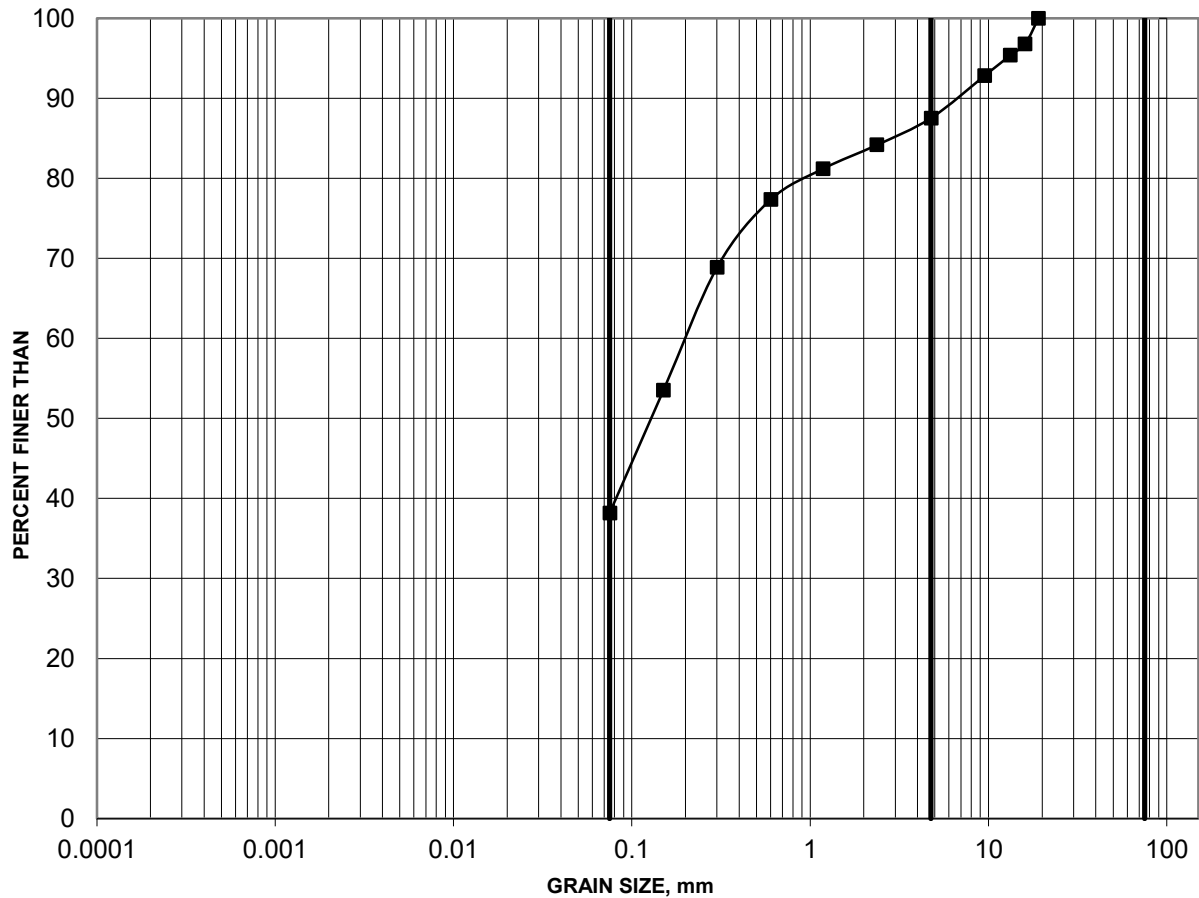
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GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY SAND (SM)



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

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■ L-22-01	4	2.29-2.90	12	50	38	

