REPORT

Preliminary Foundation Investigation and Design Report

Replacement of Highway 401/County Road 26 Underpass
(Structure Site No. 21X-0297/B0)
Municipality of Brighton, 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/COUNTY ROAD 26 UNDERPASS (STRUCTURE SITE NO. 21-297) MUNICIPALITY OF BRIGHTON, 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 in Northumberland County, Ontario. 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/County Road 26 (CR26) Underpass (MTO Structure Site No. 21X-0297/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 Highway 401/CR26 site is located approximately 2.2 km east of County Road 30 in the Municipality of Brighton in Northumberland County, Ontario. The site location is shown on the key plan on Drawing 1.

At this location, Highway 401 has a four-lane cross-section with two eastbound and two westbound through lanes with paved shoulders separated by a concrete median wall. Steel beam guiderails are also present along both sides of the highway in the vicinity of the underpass structure. There are no interchange ramps at this location.

CR26 is an undivided road with a rural cross-section and a single travel lane in each direction that carries traffic over Highway 401 at a skew of approximately 30 degrees. Parapet walls with railing are present along the bridge and steel beam guiderails are present along both side of CR26 beyond the bridge.

The land surrounding the structure site is agricultural, with a rolling, hummocky topography. Highway 401 has been constructed partially in cut with the pavement grade at the structure site at approximately Elevation 196 m; this is lower than the natural ground surface immediately south of the highway, which is up to approximately Elevation 200 m. The CR26 grade is at approximately Elevation 201.5 m immediately adjacent to the existing bridge abutments; the existing approach embankments are approximately 5 m to 5.5 m high relative to the Highway 401 grade, although the south approach embankment consists of approximately 1 m to 2 m of fill relative to the surround natural ground surface.

The existing bridge was constructed in 1965 under MTO Contract 65-03. It is a four-span structure with perched abutments and piers founded on spread footings The Structural Design Report for Site 21X-0297/B0 indicates that the structure itself is in fair to good condition. Based on visual observation at the time of the investigation, there are no signs of embankment instability or approach embankment settlement.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between July 11 and July 14, 2022 and included advancing three boreholes (CR26-01 to CR26-03) through the travelled lanes of CR26. 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.

After sampling to a depth of approximately 18.9 m, Borehole CR26-02 was advanced to refusal without sampling, using Dynamic Cone Penetration Testing (DCPT). Borehole CR26-03, located approximately 1.5 m north of Borehole CR26-02, was augered without sampling to a depth of 19.8 m and was further advanced by SPT sampling to a termination depth of 33.7 m.

A monitoring well was installed at Borehole CR26-01 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 record for Borehole CR26-02 provided in Appendix A. The boreholes without a monitoring well were backfilled with 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 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, °)		
CR26-01	4882907.0 (44.079410)	205320.6 (-77.742080)	201.3	29.5[1]
CR26-02	4882820.7 (44.078630)	205303.7 (-77.74227)	201.3	26.8[2]
CR26-03	4882822.0 (44.078640)	205303.7 (-77.742270)	201.3	33.7[1]

Notes: [1] Borehole terminated within glacial till
[2] Borehole terminated at DCPT refusal

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario* Site 21-297 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 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 Subsurface Conditions

The subsurface soil, and groundwater conditions encountered in the boreholes and the results of in-situ testing from the investigation are shown on the borehole records presented in Appendix A. The results of the geotechnical laboratory testing are presented on the borehole records as well as on Figures B1 to B5 in Appendix B. The borehole locations and the interpreted stratigraphic profile projected along the proposed structure alignment are provided in Drawing 1.

The results of the basic chemical testing/analysis completed on a select soil sample are provided in Appendix C.

The stratigraphic boundaries shown on the borehole and drillhole records and on the interpreted stratigraphic section in Drawing 1 are inferred from observations of the drilling progress together with continuous soil sampling and may 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 CR26 embankment, underlain by a sand to silt to silt and sand, which is further underlain by a glacial till deposits comprising silty sand to silty gravel containing cobbles and boulders, up to the termination depth of the boreholes. A more detailed description of the overburden soil deposits, conditions 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 the boreholes. Approximately 0.1 to 0.2 m of granular material consisting of gravelly sand to sand and gravel was encountered beneath the asphalt in both boreholes that were sampled over this zone.

4.2.2 Fill

Underlying the existing pavement structure, a non-cohesive fill consisting of sand with varying amounts of gravel was encountered at all boreholes The top of this layer was encountered at Elevations 201.0 m and 201.1 m. The layer extends to Elevations 196.1 m and 199.8 m with thicknesses of 4.9 m and 1.3 m at Boreholes CR26-01 and CR26-02 respectively. The SPT ‘N’-values measured within this fill range from 22 to 88 blows per 0.3 m of penetration but are more typically greater than 30 blows to 56 blows indicating a generally dense to very dense state of compactness. Within the fill layers, the presence of gravel, cobbles and/or boulders were noted; in addition, the higher blow count (e.g., 88 blows per 0.3 m of penetration) is considered to represent the presence of cobbles and/or boulders and may not represent the state of compactness of the fill matrix.

The measured moisture contents of two samples of the fill were 5% and 8%. The results of grain size distribution testing carried out on two samples of the fill are shown on Figure B1 in Appendix B.

4.2.3 Upper Interbedded Sand to Silt

An interbedded non-cohesive deposit was encountered below the fill in Boreholes CR26-01 and CR26-02. The soils in this upper non-cohesive deposit vary in composition from sand containing trace to some silt, to silty sand, to silt and sand, to silt with varying proportions of gravel and/or clay. The top of this layer was encountered at Elevations 196.1 m and 199.8 m. This layer extends to Elevations 192.2 m and 192.6 m and is 3.9 m and 7.2 m in thickness at Boreholes CR26-01 and CR26-02 respectively.

The SPT ‘N’-values measured within the interbedded layers ranges from 12 blows to 48 blows per 0.3 m of penetration but more typically 16 blows to 26 blows indicating a generally compact state of compactness.

The measured moisture content of the tested samples of interbedded sand to silt layers ranges between approximately 2% to 20%. The results of grain size distribution testing carried out on five samples of the silt to sand and silt are provided in Figure B2 in Appendix B, while the result of grain size distribution testing on one sample of sand from this upper interbedded deposit is included on Figure B3 in Appendix B.

4.2.4 Sand

Sand with trace silt and gravel was encountered below the interbedded sand to silt layers in Boreholes CR26-01 and CR26-02. The top of this layer was encountered at Elevations 192.2 m and 192.6 m. The layer extends to Elevations 176.0 m and 171.4 m and is 16.2 m and 21.2 m in thickness at Boreholes CR26-01 and CR26-02 respectively. The SPT ‘N’-values measured within the sand ranges from 13 blows to 110 blows per 0.3 m of penetration but more typically 32 blows to 75 blows indicating a generally dense to very dense state of compactness.

The measured moisture content of tested samples ranges between approximately 2% and 15%. The results of grain size distribution carried out on four samples of this sand deposit are shown on Figure B3 in Appendix B (which also contains the grain size distribution test for one sample of sand from the upper interbedded layers).

4.2.5 Gravelly Silty Sand to Silty Gravel Till

A gravel and sand till with varying amounts of silt was encountered below the sand layer at all boreholes advanced at the site. The glacial till is described as consisting of a gravelly silty sand to silty gravel containing cobbles and boulders. The top of this layer was encountered at Elevations 176.0 m and 171.4 m. Boreholes CR26-01 and CR26-03 were terminated in this layer at Elevations 171.8 m and 167.6 m and Borehole CR26-02 was terminated at DCPT refusal at Elevation 174.5 m in inferred till.

The recorded SPT N-values were all greater than 100 blows per 0.3 m of penetration, suggesting a very dense compactness. The frequent spoon sampler refusals observed in Boreholes CR26-01 and CR26-03 suggests the possibility of cobbles and boulder that may have influenced the noted higher blow counts noted rather than the consistency of the soil matrix.

