

FINAL REPORT

Preliminary Foundation Investigation and Design Report

Replacement of Highway 401/Herley Road Underpass (Structure Site 21-294)

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/HERLEY ROAD UNDERPASS
(STRUCTURE SITE 21-294)
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) has been retained by WSP Canada Inc. (WSP) on behalf of the Ministry of Transportation, Ontario (MTO) 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 in Northumberland County, Ontario. The overall project includes the replacement of three underpass structures and four structural culverts.

This report presents the results of the preliminary foundation investigation carried out for the replacement of the Highway 401/Herley Road Underpass (MTO Structure Site No. 21-294).

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/Herley Road underpass site is located in the Township of Cramahe in Northumberland County. Herley Road is located approximately 1.6 km east of Percy Street, Colborne, and approximately 10.5 km west of County Road 30, Brighton. The site location is shown on the key plan on Drawing 1.

Herley Road is an undivided road with a single travel lane in each direction over Highway 401. Steel beam guiderails are present along both side of Herley Road in the vicinity of the underpass structure. The typical existing cross section of Highway 401 through the area consists of two lanes and a paved shoulder in each direction, separated by a paved median and a concrete tall wall barrier. The area is surrounded by agricultural land with a rolling topography.

At the site, Highway 401 has been constructed near the original ground surface, with its grade at approximately Elevation 171 m to 171.5 m. Herley Road has been constructed on embankment fill, with its grade at approximately Elevation 178.5 m to 176.5 m, declining to the south across the structure; the north and south approach embankments are up to approximately 6.5 m high relative to the surrounding ground surface. Based on visual observation at the time of the investigation, there is no visual evidence of structure deformation or embankment instability.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out in July 2022 and included advancing two boreholes (H22-01 and H22-02) through Herley 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. (CCC) of Ottawa, Ontario. Soil samples were obtained using a 50 mm outer diameter split-spoon sampler in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586¹). Soil samples were obtained at vertical sampling intervals of about 0.76 m to 1.5 m.

A monitoring well was installed in Borehole H22-02 to observe the stabilised groundwater level at the site. The monitoring well consists of 52 mm outside diameter PVC tubing with a 1.5 m long slotted screen; well installation details are shown on the borehole record in Appendix A. The borehole without a monitoring well was backfilled

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

with bentonite within the bedrock, and bentonite mixed with soil cuttings within the overburden, in general accordance with the intent of Ontario Regulation (O.Reg.) 903, as amended. The site conditions were restored following completion of the fieldwork.

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, in accordance with MTO and/or ASTM Standards, as applicable.

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

The borehole locations and elevations were surveyed by WSP 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 the following table.

Borehole No.	MTM NAD 83 Northing (m) (Latitude, °)	MTM NAD 83 Easting (m) (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
H22-01	4877411.9 (44.028336)	193955.5 (-77.882790)	178.5	15.4 ¹
H22-02	4877338.7 (44.027682)	193983.6 (-77.882420)	176.2	14.1 ¹

Notes: ¹ Borehole terminated within glacial till

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The Herley Road underpass lies in the physiographic regions known as the Iroquois Plain, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)².

The Iroquois Plain physiographic region extends around the western part of Lake Ontario, from Niagara River to Trent River. The width of the plain varies from a few hundred meters to approximately 13 km north of the current Lake Ontario shoreline, and it extends inland to include a large area in the Trent River valley. In the area east of Colborne, the surficial glaciolacustrine deposits of the plain consist of sand, gravelly sand and gravel, and nearshore and beach deposits.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes including piezometer installation details and water level readings, and the results of the in-situ and laboratory tests are provided on the

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

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 in Section 4 are uncorrected and are based on use of an automatic hammer. The detailed results of the geotechnical laboratory testing on soil samples are presented on the laboratory test figures in Appendix B. The results of the analytical testing are provided in Appendix C.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

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 Herley Road embankment, underlain by a relatively thin deposit of dense to very dense sand, which is further underlain by non-cohesive glacial till deposits comprising silty sand to sandy silt containing cobbles and boulders, up to the termination depth of the boreholes. More detailed descriptions of the major soil layers encountered in the boreholes are provided in the following sections.

4.2.1 Existing Pavement Structure

An approximately 100 mm thick layer of asphalt pavement was encountered at the ground surface in both boreholes. Approximately 0.8 m of granular material consisting of gravel and sand was encountered beneath the asphalt in both boreholes.

4.2.2 Sand and Gravel Fill

Underlying the existing pavement structure, non-cohesive fill consisting of sand and gravel containing trace non-plastic fines was encountered in Boreholes H22-01 and H22-02. This fill was encountered at a depth of 0.8 m below ground surface in both boreholes, and was about 5.9 m and 6.8 m thick extending to Elevation 170.9 m and 169.5 m at the borehole locations north and south of Highway 401, respectively.

The SPT ‘N’-values measured within this fill range from 19 to greater than 100 blows per 0.3 m of penetration indicating a compact to very dense state of compactness. Within the fill layers, auger grinding was observed between depths of 1.5 m and 6.1 m in Borehole H22-01, and this is interpreted to represent the presence of gravel, cobbles and/or boulders within the fill; in addition, the high SPT ‘N’-values (greater than 100 blows per 0.3 m of penetration, and 39 blows without penetration) are considered to represent the presence of gravel, cobbles and/or boulders and may not represent the state of compactness of the fill matrix.

The water content measured on select samples of the fill ranges between approximately 2% and 4%. The results of grain size distribution carried out on five samples of the sand and gravel fill are shown on Figure B1 in Appendix B.

4.2.3 Sand

A relatively thin layer of sand, some to trace gravel was encountered at a depth of 7.6 m below ground surface (Elevation 170.9 m) in Borehole H22-01 north of Highway 401, and at 6.7 m below ground surface (Elevation 169.5 m) in Borehole H22-02 south of Highway 401. The thickness of this layer ranges from 1.2 m to 3.1 m as encountered in the boreholes. Auger grinding was observed within this layer between the depths of 6.7 m to 7.6 m in Borehole H22-02 and is interpreted to represent the presence of zones of gravel, cobbles and/or boulders.

The SPT 'N'-values measured within the sand layer ranges from 30 to 92 blows per 0.3 m of penetration indicating a dense to very dense state of compactness.

The water content measured on one sample of the sand is approximately 9%. The result of a grain size distribution test is shown on Figure B2.