Project: 1773612-WO11



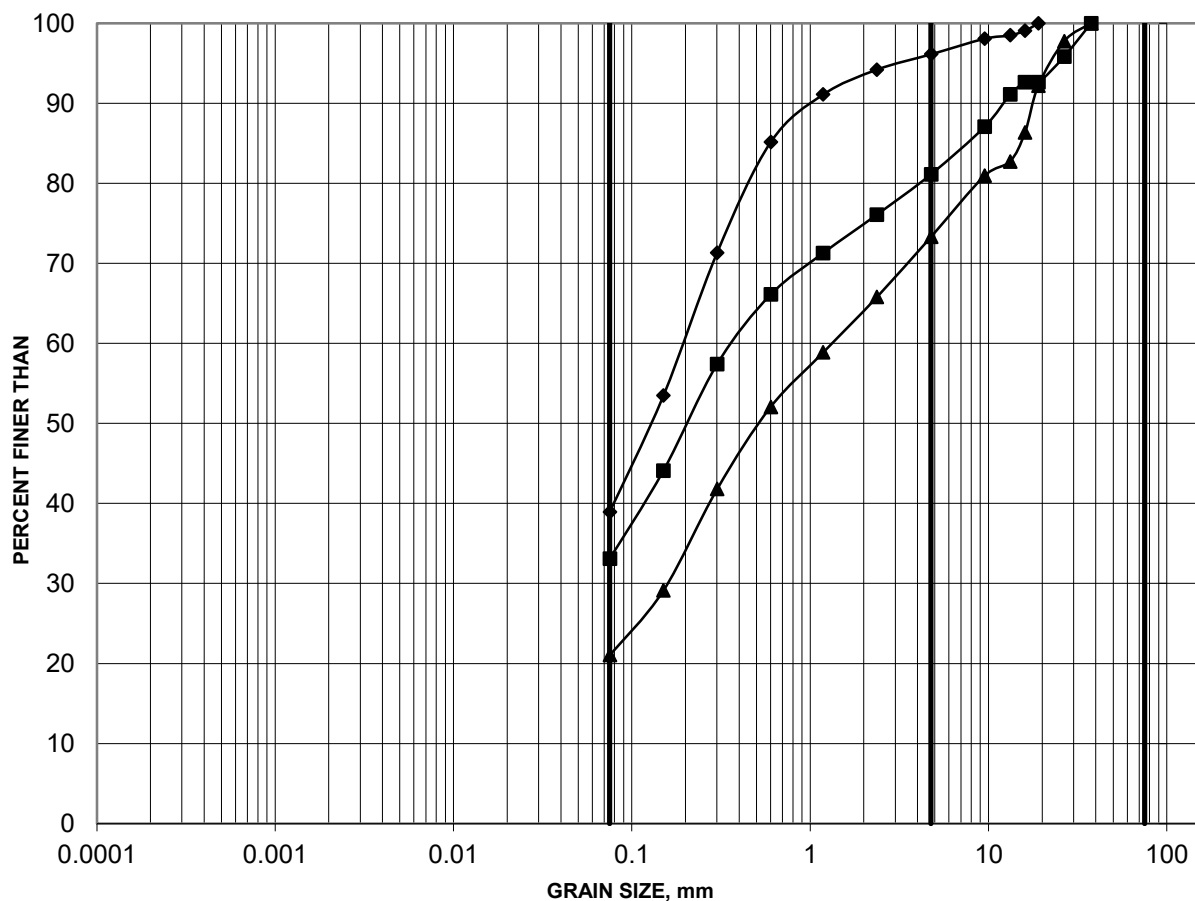
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GRAIN SIZE DISTRIBUTION

FIGURE B3

Gravelly SILTY SAND (SM) to SILTY SAND (SM) (TILL)



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

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■	L-22-01	7	4.57-5.18	19	48	33
◆	L-22-01	10	7.62-7.92	4	57	39
▲	L-22-02	5	3.05-3.66	27	52	21