The water content measured on three samples ranged from 8% to 15%. The results of grain size distribution carried out on two samples of till are shown on Figure B4 in Appendix B. The results of Atterberg limits testing completed on a single sample of the till indicate a liquid limit of 17, plastic limit of 15 and plasticity index of 2. The Atterberg Limits analysis results are provided on Figure B5 in Appendix B and indicate that the fines portion of the till is a silt of low plasticity (ML).

4.3 Groundwater Conditions

A standpipe piezometer was installed at Borehole CR26-01 to measure the stabilized groundwater level at the site. The groundwater level recorded in the piezometer is shown on the borehole record in Appendix A and is summarized in Table 2.

Table 2: Summary of Groundwater Conditions

Borehole No.	Screened Interval	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Groundwater Elevation (m)	Date
CR26-01	Sand / Till	201.3	20.7	180.6	July 21, 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 Laboratory Testing Results

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

Table 3: Steel Corrosion and Sulphate Attack, Chemical Analysis

Borehole No.	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
CR26-01	1.5-2.1	0.058	0.01	1.27	8.88	787

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Kinjal Gajjar, a geotechnical consultant 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.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT REPLACEMENT OF HIGHWAY 401/COUNTY ROAD 26 UNDERPASS (STRUCTURE SITE NO. 21-297) MUNICIPALITY OF BRIGHTON, 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 / County Road 26 (CR26) Underpass (MTO Structure Site No. 21-297). 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 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 February 2023), the proposed CR26 underpass replacement will be maintained along the existing CR26 alignment. The existing four-span structure is proposed to be replaced with a two-span structure with a total length of approximately 70 m, with a centre pier located in the Highway 401 median. The structure will be maintained on the current CR26 alignment and will maintain the same skew angle (approximately 30°) relative to Highway 401. The new north and south abutments are proposed to be located immediately in front of the existing abutments, and the new centre pier is proposed to be located at the existing centre pier; removal of the existing abutments and centre pier footings is therefore expected to be required.

Highway 401 will be widened from four lanes to eight lanes. The existing Highway 401 grade will be maintained, while the CR26 grade will be raised by up to approximately 1 m, such that the approach embankments will be up to approximately 7.5 m in height (relative to the lowest surrounding grade for ditching adjacent to the embankments beyond the Highway 401 platform).

6.3 Foundation Options

Based on the proposed two-span configuration with span lengths of approximately 35 m each 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 pier. 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 based on such additional investigation.

Based on the high skew angle, it is understood that integral abutments are not feasible at this structure site. Further, for the proposed span arrangement, all foundation options will require removal of the existing abutment and centre pier spread footings.

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 are feasible at the proposed abutments and centre pier. Based on the proposed structure geometry, it is understood that abutment footings would need to be founded at or below approximately Elevation 197 m to avoid conflicts with the concrete slope paving in front of the abutments. At this level, the south abutment footing will be founded on compact to dense sand to silt; however, at the north abutment existing embankment fill remains at this level and some subexcavation would be required. 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 compact to 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 caisson 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. Driven steel piles are also feasible at the centre pier, although construction of battered piles for the pier would likely require more working space and present more constraints to traffic staging during construction as compared with a drilled shaft or spread footing option.

- **Drilled shafts (caissons):** Drilled shafts penetrating through the water-bearing sand deposit to extend into the “100-blow” glacial till are feasible at this site, this foundation type offers an excellent alternative to a strip footing for support of the center pier and would permit elimination of a below-grade pile cap for support of the structural columns. Caissons are also feasible at the abutments, particularly as the structure skew angle precludes the use of integral abutments; caissons could be adopted in conjunction with a perched pile cap in a similar configuration to that for driven piles. 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 bridge structure and its 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”; however, a “low degree of site understanding” has been assessed for geotechnical design of shallow foundations at the centre pier at this stage. 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.

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, $\overline{N}_{60}$, below the founding level (assumed to be existing ground surface), the site may be classified as Site Class D in accordance with Clause 4.4.3.2 and Table 4.1 of CHBDC (2019), in the absence of any geophysical testing.

The Preliminary Seismic Site Class D was determined based on correlations from the energy-corrected average SPT N_{60} values measured at the site. As outlined in Section 6.4.2.2 below, the higher Site Class D Spectral Values lead to a higher Preliminary Seismic Performance Categories (SPC) than that would be anticipated if the SPC was assessed with Site Class C Spectral Values.

It may be beneficial, depending on the proposed replacement plan, to carry out Multi-Channel Analysis of Surface Wave (MASW) or Vertical Seismic Profiling from within new boreholes to assess the average shear wave velocity, V_{s30}, of the 30 m of soil/bedrock beneath proposed abutment/pier foundation locations. It may be possible but not guaranteed to upgrade the Preliminary Seismic Site Class D based on the site-specific shear wave velocity profile.

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 D peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca) are provided in Table 4.

Table 4: Site Class D Spectral Values for Subject Site

Parameter	2% Probability of Exceedance in 50 years (2,475-year return period) (g)
PGA	0.203
Sa(0.2)	0.351
Sa(0.5)	0.335
Sa(1.0)	0.2
Sa(2.0)	0.0958
Sa(5.0)	0.0256
Sa(10.0)	0.00802
PGV [m/s]	0.218

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 CHBDC the bridge would fall in a SPC 3 if the fundamental period of the structure is less 0.5 s, or SPC 2 if the fundamental period of the structure is greater than or equal to 0.5 s.

As noted above, 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 compact to dense sand to silt to sand and silt, compact to very dense sand, very dense silty sand glacial till and hard clayey silt 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 Ontario Provincial Standard Drawing (OPSD) 3090.101.

6.5 Shallow Foundations

Strip or spread footings are considered feasible for support of the proposed abutments and centre pier. Based on the proposed structure geometry, it is understood that abutment footings would need to be founded at or below approximately Elevation 197 m to avoid conflicts with the concrete slope paving on the abutment foreslopes. At this level, the south abutment footing will be founded on compact to dense sand to silt; however, at the north abutment existing embankment fill remains at this level and approximately 1 m of subexcavation would be required to reach the native soil. This subexcavation should be backfilled with compacted OPSS.PROV 1010 Granular A or Granular B Type II.

The geotechnical resistances provided in Table 5 may be used for preliminary design assuming a 3 m or 5 m wide footing; for the north abutment footing, a minimum 1 m thick layer of compacted granular fill has been assumed in the assessment of these factored geotechnical resistances.

Table 5: Preliminary Factored Ultimate and Serviceability Geotechnical Resistances

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]	Requires 1 m of subexcavation to Elevation 196.0 m to reach native compact to dense sand to silt, and placement of compacted Granular A or Granular B Type II	197.0	3	700	400
			5	850	300
Centre Pier[1]	Compact to dense sand to silt to sand and silt over compact to very dense sand	193.8	3	600	425
			5	750	300
South Abutment[1]	Compact to dense sand to silt to sand and silt over compact to very dense sand	197.0	3	700	400
			5	850	300

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 compacted 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 that may permit use of 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 considered 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 factored geotechnical resistances that may be used for preliminary design our summarized in Table 6

Table 6: Preliminary Geotechnical Pile Design Recommendations

Foundation Element	Pile Type	Estimated Pile Tip Elevation (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance[1] (kPa)
North Abutment	HP310x110 or 324 mm dia. tube	173	1,600	>1,600
Centre Pier	HP310x110 or 324 mm dia. tube	170		
South Abutment	HP310x110 or 324 mm dia. tube	168		

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

Piles must be installed in accordance with OPSS.PROV 903 Section 903.07.02.07.03 Driving to a Specified Ultimate Resistance. The ultimate resistance should be verified by High-Strain Dynamic Testing (also referred to as Pile Dynamic Analyzer or PDA testing) on a minimum of 10% of piles at end of initial driving and on restrike.

For the installation of the steel H-piles or steel pipe piles, consideration must be given the presence of cobble and boulders within the fill and native soils. In this regard, steel H-piles are preferred over steel pipe piles, as pipe piles 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. As a result, piles should be fitted with appropriate driving shoes as per OPSS.PROV 903 Section 903.07.02.02 (Driving Shoes and Rock Points). 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. If piles are adopted, a Non-Standard Special Provisions should be developed during detailed design to be included in the Contract Documents to warn the contractor of the potential for the presence of obstructions (cobbles and boulders) in the overburden.