4.2.4 Sandy Silt to Silty Sand Till

Glacial Till consisting of sandy silt to silty sand, trace gravel to gravelly was encountered at a depth of 8.8 m (Elevation 169.7 m) in Borehole H22-01 and at a depth of 9.8 m (Elevation 166.5 m) in Borehole H22-02 on the north and south sides of Highway 401, respectively; the boreholes were terminated in this deposit after penetrating it for 4.3 m to 6.4 m.

Frequent auger grinding was observed in both boreholes, suggesting the possibility of cobbles and boulder with the glacial till deposit. The SPT 'N' values measured in the till range from 43 to greater than 100 blows per 0.3 m penetration suggesting a dense to very dense state of compactness.

The water content measured on samples of the non-cohesive till range from 6% to 12%. The results of grain size distribution testing carried out on three samples of the till are presented on Figure B3.

4.3 Groundwater Conditions

A standpipe piezometer was installed in Borehole H22-02 to allow monitoring of the groundwater level at this site. The groundwater level recorded in the piezometer is shown on the borehole record in Appendix A and is summarized below.

Borehole No.	Screened Interval	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Groundwater Elevation (m)	Date
H22-02	Glacial Till	176.2	10.0	166.2	21 July 2022

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 Testing Results

One soil sample was submitted for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix C and the test results are summarized below.

Borehole No.	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
H22-02	0.8-1.4	0.005	0.01	0.25	9.89	4000

5.0 CLOSURE


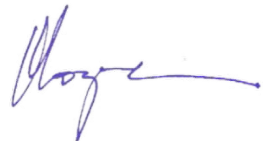
This Preliminary Foundation Investigation Report was prepared by Mr. Ali Khan, a Geotechnical Engineer at WSP Golder, and reviewed by Mr. Kenton Power, P.Eng., Senior Geotechnical Engineer. Ms. Lisa Coyne, P.Eng., a Fellow and MTO Foundations Designated Contact with WSP Golder, conducted a technical and quality review of the report.

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[https://golderassociates.sharepoint.com/sites/11407g/wo11_colborne_to_brighten/3_reporting/2-herley_road/3-final/1773612_rev0_final_pfdr_2023'07'12_herley_\(gwp_4054-17-00\).docx](https://golderassociates.sharepoint.com/sites/11407g/wo11_colborne_to_brighten/3_reporting/2-herley_road/3-final/1773612_rev0_final_pfdr_2023'07'12_herley_(gwp_4054-17-00).docx)

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
REPLACEMENT OF HIGHWAY 401/HERLEY ROAD UNDERPASS
(STRUCTURE SITE 21-294)
TOWNSHIP OF CRAMAHE, NORTHUMBERLAND COUNTY
MTO GWP 4054-17-0, AGREEMENT NO. 4016-E-0034-011**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for planning and preliminary design of the Herley Road Underpass. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced as part of the current preliminary investigation.

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 factual 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 April 2022), the proposed Herley Road underpass replacement will be maintained along the existing Herley Road alignment. The existing three-span structure is proposed to be replaced with a two-span structure with a total length of approximately 64 m, with a centre pier located in the Highway 401 median. The new north and south abutments are proposed to be located immediately behind the existing abutment pile caps; it is anticipated that the existing pile caps will need to be removed, and there will be relatively low risk of interference with the existing pile foundations.

Highway 401 will be widened from four lanes to eight lanes. The existing Highway 401 grade will be maintained, while the Herley Road grade will be raised by approximately 0.5 m to about Elevation 179 m at the north abutment and Elevation 177 m at the south abutment, such that the approach embankments are up to approximately 7 m in height adjacent to the abutments.

6.3 Foundation Options

Based on the proposed two-span configuration with span lengths of approximately 30 m and 31 m and the subsurface conditions encountered at the site, both shallow and deep foundation options have been considered for support of the new abutments and centre piers. The preliminary recommendations provided herein will be subject to change subject to future investigations and testing in detail design, and when the geotechnical resistance factors may be increased on the basis of such additional investigation.

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 1 following the text of this report.

- **Strip footings:** Shallow foundations “perched” within the approach embankments on a compacted granular pad or founded on the dense to very dense sand or glacial till at the abutments or pier are feasible at this site. However, use of “perched” abutment footings would likely require longer bridge spans to allow the abutment footings to be founded appropriately relative to the abutment foreslopes, and so may encounter

conflicts with and/or require removal of the existing abutment pile caps and portions of the piles. Abutment footings founded on native soil would require significant excavations and temporary protection systems, which are also expected to encounter conflicts with the existing abutment foundations. Therefore, founding the abutments on shallow foundations is not preferred from a geotechnical perspective. Subject to further investigation in detail design to confirm the shallow subsoil conditions, a strip footing at the median pier is likely to be a preferred option given the generally dense to very dense nature of the soils at this site. However, a strip footing is 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.

- **Driven piles:** Steel H-piles or tube piles driven into the “100-blow” glacial till are preferred for the abutments from a geotechnical/foundations perspective as they allow the pile caps to be perched within the approach embankments, thus minimizing excavation and temporary protection system requirements. A single row of vertical piles is also likely to minimize or eliminate conflicts with the existing abutment piles for the currently proposed 71 m long bridge. At the centre pier, while driven piles are nominally feasible, the very shallow depth to “100-blow” soil poses a risk to the piles being misaligned. Pre-drilling may be used in these conditions, although such operations would require additional large equipment in the median working area, and may require pre-drilling on a batter. Between these conditions and the relatively larger working footprint required for the pier pile cap, a centre pier foundation supported on driven steel piles is not preferred from a geotechnical perspective.
- **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 an excellent alternative to a strip footing for support of the centre pier, and would permit elimination of a below-grade pile cap for support of the structural columns. Caissons are not preferred at the abutments from a geotechnical perspective as they preclude the use of integral abutments, and are more likely to encounter conflicts with the existing abutment foundations for the currently proposed span configuration. 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.

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 structure and its foundation system may be classified as having medium traffic volumes and its performance as having potential impacts on other transportation corridors, resulting in a “typical consequence level” associated with exceeding limit states design. The consequence factor, Ψ , has been taken as 1.0 per Table 6.1 of CHBDC 2019.

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”; however, a “low degree of site understanding” has been assessed for geotechnical design of shallow foundations at the centre pier at this stage. The applicable ultimate limit state

(ULS) and serviceability limit state (SLS) geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , for typical and low degrees of site understanding per Table 6.2 of CHBDC 2019 have been used at this stage of preliminary design.