Project: 1773612-WO11



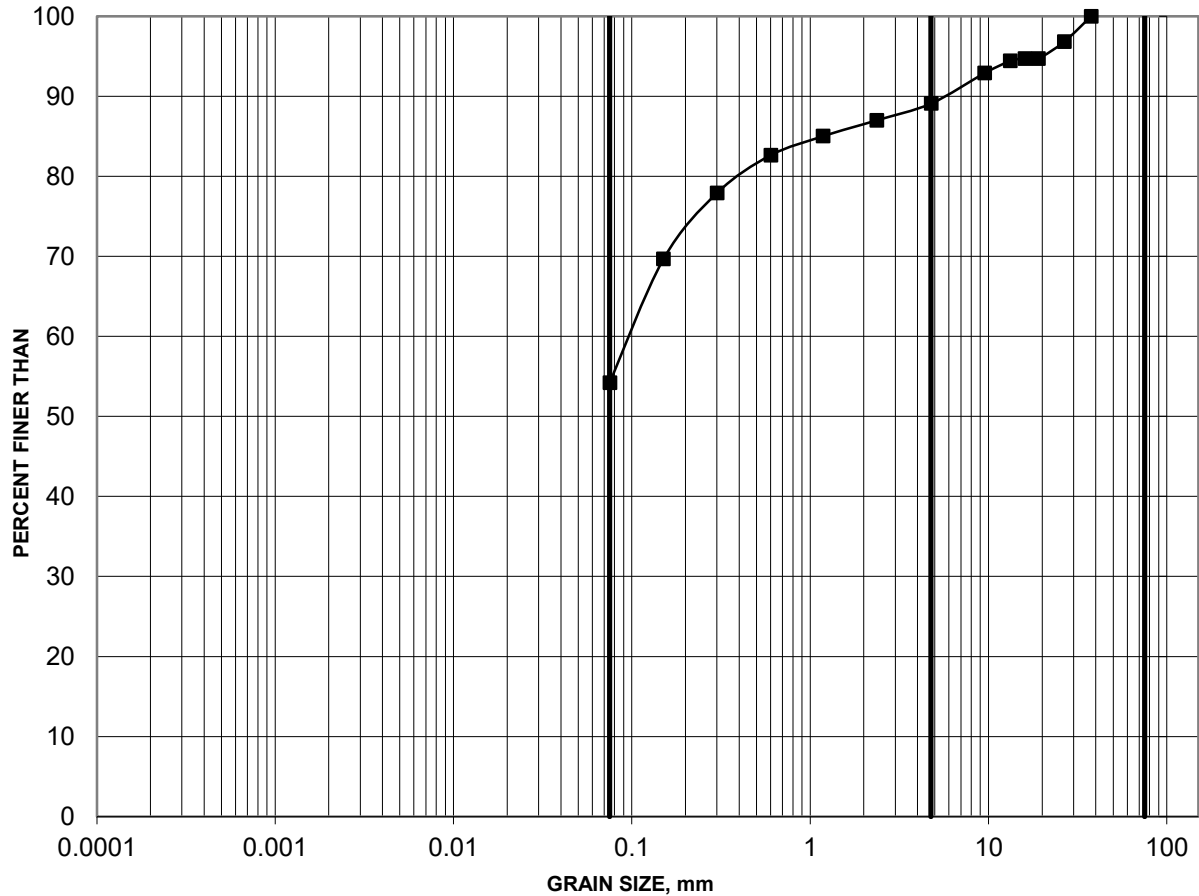
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GRAIN SIZE DISTRIBUTION

FIGURE B4

non-plastic SILT (ML), and sand (TILL)



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

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■ L-22-02	8	5.33-5.94	11	35	54	

Project: 1773612-WO11



Created by: BW

Checked by: CW

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2017/1773612/Lake Rd/figures/>

APPENDIX C

Analytical Laboratory Test Results



Certificate of Analysis

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

Page 1 of 3

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

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	0.24
	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	0.26	0.89
	pH	2.00				8.15	9.01	8.15
	Resistivity	1	ohm-cm			1618	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

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

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



Photograph 1: Looking south from Borehole Lake22-02



Photograph 2: Looking north from Borehole Lake22-01



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