6.7 Drilled Shafts (Caissons)

Caissons founded within the very dense (“100-blow”) glacial till are feasible for supporting the abutments and piers. The geotechnical resistances provided in Table 7 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:

Table 7: Preliminary Geotechnical Drilled Shafts Design Recommendations

Foundation Element	Estimated Caisson Base Elevation (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance[1] (kN)
North Abutment	173	1.2 m dia: 13,000 1.5 m dia: 17,000	1.2 m dia.: >13,000 1.5 m dia.: >17,000
Centre Pier	170		
South Abutment	168		

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 CR26 underpass is proposed to be completed on the existing alignment, with a grade raise of up to approximately 1 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 in Table 8.

Table 8: Summary of Geotechnical Parameters

Stratigraphic Unit	γ' (kN/m^3)	ϕ' (°)	E' (MPa)
Existing compact to very dense gravelly silty sand to sand fill	21	32	--
Compact to dense sand to sand and silt to silt	19	32	50-80
Compact to very dense sand	20	32	50-100
Very dense silty sand to silty gravel 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 a grade raise of up to approximately 1 m and nominal widening, with side slopes maintained no steeper than 2H:1V, will have Factors 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-A23.1-19 Table 3 for potential sulphate attack on concrete. The sulphate concentrations measured in one tested sample was 0.01% and is below the exposure class of S-3 (Moderate). Therefore, based on the soil sample tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

6.9.2 Potential for Corrosion

The test results indicate a pH value of 8.9 and a resistivity of 787 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is considered detrimental to concrete durability. The resistivity indicates that the soil corrosiveness is Severe (2000 ohm-cm > R), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014), and appropriate corrosion protection should be applied to the foundation element / materials. Further, given that the foundations are located adjacent to the highway and may be exposed to de-icing salt, consideration should be given to selection of a “C” type exposure class as defined by CSA A23.1 Table 1.

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

6.10 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 2H:1V. It may be possible to use earth fill, including soils excavated from elsewhere on the future construction contract, for this widening; however, assessment of global and surficial stability should be completed at detailed design in consideration of likely available material types and timing for excavation/staging.

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 Temporary Excavations and Temporary Protection Systems

Temporary excavations will be required construction of the new abutments and centre pier, including removal of existing abutment and centre pier foundations.

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 compact to dense sand to silt to sand and silt, compact to very dense sand deposits are classified as Type 2 soils, and the very dense and hard 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 CR26 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 180.6 m in the monitoring well installed at Borehole CR26-01 located on the north side of the highway. This water level is about 20 m below the CR26 grade and 14 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 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. MTO Guidelines for Foundation Engineering Services generally recommend a minimum of two boreholes per foundation element, and therefore consideration should be given to advancing an additional borehole at each abutment, as well as at the centre pier and any significant 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 176 m near the north abutment, and Elevation 172 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 at Boreholes CR26-01) 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) at ground surface or Vertical Seismic Profiling (VSP) from within new boreholes to assess the average shear wave velocity, V_{s30}, of the 30 m of soil/bedrock beneath proposed abutment/pier foundation locations. Such data may support upgrading the Preliminary Seismic Site Class D, although improvement to Site Class C is not guaranteed.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Kinjal Gajjar, a geotechnical consultant 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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KG/KCP/LCC/ljv

https://golderassociates.sharepoint.com/sites/11407g/wo11 colborne to brighten/3. reporting/1-cr26/3-final/1773612 rev0 final pfidr 2023'07'12 - cr26 (gwp 4054-17-00).docx

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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 (MERO), 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 1004	Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Regulations

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

Table 9: Comparison of Foundation Alternatives – CR26 Underpass

Foundation Option	Advantages	Disadvantages	Risk / Consequences	Relative Costs
Strip / spread footings founded on dense to very dense sand or glacial till at abutments and pier, or on an engineered granular pad at abutments	■ Conventional construction ■ Competent non-cohesive soils will provide adequate geotechnical resistance and satisfactory total and differential settlement performance	■ May require deeper excavation at abutments compared to a deep foundation option with perched pile caps; some further subexcavation would be required at north abutment to extend below existing embankment fill ■ Generally larger footprint required at centre pier compared with caisson option ■ Temporary protection systems expected to be required within median for centre pier foundation excavation	■ Less competent near surface soils may be encountered during detail design investigation at centre pier, requiring deeper subexcavation ■ Otherwise, limited or negligible risk of post-construction settlement ■ Some constructability and staging challenges associated with footprint of pier excavations relative to traffic staging	■ Lower cost than deep foundations, although this may be offset by costs for deeper excavation and temporary protection systems
Steel H-piles or tube piles driven into "100-blow" soils (glacial till)	■ Conventional construction methods for H-pile foundations. ■ Pile caps may be "perched" within approach embankment fill to reduce excavation and protection system requirements ■ Negligible post-construction settlement with piles founded in 100-blow till	■ Larger working area required for driving battered piles within Highway 401 centre median	■ Negligible risk of post-construction settlement ■ Reduced impact on design if variable near-surface soils are encountered at centre pier during detailed investigation ■ Low to moderate risk of encountering cobbles and boulders during pile driving; potential for pile damage/deflection if cobbles and boulders are encountered during pile driving; slightly greater risk of pile damage/deflection for tube piles as compared with H-piles if cobbles/boulders are encountered during driving	■ Higher cost than shallow foundations, but generally lower relative cost than drilled shafts (caissons) at abutments ■ At centre pier, costs for driven piles with below-grade pile caps may be greater than that for a caisson foundation that can eliminate below-grade pile cap
Drilled shafts (caissons) founded within very dense glacial till	■ 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 pile cap and temporary excavations as the caissons could be cast continuously with structural columns to underside of superstructure	■ Temporary or permanent liner will be required, plus special measures such as use of polymer slurry to counterbalance hydrostatic head and groundwater pressures to reduce risk of loosening / softening of the sides of excavation and blow-out at base of shaft during drilling and concrete placement (by tremie methods).	■ 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	■ 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.