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

During detail design, additional investigation and testing would be required to increase the site understanding and modify the geotechnical resistance factors as appropriate.

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 (assumed to be existing ground surface), 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 CHBDC (2019) and based on the structure location, the Class C peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca) are provided in the following table.

Parameter	2% Probability of Exceedance in 50 years (2,475-year return period) (g)
PGA	0.150
Sa(0.2)	0.306
Sa(0.5)	0.200
Sa(1.0)	0.110
Sa(2.0)	0.0521
Sa(5.0)	0.0139
Sa(10.0)	0.00478
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 in the table above and, in accordance with Table 4.10 of CHBDC (2019) the bridge would fall in Seismic Performance Category 2 regardless of fundamental period of the structure.

Geophysics testing such as vertical seismic profiling or multi-channel analysis of surface waves may provide a more favourable average shear wave velocity, and hence the SPC assessment in detail 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 – *Foundation Frost Penetration Depths for Southern Ontario*).

6.5 Shallow Foundations

Strip or spread footings founded below the existing fills on the native dense to very dense sand or glacial till at or below the approximate elevations identified below, or founded on a compacted Granular 'A' pad perched within the existing approach embankment, are considered feasible for support of the proposed abutments and piers.

The following geotechnical resistances may be used for preliminary design assuming a 3 m or 5 m wide footing founded on the native soil. The values for footings perched within the approach embankments assume a minimum 3 m thick pad comprised of compacted OPSS.PROV 1010 Granular A or Granular B Type II is incorporated below the footing level.

Foundation Element	Founding Stratum	Maximum (Highest) Founding Elevation (m)	Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance ² (kPa)
North Abutment ¹	Perched on granular pad	176.0	3	900	350
	Dense sand over dense to very dense till	170.5	3	800	500
			5	950	350
Centre Pier ¹	Dense to very dense sand over dense to very dense till	169.5	3	650	450
			5	800	300
South Abutment ¹	Perched on granular pad	174.0	3	900	350
	Very dense silty sand over very dense till	169.5	3	750	600
			5	900	400

Notes:

1. Geotechnical resistance factors for a “typical” degree of site understanding have been used for the abutment values, while those for a “low” degree of site understanding have been used at the pier.
2. For 25 mm of settlement.

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width, founding elevation, and thickness of granular pad (as applicable) and as such, the geotechnical resistances must be reviewed and revised if the footing width varies from that specified above or if the founding soils differ from that given in the previous section. 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, and using geotechnical resistance factors for a typical degree of understanding at the centre pier at that time.

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.

6.6 Driven Steel H-Pile or Tube Foundations

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. The depth to “100-blow” soil is interpreted to be within 2 m to 3 M below the Highway 401 grade at the centre pier, which poses a higher risk of piles being misaligned and which would likely require pre-drilling for placement of piles. Hence, driven piles are not recommended at the centre pier.

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 fill and native soils as auger grinding was frequently observed within these deposits. In this regard, 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 accordance 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.

The following factored geotechnical resistances may be used for preliminary design:

Foundation Element	Pile Type	Approximate Pile Length (m)	Estimated Pile Tip Elevation (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ² (kN)
North Abutment	HP 310x110 or 324 mm dia. tube	10	164	1,300	>1,300
South Abutment	HP 310x110 or 324 mm dia. tube	8.5	163	1,300	>1,300

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 Ontario Provincial Standard Specification (OPSS) PROV 903 (*Deep Foundations*) 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 pipes (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 investigation and testing in detail design.

Foundation Element	Estimated Caisson Base Elevation (m)	Caisson Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ² (kN)
North Abutment	164	1.2 1.5	7,000 10,000	>7,000 >10,000
Centre Pier	163	1.2 1.5	4,500 7,000	>4,500 >7,000
South Abutment	163	1.2 1.5	7,000 10,000	>7,000 >10,000

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 (Deep Foundations). Where caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner is required to support the soils during construction, to reduce disturbance and loss of ground in the water-bearing cohesionless soils. Specialized construction techniques would be required during advancement of the caisson to maintain a sufficient head of water and/or drilling fluid (polymer slurry) within the liner 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

The replacement of the Herley Road underpass is proposed to be completed on the existing alignment, with a grade raise of up to approximately 0.5 m. Limited additional fill will be required to be placed on the existing embankment side slopes associated with this grade raise, to maintain a 2 horizontal to 1 vertical (2H:1V) side slope configuration; it is recommended that existing vegetation and topsoil on the side slopes be stripped prior to placement of this fill to minimize the potential for surficial erosion and sloughing prior to re-establishment of vegetation on the side slopes. As the approach embankments are estimated to up to approximately 7 m in height (i.e., below 8 m high), mid-height benches are not required to be incorporated.

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

Stratigraphic Unit	γ' (kN/m ³)	ϕ' (°)	E' (MPa)
Existing compact sand and gravel fill	21	32	--
Dense to very dense silty sand to sand	20	32	50-100
Dense to very dense till	21	34	200

6.8.1 Global Stability

Minimum target Factors of Safety of 1.3 and 1.5 are considered appropriate for global stability of the approach embankment slopes 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 including an approximately 0.5 m grade raise and nominal widening, with side slopes maintained no steeper than 2H:1V will have an adequate factor of safety of greater than 1.3 and 1.5 in short-term and long-term conditions, respectively.

6.8.2 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-A23.1-14 Table 3 ("*Additional requirements for concrete subjected to sulphate attack*") 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 samples 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 test results indicate a pH value of 9.9 and a resistivity of 4000 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to concrete durability. However, the resistivity

indicates that the soil corrosiveness is Moderate ($4500 \text{ ohm-cm} > R > 2000 \text{ ohm-cm}$) to Severe ($R < 2000 \text{ ohm-cm}$), 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.

6.10 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 Select Subgrade Material (SSM), Granular A or Granular B soils, such that the permanent embankment side slopes are maintained no steeper than 2 horizontal to 1 vertical (2H:1V).

To reduce surface water erosion on the widening 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 Temporary Excavations and Temporary Protection Systems

Temporary excavations will be required construction of the new abutments and centre 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 to 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 Herley Road will be closed during the structure replacement, and therefore there may be sufficient space for open-cut excavations at the abutment. However, temporary protection systems are likely to be required at the centre 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 (*Temporary Protection System*) and Special Provision 105S09. 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.

6.10.3 Groundwater Control

The highest groundwater level measured during the foundation investigation was at about Elevation 166.2 m in the monitoring well installed in Borehole H22-02 located on the south side of the highway. This water level is about 10 m below the Herley Road grade and 5 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. In particular, surface water drainage on the west side of the site must be properly diverted / controlled such that the integrity of any foundation subgrade is maintained.

6.10.4 Obstructions during Pile Driving / 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 during drilling. 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. Caisson installation equipment and procedures 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 centre pier 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 168 m near the north abutment, and Elevation 166 m near the south abutment. 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 H22-02) 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).

7.0 CLOSURE


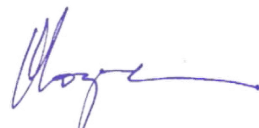
This Preliminary Foundation Design Report was prepared by Mr. Ali Khan, P.Eng., Geotechnical Engineer, and reviewed by Mr. Kenton Power, P.Eng., Senior Geotechnical Engineer with WSP Golder. Ms. Lisa Coyne, P.Eng., a Fellow and MTO Foundations Designated Contact for WSP Golder, also conducted a technical and quality review of this report.

Signature Page

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AK/KCP/LCC/ljv

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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

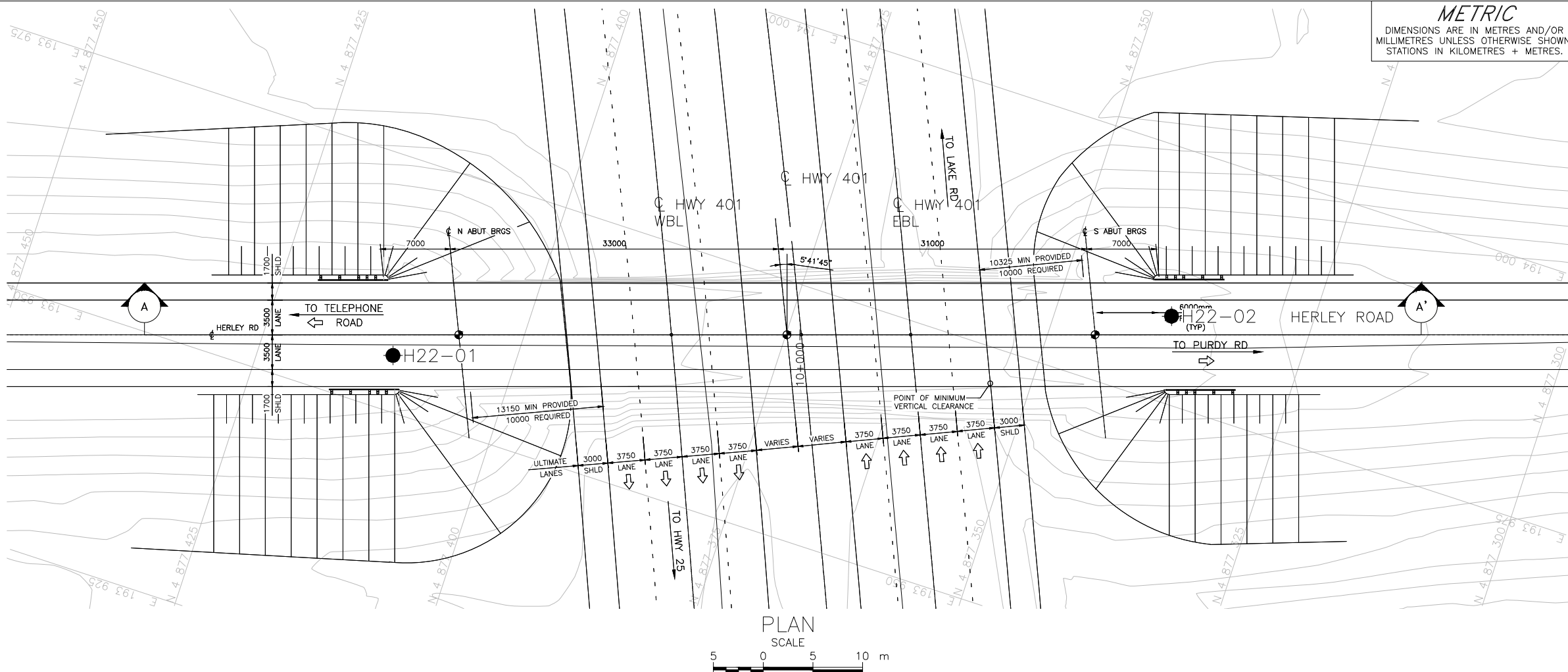
OPSS.PROV 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Regulations

O.Reg. 213	Construction Projects (as amended)
O.Reg. 903	Wells (as amended)