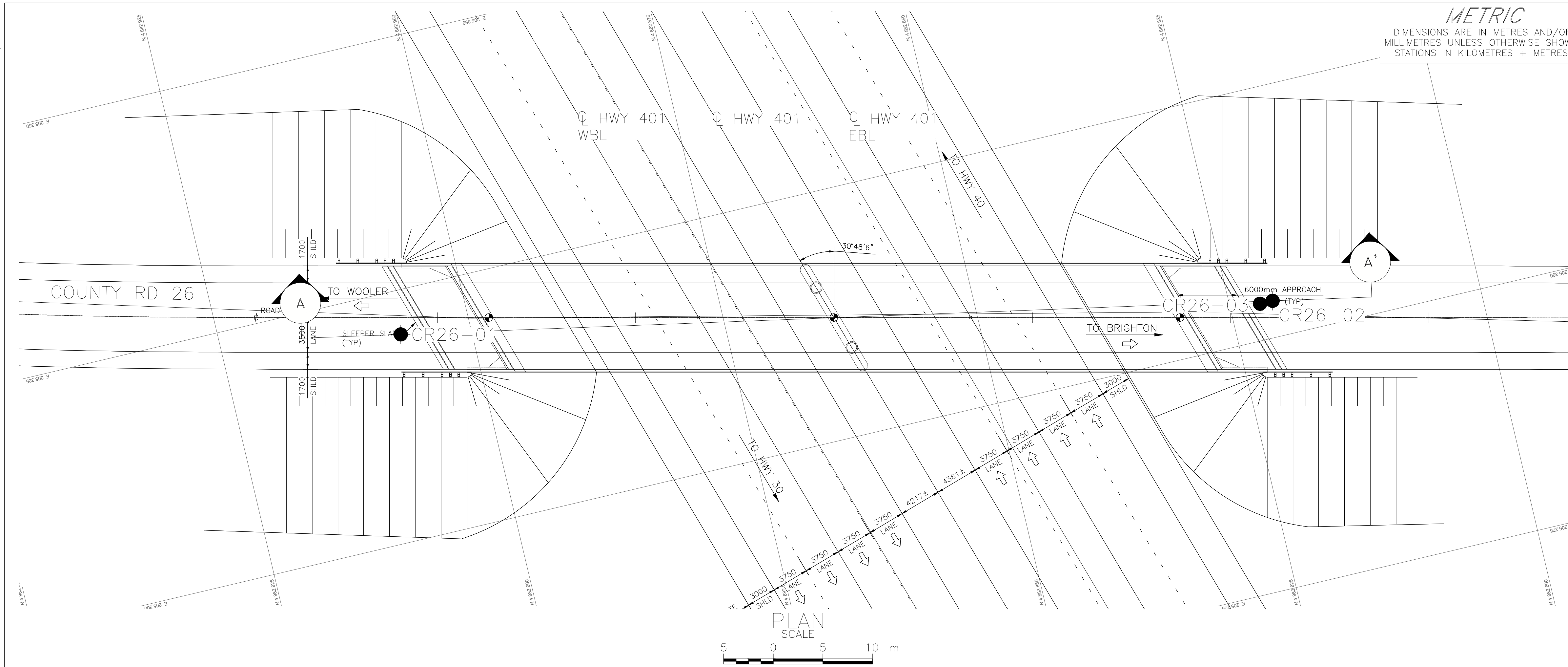
CONT No.
GWP No. 4054-17-00

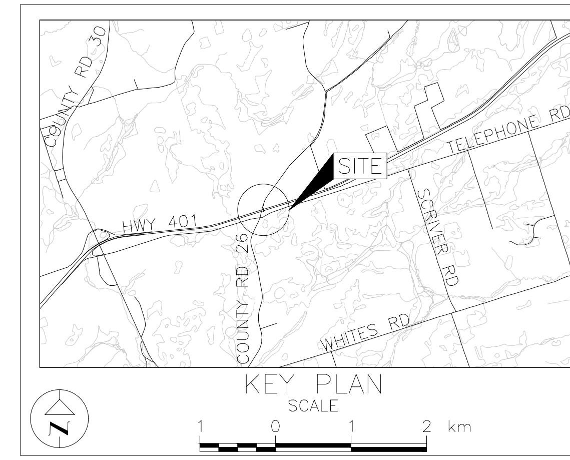
REPLCEMENT OF HIGHWAY 401 UNDERPASS AT COUNTY RD 26

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

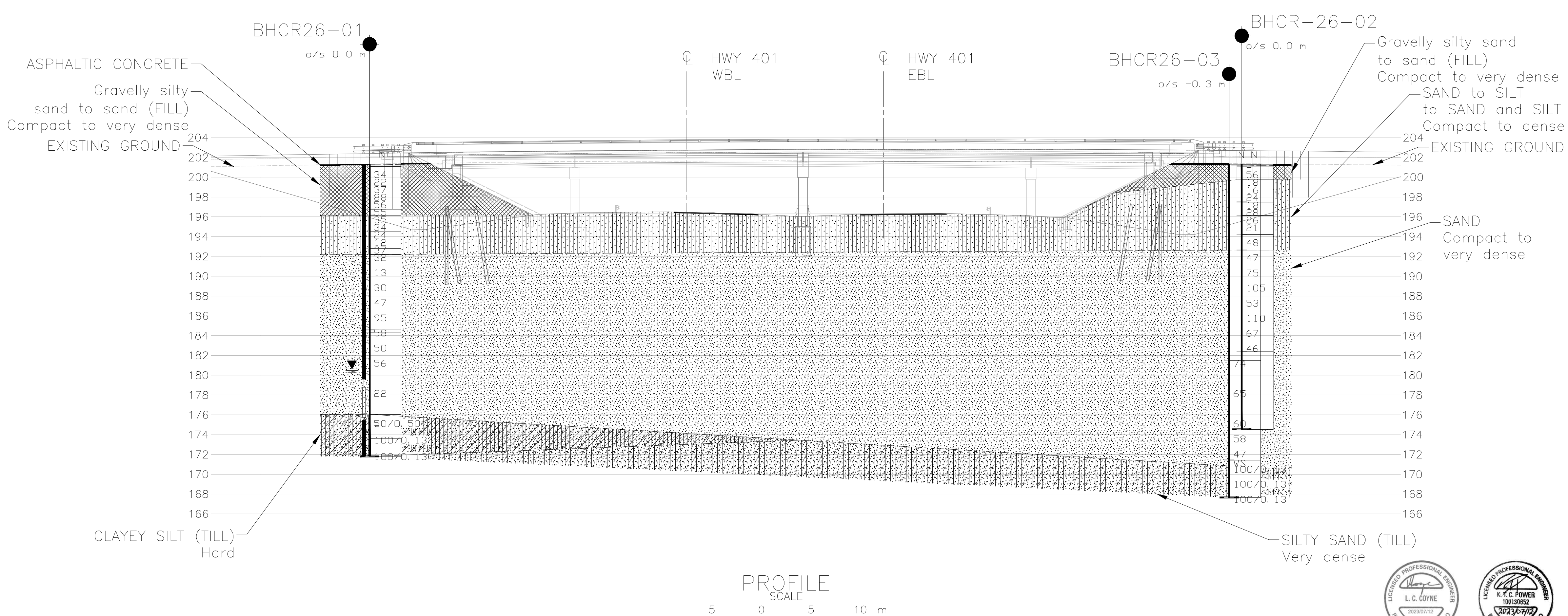
WSP GOLDER





LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer
- WL in piezometer, measured on July 21, 2022.



BOREHOLE CO-ORDINATES NAD 83 (CSRS)/MTM ZONE 9			
No.	ELEVATION	NORTHING	EASTING
BHCR26-01	201.3	4882907.0	205320.6
BHCR26-02	201.3	4882820.7	205303.7
BHCR26-03	201.3	4882822.0	205303.7

Structural Site Location Latitude: 44.07941 Longitude: -77.74208

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. S17M-0172-11-300-001GA.dwg received April 05, 2022, and General Arrangement Drawing file no. S17M-0172-11-300-001GA, received February 2023.

L. C. COYNE 2023/07/12 — LICENSED PROFESSIONAL ENGINEER, PROVINCE OF ONTARIO

K. T. C. POWER 100130652 2023/07/12 — LICENSED PROFESSIONAL ENGINEER, PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 31C-321

HWY. 401		PROJECT NO. 1773612	DIST. EASTERN
SUBM'D. KCP	CHKD. KG	DATE: 7/12/2023	SITE: 21-297
DRAWN: ZS	CHKD. KCP	APPD. LCC	DWG. 1

APPENDIX A

Borehole Records

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

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS[1,2]

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (*i.e.,* SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (*i.e.,* some sand)
≤ 10	trace (*i.e.,* trace fines)

1. Only applicable to components not described by Primary Group Name.
2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

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

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

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

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

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness[1]

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

3. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
4. 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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
U	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	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
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

RECORD OF BOREHOLE No CR26-01 SHEET 1 OF 3 METRIC

PROJECT 1773612
G.W.P. 4054-17-00 LOCATION N 4882907.0; E 205320.6 MTM NAD ZONE 9 (LAT. 44.079410; LONG. -77.742080) ORIGINATED BY JS
DIST Eastern HWY 401 BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID Hollow Stem Augers COMPILED BY TR
DATUM GEODETIC DATE July 14, 2022 CHECKED BY KCP/LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
201.3	GROUND SURFACE								
0.0	ASPHALT / CONCRETE (100 mm)	1A	AS	-					
0.3	(SW) gravelly SAND (PAVEMENT STRUCTURE) (FILL) Brown Moist	1B							
	(SM) Gravelly silty sand, contains cobbles (FILL) Compact to very dense Brown Moist	2	SS	34	o	25	55	(20)	
		3	SS	22					
		4	SS	37					
		5	SS	88					
		6	SS	56					
196.7 4.6	(SM) Gravelly silty sand, some clay (FILL) Very dense Black to light brown Moist	7A 7B	SS	55	o	22	47	(31)	
196.1 5.2	(SW) SAND, trace silt, trace gravel Dense Brown Moist	8	SS	35					
		9	SS	34	o	3	90	(7)	
194.4 6.9	(ML) SILT, some sand, some clay Compact Brown Moist	10	SS	24					
		11	SS	12	o	0	16	67	17
192.8 8.5	(SM/ML) SAND and SILT Dense Brown Moist	12A 12B	SS	37	o	0	58	(42)	
192.2 9.1	(SP) SAND, trace to some silt and gravel Very dense to compact Light brown to brown Moist to wet	13	SS	32					
		14	SS	13					

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (201, 200, 199, 198, 197, 196, 195, 194, 193, 192, 191, 190); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

Continued Next Page

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

RECORD OF BOREHOLE No CR26-01

SHEET 2 OF 3 **METRIC**

PROJECT 1773612
G.W.P. 4054-17-00
LOCATION N 4882907.0; E 205320.6 MTM NAD ZONE 9 (LAT. 44.079410; LONG. -77.742080)
ORIGINATED BY JS
DIST Eastern HWY 401
BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID Hollow Stem Augers
COMPILED BY TR
DATUM GEODETIC
DATE July 14, 2022
CHECKED BY KCP/LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(SP) SAND, trace to some silt and gravel Very dense to compact Light brown to brown Moist to wet	15	SS	30	189		
		16	SS	47	188 / 187		
		17	SS	95	186		
		18A 18B	SS	58	185 / 184		
		19	SS	50	183	o	2 86 (12)
		20	SS	56	182 / 181		
		21	SS	22	180 / 179 / 178		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE; ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m^3

Continued Next Page

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

RECORD OF BOREHOLE No CR26-01

SHEET 3 OF 3 **METRIC**

PROJECT 1773612
G.W.P. 4054-17-00
DIST Eastern HWY 401
DATUM GEODETIC
LOCATION N 4882907.0; E 205320.6 MTM NAD ZONE 9 (LAT. 44.079410; LONG. -77.742080)
BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID Hollow Stem Augers
DATE July 14, 2022
ORIGINATED BY JS
COMPILED BY TR
CHECKED BY KCP/LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	(SP) SAND, trace to some silt and gravel Very dense to compact Light brown to brown Moist to wet				177					
176.0 25.3	(GM) SILTY GRAVEL, trace to some sand, some clay, contains cobbles and boulders (TILL) Dense				176					
		22	SS	50/0.05	175					
		23	WS	-		o	63	6	18	13
		24	SS	100/0.13	174					
173.6 27.7	(SM) SILTY SAND, trace to some clay, some gravel, contains cobbles and boulders (TILL) Dense Brown Wet				173					
		25	SS	100/0.13	172					
171.8 29.5	END OF BOREHOLE NOTES: 1. Water level measured in monitoring well at 20.7 m (Elev. 180.6 m) below ground surface on July 21, 2022.									

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

WSP GOLDER

RECORD OF BOREHOLE No CR26-02 SHEET 1 OF 3 METRIC

PROJECT 1773612
G.W.P. 4054-17-00
DIST Eastern HWY 401
DATUM GEODETIC
LOCATION N 4882820.7; E 205303.7 MTM NAD ZONE 9 (LAT. 44.078630; LONG. -77.742270)
BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID Hollow Stem Augers
DATE July 11, 2022
ORIGINATED BY JS
COMPILED BY TR
CHECKED BY KCP/LCC

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
201.3	GROUND SURFACE					
0.0	ASPHALT / CONCRETE (100 mm)	1A	AS	-		
0.2	(SW-SW/GW) gravelly SAND to SAND and GRAVEL (PAVEMENT STRUCTURE) (FILL) Brown Moist	1B				
	(SW) SAND, some gravel, trace silt (FILL) Very dense Brown Moist	2	SS	56		
199.8 / 1.5	(ML) Sandy SILT, trace gravel, some clay Compact Brown Moist	3	SS	18		
		4	SS	16	~11	2 22 (76)
		5	SS	24		
197.5 / 3.8	(ML/SM) SILT and SAND, some clay Compact Brown Moist	6	SS	18	~11	0 33 55 12
		7	SS	28		
196.1 / 5.2	(SP) SAND, trace to some silt Compact Brown Moist	8	SS	26		
		9	SS	21		
194.2 / 7.1	(ML/SM) SILT and SAND, some clay Dense Brown Moist	10	SS	48	~9	0 42 (58)
192.6 / 8.7	(SP) SAND, trace silt Dense to very dense Brown Moist	11	SS	47		
		12	SS	75		

Elevation scale: 201, 200, 199, 198, 197, 196, 195, 194, 193, 192, 191, 190

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m^3

Continued Next Page

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

WSP GOLDER

RECORD OF BOREHOLE No CR26-02

METRIC

PROJECT 1773612

G.W.P. 4054-17-00

LOCATION N 4882820.7; E 205303.7 MTM NAD ZONE 9 (LAT. 44.078630; LONG. -77.742270)

ORIGINATED BY JS

DIST Eastern HWY 401

BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID Hollow Stem Augers

COMPILED BY TR

DATUM GEODETIC

DATE July 11, 2022

CHECKED BY KCP/LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---						
	(SP) SAND, trace silt Dense to very dense Brown Moist					189	
		13	SS	105			0 88 (12)
						188	
		14	SS	53		187	
						186	
		15	SS	110		185	
		16	SS	67		184	0 85 (15)
		17	SS	46	∇	183	
182.4 18.9	START OF DCPT Inferred SAND, some silt					182	
						181	
						180	
						179	
						178	

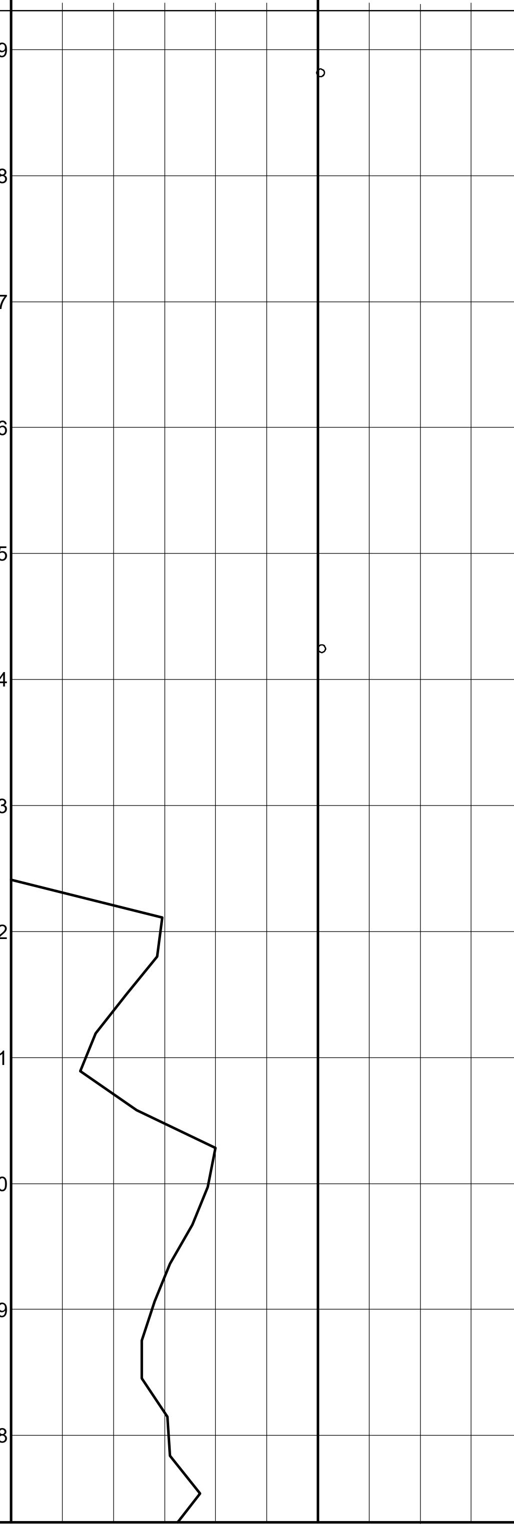
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³



Continued Next Page

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

RECORD OF BOREHOLE No CR26-02

SHEET 3 OF 3 **METRIC**

PROJECT 1773612

G.W.P. 4054-17-00

DIST Eastern HWY 401

DATUM GEODETIC

LOCATION N 4882820.7; E 205303.7 MTM NAD ZONE 9 (LAT. 44.078630; LONG. -77.742270)

BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID Hollow Stem Augers

DATE July 11, 2022

ORIGINATED BY JS

COMPILED BY TR

CHECKED BY KCP/LCC

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
							177				
							176				
							175				
174.5 26.8	END OF DCPT END OF BOREHOLE at DCPT Refusal NOTES: 1. Open Borehole dry at a depth of 18.9 m (Elev. 182.4 m) on completion of augering.										