Table 1: Comparison of Foundation Alternatives – Herley Road Underpass

Foundation Option	Feasibility	Advantages	Disadvantages	Risk / Consequences	Relative Costs
Strip / spread footings founded on dense to very dense sand or glacial till	<ul style="list-style-type: none">■ Feasible but not preferred at abutments due to depth of excavation through Herley Road■ Feasible at centre pier (subject to further investigation in detail design), although a wider foundation footprint may be required compared to caisson foundations	<ul style="list-style-type: none">■ Conventional construction for a “closed” structure configuration■ Competent soils present at shallow depth will provide suitable geotechnical resistances for design■ Foundation excavations are anticipated to be maintained above the groundwater level, minimizing dewatering requirements	<ul style="list-style-type: none">■ Relatively deep excavations required at abutments relative to existing Herley Road grade, with associated protection systems; potential for conflict between protection systems and existing abutments■ Temporary protection systems also required for median pier■ Does not allow for integral abutment design	<ul style="list-style-type: none">■ Negligible risk related to settlement performance of new structure■ Low to moderate risks associated with construction of protection systems if half-and-half staging is adopted, including potential for conflicts with existing abutment foundations	<ul style="list-style-type: none">■ Generally lower cost compared to deep foundations; however, this will be offset by costs for more significant excavation and protection systems
Strip footings perched on a granular pad in approach embankments	<ul style="list-style-type: none">■ Feasible but not preferred at abutments due to likely requirement for longer bridge deck	<ul style="list-style-type: none">■ Conventional construction for an “open” structure configuration■ Competent soils present within and below approach embankments, providing suitable geotechnical resistances for design and acceptable settlement performance of structure■ Minimizes depth of excavation through existing Herley Road embankment fill, as compared with strip footings in a “closed” structure configuration	<ul style="list-style-type: none">■ Would require a longer-span structure to place footings appropriately relative to abutment foreslopes, with associated greater costs■ Perched footings in a longer span configuration may conflict with existing abutment foundations, requiring more extensive removals	<ul style="list-style-type: none">■ Low risk related to settlement performance of new structure (there is some potential for a zone of weaker/looser fill to be present below the perched footings)■ Moderate risk of conflicts with existing abutments requiring more extensive removals	<ul style="list-style-type: none">■ Generally lower cost compared to deep foundations; however, this may be offset by costs for longer bridge deck and more extensive removals compared to other options
Steel H-piles or tube piles driven into “100-blow” till deposit	<ul style="list-style-type: none">■ Feasible and preferred for perched abutments; permits integral abutments and a single row of vertical piles would minimize conflicts with existing abutment foundations■ Not recommended at centre pier where pre-drilling would be required due to shallow depth to “100-blow” soil	<ul style="list-style-type: none">■ Conventional construction methods■ Pile caps may be “perched” within approach embankment fill, reducing excavation and temporary protection system requirements compared to strip footing option, and likely minimizing conflicts with existing abutment foundations■ Allows for integral abutment design; corrugated steel pipe (CSP) liners would be required over the upper 3 m of the piles in this case, and augering would be required for installation of such CSPs	<ul style="list-style-type: none">■ Shallow depth to “100-blow” soil at centre pier would necessitate pre-drilling to ensure pile penetrates to appropriate tip elevation without damage or obstruction, introducing a second operation/equipment type in the median area during construction■ Larger working area generally to accommodate battered piles to resist horizontal forces; pre-drilling on a batter can be technically challenging	<ul style="list-style-type: none">■ Negligible risk related to settlement performance of new structure■ Low to moderate risk of pile damage/deflection if cobbles and boulders are encountered during pile driving, particularly given relatively short pile length; this can be mitigated with use of driving shoes and/or pre-drilling in very dense soils	<ul style="list-style-type: none">■ Generally more expensive than shallow foundations, although this will be offset by reduced excavation and protection system costs
Drilled shafts (caissons) founded within very dense glacial till	<ul style="list-style-type: none">■ Feasible at abutments, although this would preclude integral abutments and may have greater potential for conflict with existing abutment foundations■ Preferred at centre pier as this option may permit direct support of structural columns and elimination of below-grade pile cap	<ul style="list-style-type: none">■ Offers higher geotechnical resistance per foundation element compared to driven steel piles, requiring fewer foundation elements■ Requires a smaller footprint for construction in constrained working areas, as compared with multiple rows of vertical or battered piles■ May be designed to eliminate below-grade pile cap at the centre pier, with each caisson supporting a structural column, minimizing final footprint and costs associated with forming and pouring below-grade cap	<ul style="list-style-type: none">■ Temporary liner or polymer slurry required to counteract water pressures and eliminate risk of ground disturbance/loss during caisson installation, along with use of tremie concrete; however, this operation is not considered highly complex or challenging at this site■ Does not allow for conventional integral abutment design if used at the abutments.	<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 caissons■ Negligible risk of post-construction settlement provided caisson bases are properly cleaned and inspected via SID or SQUID	<ul style="list-style-type: none">■ Generally higher cost than shallow foundations, although this can be offset by reduced excavation and protection system costs if below-grade pile cap can be eliminated at centre pier■ Generally higher relative cost compared with driven piles, but at this site it is anticipated that caisson construction will cost less than driven piles due to fewer required elements and opportunity to eliminate below-grade pile cap and protection systems

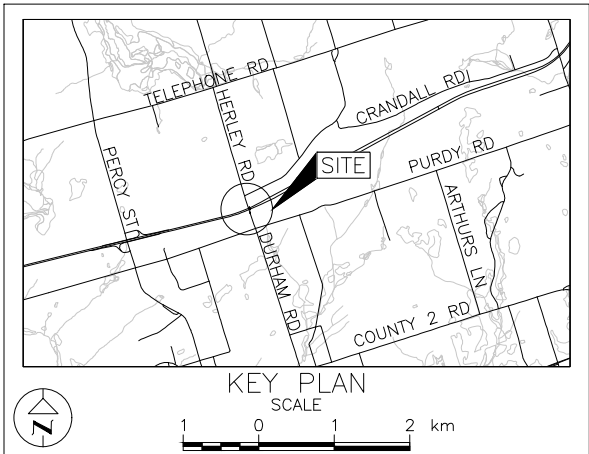


METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4054-17-00

HWY 401 AND HERLEY ROAD
UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



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
H22-01	178.5	4877411.9	193955.5
H22-02	176.2	4877338.7	193983.6

Structural Site Location Latitude: 44.02833 Longitude: -77.88279

NOTES

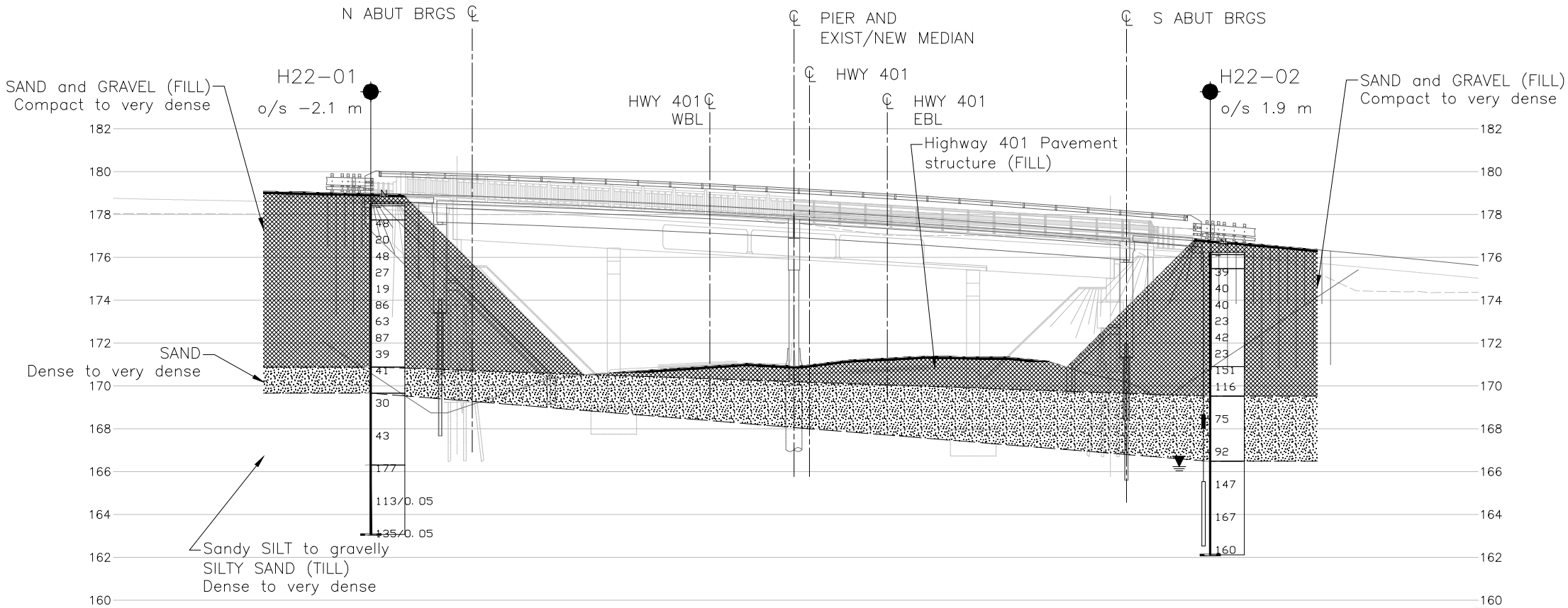
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