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

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

RECORD OF BOREHOLE No CR26-03

SHEET 1 OF 3

METRIC

PROJECT 1773612

G.W.P. 4054-17-00

LOCATION N 4882822.0; E 205303.7 MTM NAD ZONE 9 (LAT. 44.078640; LONG. -77.742270)

ORIGINATED BY JS

DIST Eastern HWY 401

BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID HSA then Wash Boring

COMPILED BY TR

DATUM GEODETIC

DATE July 12 & 13, 2022

CHECKED BY KCP/LCC

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
201.3	GROUND SURFACE										
0.0	For soil stratigraphy from 0 m to 19.8 m refer to Record of Borehole BHCR26-02						201				
							200				
							199				
							198				
							197				
							196				
							195				
							194				
							193				
							192				
							191				
							190				

Continued Next Page

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

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

RECORD OF BOREHOLE No CR26-03 SHEET 2 OF 3 METRIC

PROJECT 1773612

G.W.P. 4054-17-00 LOCATION N 4882822.0; E 205303.7 MTM NAD ZONE 9 (LAT. 44.078640; LONG. -77.742270) ORIGINATED BY JS

DIST Eastern HWY 401 BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID HSA then Wash Boring COMPILED BY TR

DATUM GEODETIC DATE July 12 & 13, 2022 CHECKED BY KCP/LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	WATER CONTENT (%) (PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	For soil stratigraphy from 0 m to 19.8 m refer to Record of Borehole BHCR26-02						189				
							188				
							187				
							186				
							185				
							184				
							183				
							182				
181.5 19.8	(SP) SAND, trace to some silt Very dense Brown Moist to wet		1	SS	74		181				
							180				
						∇	179				
			2	SS	65		178		o (approx. 15)		0 83 (17)

Continued Next Page

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

RECORD OF BOREHOLE No CR26-03

SHEET 3 OF 3 **METRIC**

PROJECT 1773612

G.W.P. 4054-17-00

DIST Eastern HWY 401

DATUM GEODETIC

LOCATION N 4882822.0; E 205303.7 MTM NAD ZONE 9 (LAT. 44.078640; LONG. -77.742270)

BOREHOLE TYPE CME 55 Truck Mounted, 108 mm ID HSA then Wash Boring

DATE July 12 & 13, 2022

ORIGINATED BY JS

COMPILED BY TR

CHECKED BY KCP/LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	Natural Moisture Content (%)	GR	SA	SI	CL
	(SP) SAND, trace to some silt Very dense Brown Moist to wet				177					
					176					
		3	SS	60	175					
					174					
		4	SS	58	173					
		5	SS	47	172					
171.4 29.9	(SM) Gravelly SILTY SAND, some clay, contains cobbles and boulders (TILL) Very dense Brown to grey-brown Moist to wet	6A		WS	171					
		6B		100/0.13	170					
		7	SS	100/0.13	169					
					168					
167.6		8	SS	100/0.13		o	20	35	28	17
33.7	END OF BOREHOLE									

NOTES:

1. Wet soils encountered at a depth of approximately 22.2 m (Elev. 179.1 m).

2. Switched from hollow stem auger to wash boring using HQ Casing at 22.9 m depth.

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

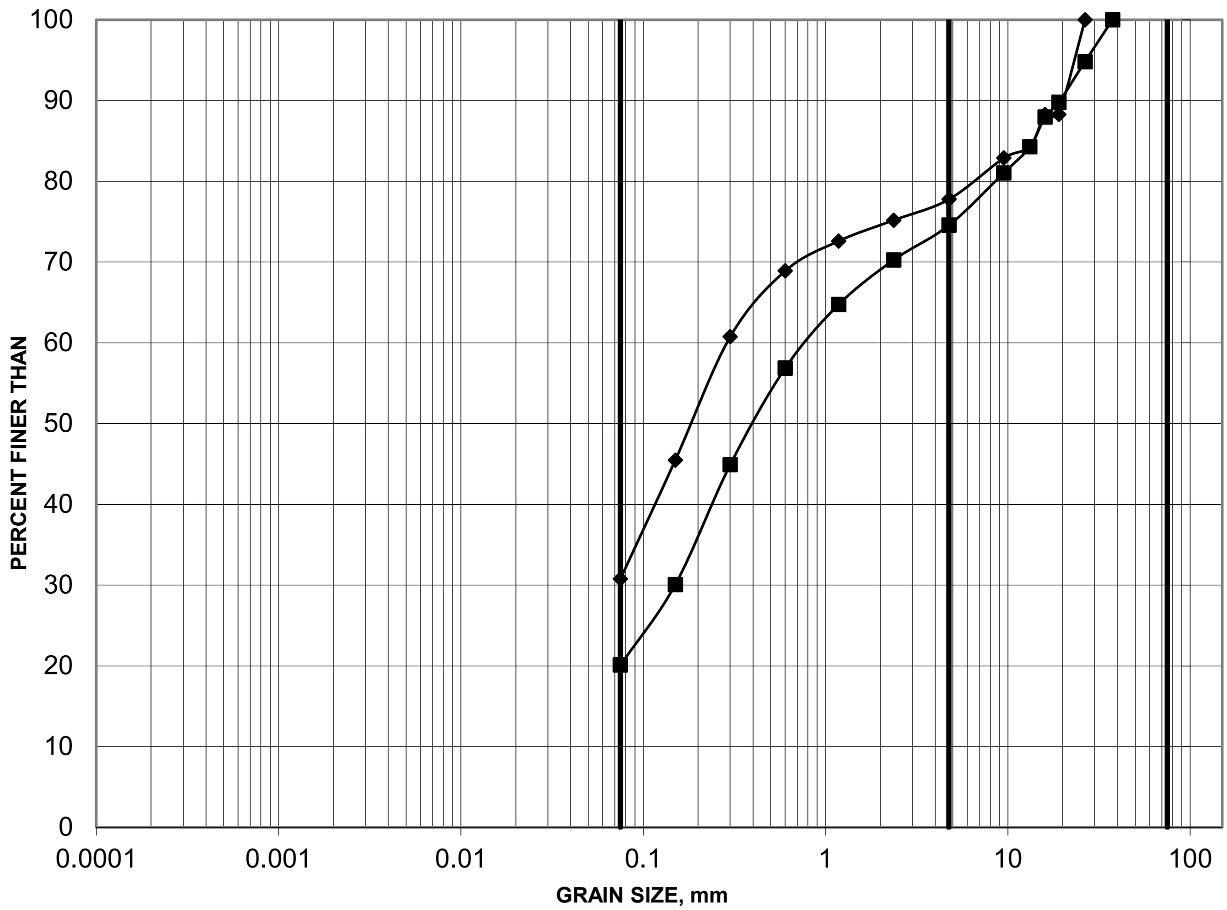
+³, ×³: Numbers refer to Sensitivity   O ³% STRAIN AT FAILURE

APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

GRAVELLY SILTY SAND FILL



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
CR26-01	2	0.76-1.37	25	55	20	
CR26-01	7A	4.57-4.88	22	47	31	