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. S17M-0172-11-301-001XG.dwg, received April 5, 2022, and general arrangement drawing file no. S17M-0172-11-301-001GA.dwg, received February 2023.

NO.	DATE	BY	REVISION
Geocres No. 31C-320			
HWY. 401	PROJECT NO. 1773612		DIST. EASTERN
SUBM'D. KCP	CHKD. KCP	DATE: 7/11/2023	SITE: 21-294
DRAWN: ZS	CHKD. AK	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 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve friction (f_s) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

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

SOIL TESTS

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

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

COARSE-GRAINED SOILS

Compactness¹

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

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 $= \sigma'_p / \sigma'_{vo}$

(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

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

PROJECT <u>1773612</u>		RECORD OF BOREHOLE No H22-01				SHEET 2 OF 2		METRIC	
G.W.P. <u>4054-17-00</u>		LOCATION <u>N 4877411.9; E 193955.5 MTM NAD ZONE 9 (LAT. 44.028336; LONG. -77.882790)</u>				ORIGINATED BY <u>BW</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>CME 55 200 mm OD Hollow Stem Auger then Wash Boring</u>				COMPILED BY <u>TR</u>			
DATUM <u>GEODETIC</u>		DATE <u>July 21, 2022</u>				CHECKED BY <u>KCP</u>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			W _L	
166.3 12.2	--- CONTINUED FROM PREVIOUS PAGE --- (SM) gravelly SILTY SAND, contains cobbles and boulders (Glacial TILL) Dense to very dense Brown Wet - Auger grinding from 12.2 m to 13.7 m		14	SS	177	166								o				21 46 25 8
						165												
			15	SS	13/0.05	164												
163.1 15.4	END OF BOREHOLE		16	SS	135/0.05													
	NOTES: 1. Borehole advanced using Hollow Stem Augers to 12.8 m depth; below this depth, borehole advanced with casing and wash boring. 2. Groundwater level measured at 12.3 m in open borehole prior to start of wash boring.																	

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PROJECT		RECORD OF BOREHOLE				No H22-02		SHEET 1 OF 2		METRIC			
G.W.P.		4054-17-00		LOCATION		N 4877338.7; E 193983.6 MTM NAD ZONE 9 (LAT. 44.027682; LONG. -77.882420)		ORIGINATED BY		BW			
DIST		Eastern HWY 401		BOREHOLE TYPE		CME 55 200 mm OD Hollow Stem Auger		COMPILED BY		TR			
DATUM		GEODETIC		DATE		July 20, 2022		CHECKED BY		KCP			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W _P W W _L		
176.2	0.0	GROUND SURFACE		1A									
0.1		ASPHALT (100 mm)		1B	AS	-							
175.4	0.8	(GW-GM) GRAVEL and SAND (PAVEMENT STRUCTURE) (FILL) Brown Moist											
		(SW-SM) SAND and GRAVEL, trace fines (FILL) Compact to dense Brown to white Moist		2	SS	39							
				3	SS	40							
				4	SS	40							
				5	SS	23							
				6	SS	42							
				7	SS	23							
170.9	5.3	(SW-SM) SAND and GRAVEL, some silt, contains cobbles and boulders (FILL) Very dense Brown Moist		8	SS	151							
				9	SS	116							
169.5	6.7	(SP) SAND, some silt, some gravel Very dense Brown Moist to wet											
		- Auger grinding from 6.7 m to 7.6 m		10	SS	75							
				11	SS	92							
166.5	9.8	(SM) SILTY SAND, some non-plastic fines, trace gravel, contains cobbles and boulders (Glacial TILL) Very dense Brown Wet											
				12	SS	147							

Continued Next Page

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

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PROJECT 1773612		RECORD OF BOREHOLE No H22-02				SHEET 2 OF 2		METRIC									
G.W.P. 4054-17-00		LOCATION N 4877338.7; E 193983.6 MTM NAD ZONE 9 (LAT. 44.027682; LONG. -77.882420)				ORIGINATED BY BW											
DIST Eastern HWY 401		BOREHOLE TYPE CME 55 200 mm OD Hollow Stem Auger				COMPILED BY TR											
DATUM GEODETIC		DATE July 20, 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 (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100						
	(SM) SILTY SAND, some non-plastic fines, trace gravel, contains cobbles and boulders (Glacial TILL) Very dense Brown Wet		13	SS	167												
162.1			14	SS	160												
14.1	END OF BOREHOLE																
	NOTES: 1. Borehole dry upon completion of drilling. 2. Water level measured in piezometer as follows: Date Depth Elevation (m) 2022-07-21 10.0 166.2																