GOLDER
MEMBER OF WSP

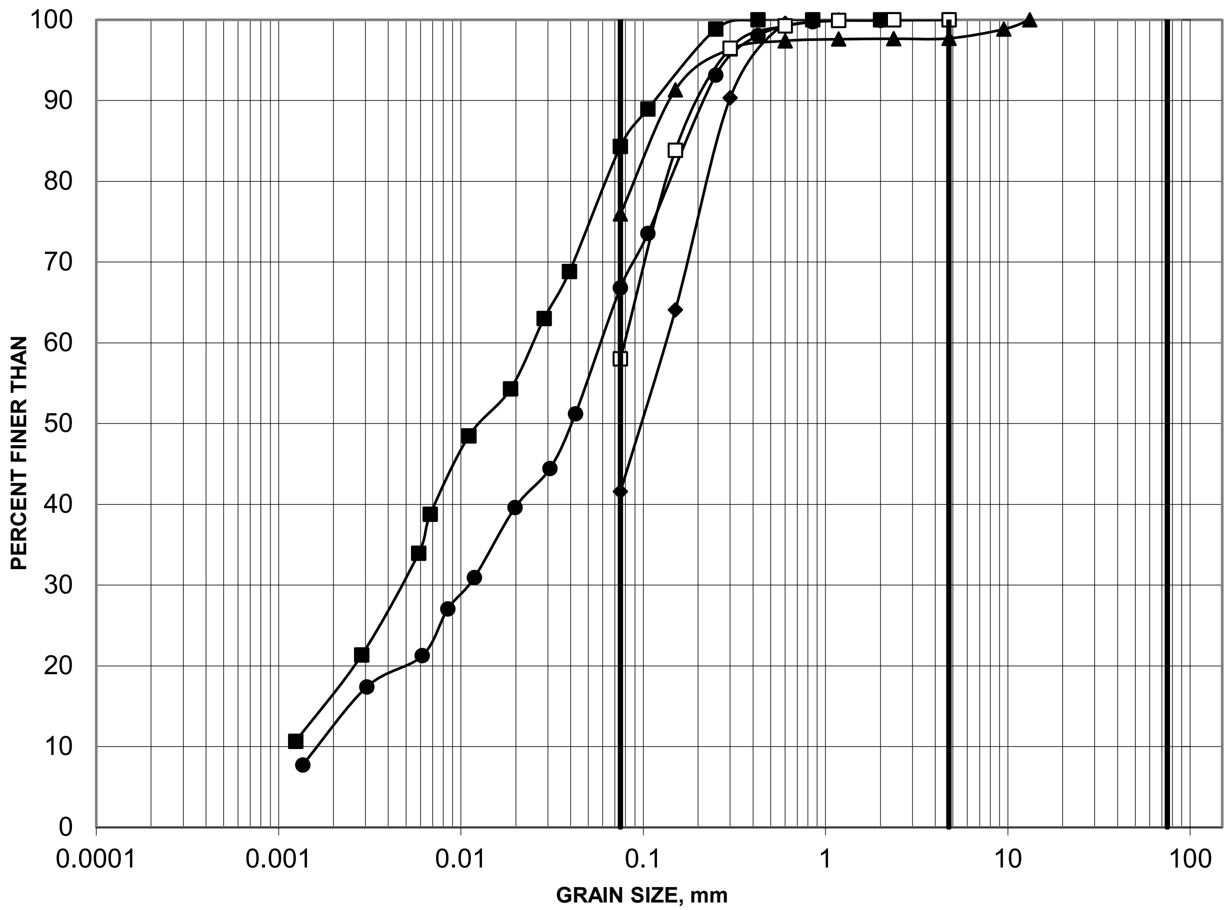
Project: 1773612_WO 11

Created by: KG
Checked by:

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab Testing/1-CR26/Figures/

GRAIN SIZE DISTRIBUTION	FIGURE B2

SILT TO SAND AND SILT



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

Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
CR26-01	11	7.62-8.23	0	16	84	
CR26-01	12B	8.54-8.99	0	58	42	
CR26-02	4	2.29-2.90	2	22	76	
CR26-02	6	3.81-4.42	0	33	55	12
CR26-02	10	7.62-8.23	0	42	58	

Project: 1773612_WO 11

GOLDER
MEMBER OF WSP

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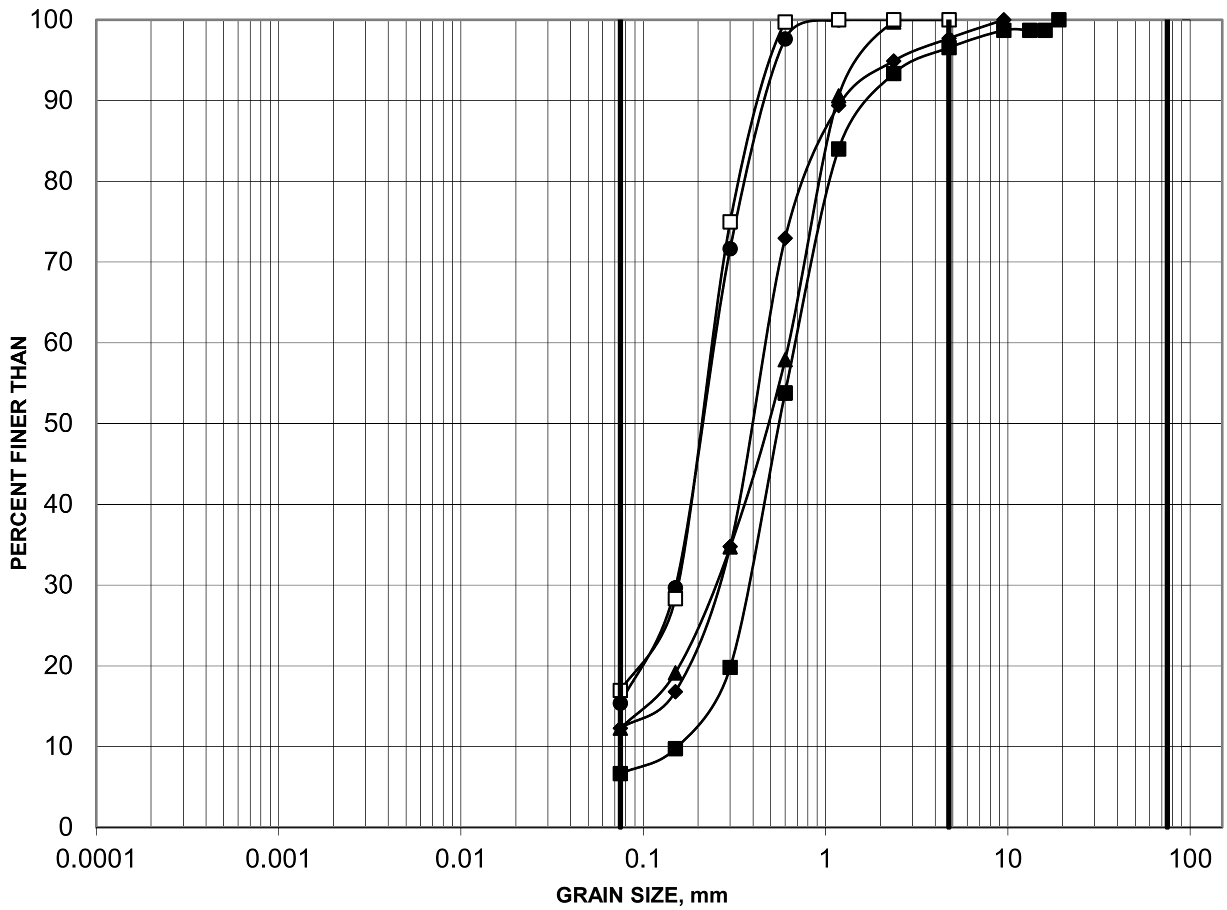
GRAIN SIZE DISTRIBUTION	FIGURE B3

SAND



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

Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
CR26-01	9	6.10-6.71	3	90	7	
CR26-01	19	18.29-18.90	2	86	12	
CR26-02	13	12.19-12.80	0	88	12	
CR26-02	16	16.76-17.37	0	85	15	
CR26-03	2	22.87-23.48	0	83	17	