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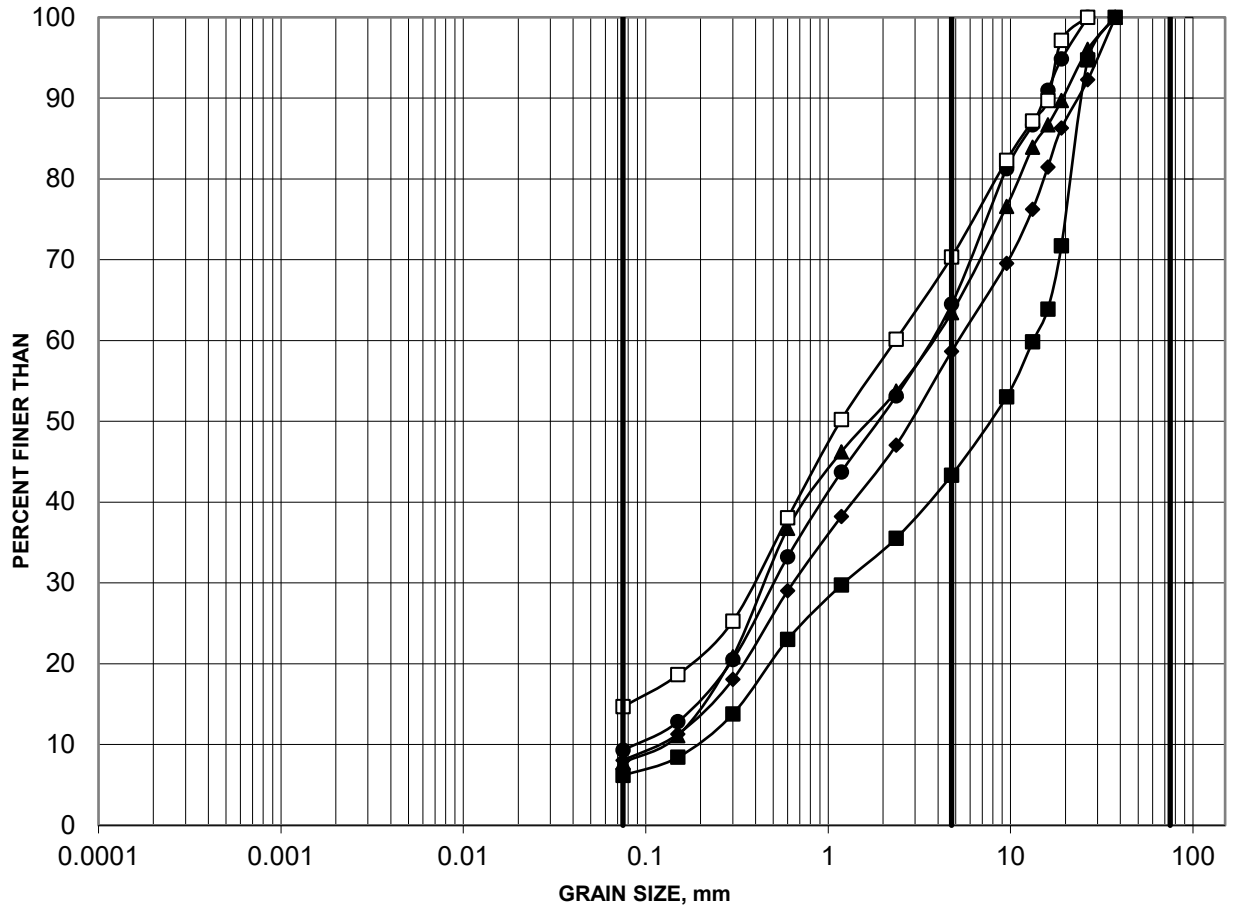
APPENDIX B

Geotechnical Laboratory Test Result

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND AND GRAVEL FILL



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

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	H-22-01	1	0.10-0.61	57	37	6	
◆	H-22-01	3	1.52-2.13	41	51	8	
▲	H-22-01	6	3.81-4.42	37	55	8	
●	H-22-02	4	2.29-2.90	36	55	9	
□	H-22-02	8	5.33-5.94	30	55	15	