Project: 1773612_WO 11

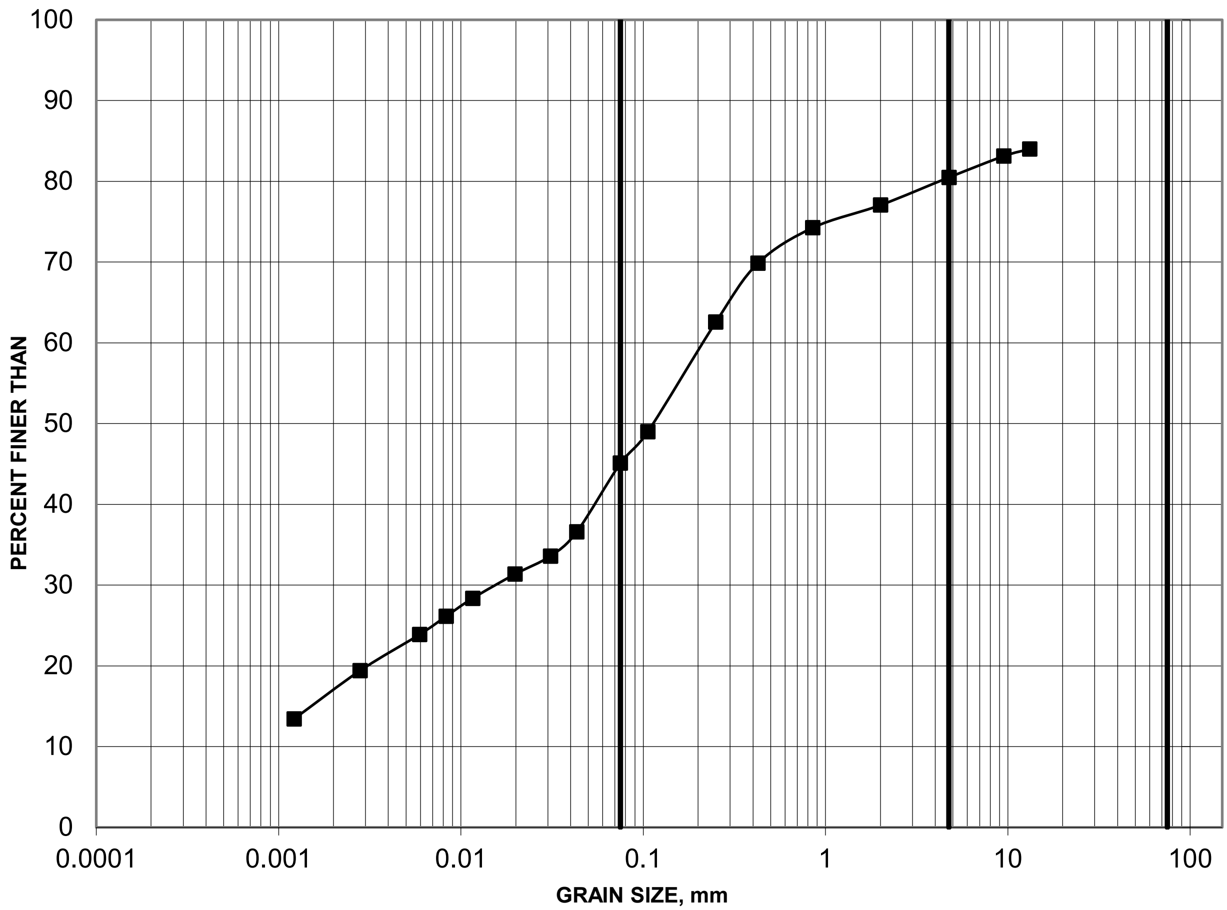
GOLDER
MEMBER OF WSP

Created by: KG
Checked by:

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab Testing/1-CR26/Figures/

GRAIN SIZE DISTRIBUTION	FIGURE B4

GRAVELLY SILTY SAND TILL



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

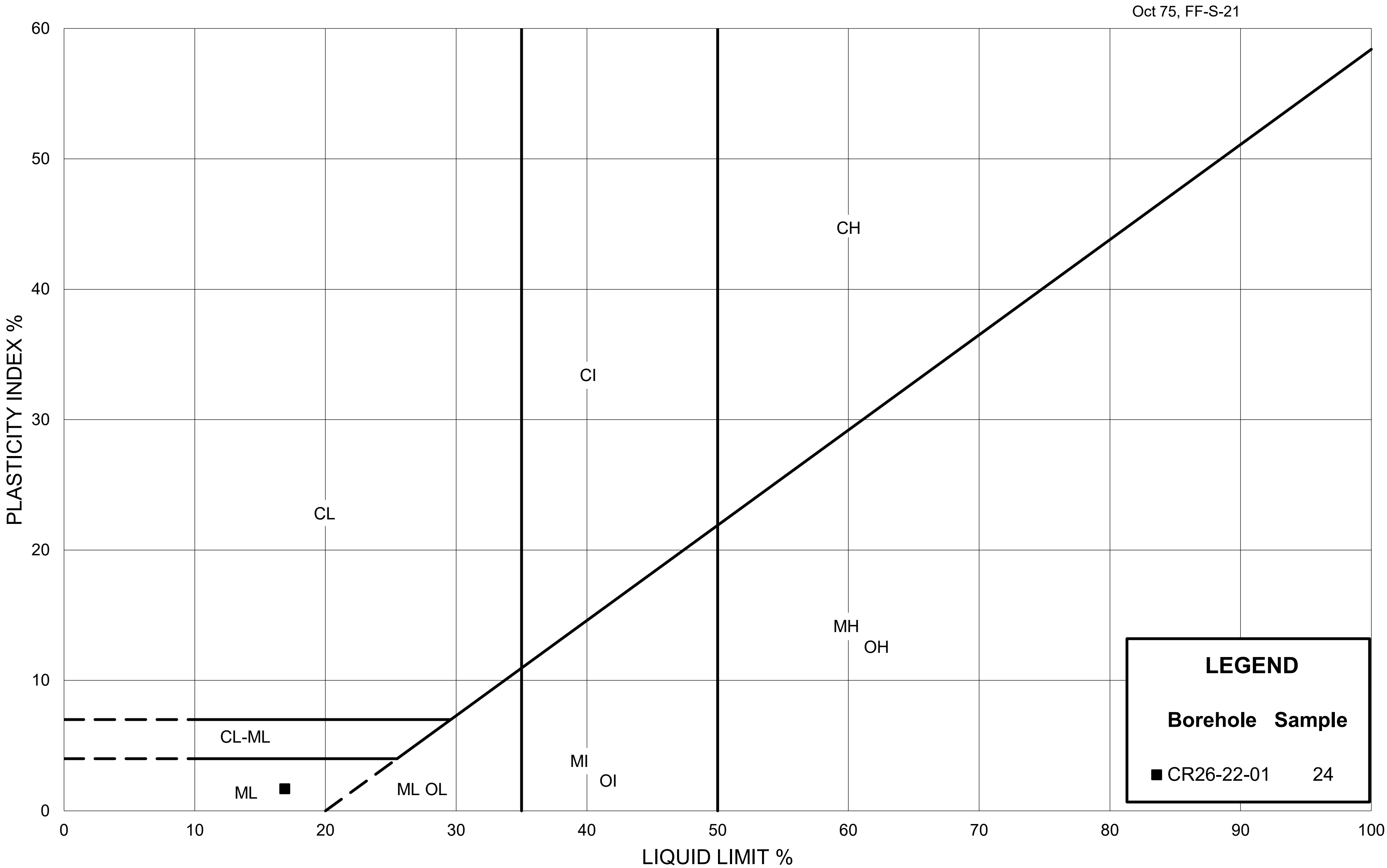
	Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
—■—	CR26-03	8	33.53-33.66	20	35	28	17

Project: 1773612_WO 11

GOLDER
MEMBER OF WSP

Created by: KG
Checked by:

https://golderassociates.sharepoint.com/sites/11407g/WO11 Colborne to Brighten/2. Technical Work/5. Lab Testing/1-CR26/Figures/



Ministry of Transportation Ontario	PLASTICITY CHART CLAYEY SILT TILL	Figure: B5 Project: 1773612_WO11 Created By: MI Checked By:

APPENDIX C

Analytical Laboratory Test Results

Certificate of Analysis

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

Dear Kenton Power:

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

Report Comments:

APPROVAL: ______________________________

Emma-Dawn Ferguson, Chemist

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

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

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

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

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

Certificate of Analysis

Environment Testing

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

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

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

Guideline = *** = Guideline Exceedence**

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

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

Certificate of Analysis

eurofins | Environment Testing

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

QC Summary

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

Guideline = *** = Guideline Exceedence**

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

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

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