Project: 1773612_WO11



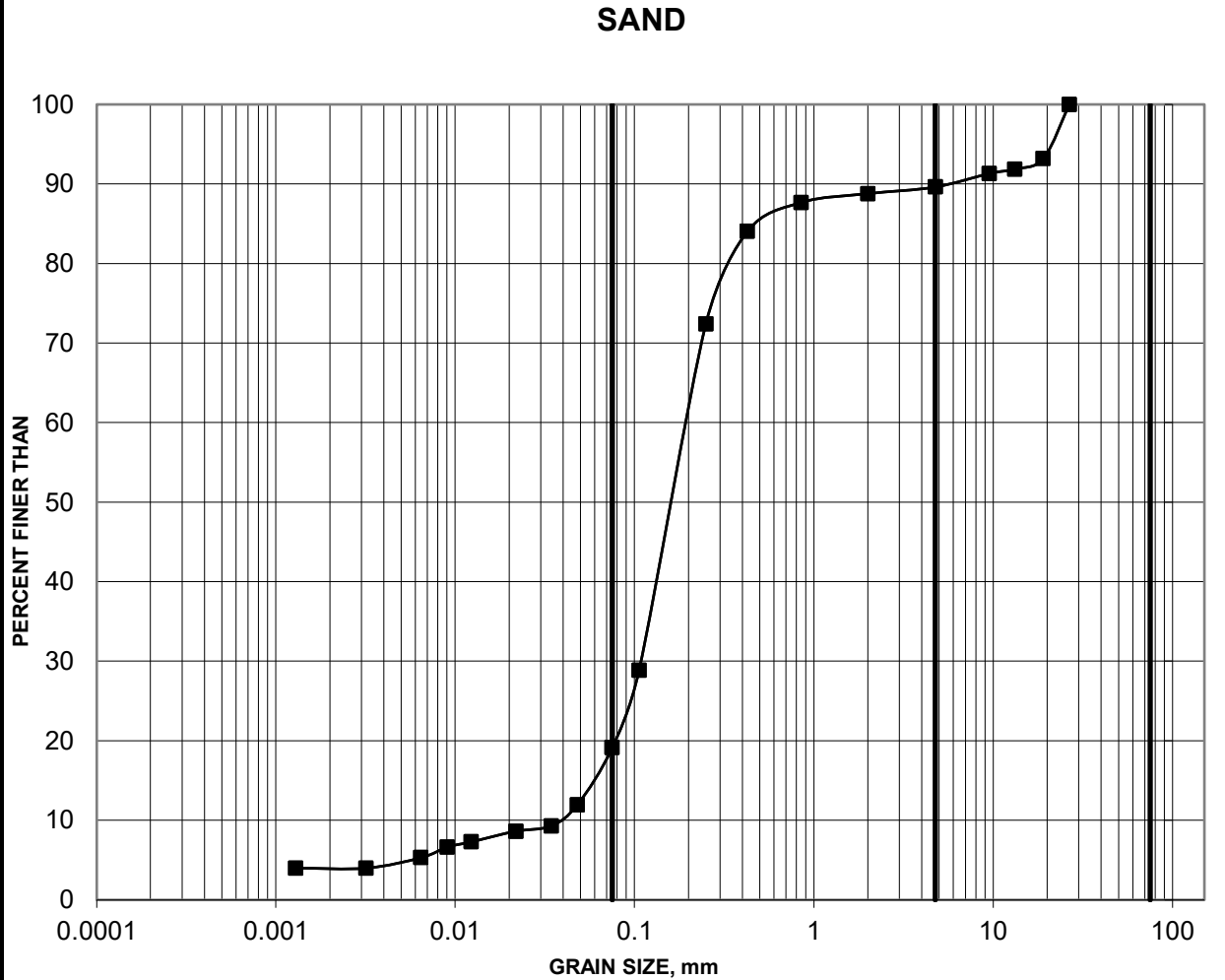
Created by: KG

Checked by: KCP

<https://golderassociates.sharepoint.com/sites/11407g/WO11> Colborne to Brighton/2. Technical Work/5. Lab Testing/2-Herley/Figures/

GRAIN SIZE DISTRIBUTION

FIGURE B2



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

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■	H-22-02	10	7.62-8.33	10	71	15

Project: 1773612_WO11



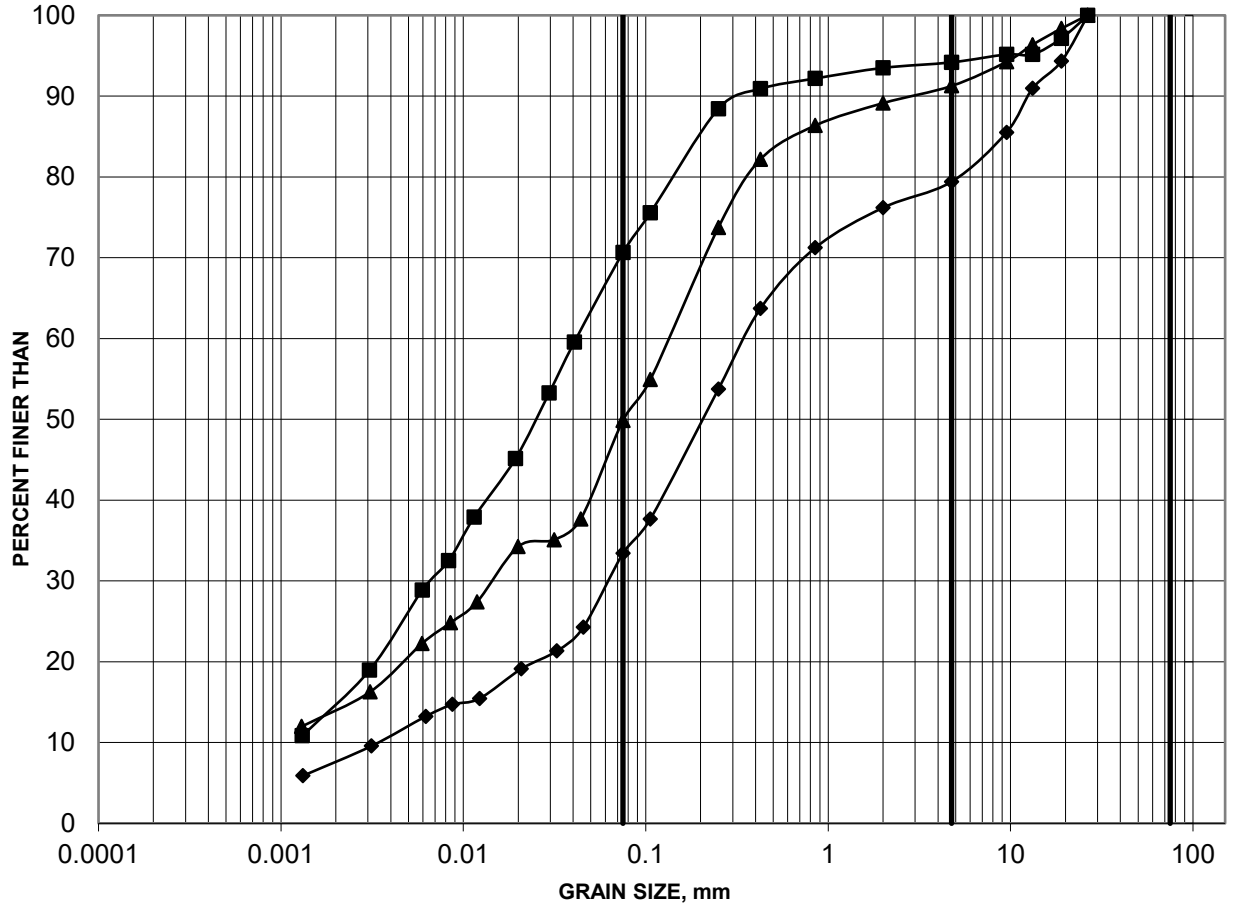
Created by: KG

Checked by: KCP

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND AND SILT TO GRAVELLY SILTY 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
■	H-22-01	12	9.14-9.75	6	23	56	15
◆	H-22-01	14	12.19-12.61	21	46	25	8
▲	H-22-02	12	10.67-11.10	9	41	36	14

Project: 1773612_WO11



Created by: KG

Checked by: KCP

<https://golderassociates.sharepoint.com/sites/11407g/WO11> Colborne to Brighton/2. Technical Work/5. Lab Testing/2-Herley/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

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Invoice to: Golder Associates Ltd

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

Group	Analyte	MRL	Units	Guideline	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'
Anions	Cl	0.002	%			0.058	0.005	0.007	0.016
	SO4	0.01	%			0.01	0.01	<0.01	0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm			1.27	0.25	0.23	0.44
	pH	2.00				8.88	9.89	9.32	9.24
	Resistivity	1	ohm-cm			787	4000	4348	2273

Group	Analyte	MRL	Units	Guideline	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'
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

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 towards Herley Road Underpass and Borehole H22-01



Photograph 2: Looking north towards Herley Road Underpass and Borehole H22-02



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