



Foundation Investigation and Design Report

*Stormwater Management Ponds A and B
Bridge Replacement and Interchange Reconfigurations at
Highway 11/12 (Old Barrie Road), Orillia
Ministry of Transportation, Ontario
GWP 2129-18-00*

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PART A

**FOUNDATION INVESTIGATION REPORT
STORMWATER MANAGEMENT PONDS A AND B
BRIDGE REPLACEMENT AND INTERCHANGE RECONFIGURATION AT
HIGHWAY 11/12 (OLD BARRIE ROAD), ORILLIA
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 2129-18-00**

1.0 INTRODUCTION

WSP Canada Inc. (WSP, formerly Golder Associates Ltd., amalgamated with WSP in 2023), has been retained by Egis Group (Egis, formerly McIntosh Perry Consulting Engineers Ltd. (McIntosh Perry)) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary design foundation engineering services as part of the design-build ready assignment for the interchange improvements at the Highway 11 and Highway 12 (Old Barrie Road) south junction. This report presents the results of the foundation investigation for the proposed stormwater management ponds (designated as Pond A and Pond B) at the Highway 11/12 (Old Barrie Road) interchange.

The purpose of this foundation investigation is to establish the subsurface conditions at the stormwater management pond locations by methods of borehole drilling, in-situ testing, and laboratory testing on selected soil samples.

This report summarizes the factual results of the current (2021-2022) foundation investigation and is supplemented with a selected borehole from a previous investigation (GEOCREC No. 31D-647) carried out in 2015 in the vicinity of the proposed stormwater management Pond B. Based on the information from the current and previous investigations, this report provides a description of the interpreted soil and groundwater conditions at the proposed stormwater management ponds.

2.0 PROJECT AND SITE DESCRIPTION

The orientation stated (i.e., north, south, east, and west) in the text of this report is referenced to project north and therefore may differ from magnetic north shown on Drawing 1. For this report, Highway 11 (in the vicinity of the City of Orillia) is considered oriented in a south-north direction and Highway 12 (Old Barrie Road) is considered oriented in a west-east direction.

2.1 Project Description

The overall assignment includes the preparation of two separate contracts. The first contract includes the replacement of the Coldwater Road Underpass and the reconfiguration/reconstruction of the Highway 11 and Highway 12 (Coldwater Road) interchange. The second contract includes the replacement of the Old Barrie Road Underpass; the reconfiguration/reconstruction of the Highway 11 and Highway 12 (Old Barrie Road) interchange, including the construction of deep cuts and high fill embankments; construction of two retaining walls (designated as Retaining Wall No. 1 and 2); construction of two noise barrier walls (designated as Noise Barrier Walls No. 1 and 2); and construction of two stormwater management ponds, designated as Pond A and Pond B.

Pond A and Pond B are located at the southeast quadrant of the Highway 11 and Highway 12 (Old Barrie Road) interchange, as shown on Drawing 1. Pond A will be a wet pond, located within the centre of the proposed W-N Ramp and Pond B will be a dry swale, located between Highway 12 (Old Barrie Road) and the proposed S-E/W Ramp.

2.2 Site Description

The Pond A footprint is generally located within the low-lying, vegetated area at the southeast quadrant of the Highway 11 and Highway 12 (Old Barrie Road) interchange, with a portion of the pond footprint extending onto the current S-E/W Ramp. The existing ground surface within the Pond A footprint ranges from about Elevation 250 m to 254 m.

The Pond B footprint is generally located within the low-lying, vegetated area south of Highway 12 (Old Barrie Road) and north of the existing S-E/W Ramp, with a portion of the pond footprint extending onto the current S-E/W Ramp.

The existing ground surface within the Pond B footprint ranges from about Elevation 244 m to 252 m. An existing drainage swale is located in the area of Pond B, flowing west to east.

3.0 INVESTIGATION

3.1 Previous (2015) Investigation

A previous foundation investigation was carried out at the site in 2015, which included the advancement of one borehole (designated as Borehole BH15-06) in the vicinity of Pond B, at the location shown on Drawing 1. The borehole location, ground surface elevation, and depth are summarized in the table below. The borehole record from Borehole BH15-06 is provided in Appendix A. Additional information from the previous foundation investigation can be found in GEOCREs No. 31D-647.

Pond ID	Borehole No.	Coordinates (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
Pond B	BH15-06	4938950.1 (44.590552)	309452.2 (-79.441410)	251.1	16.9

3.2 Current (2022 – 2023) Investigation

The fieldwork for the current subsurface investigation was carried out between April 28, 2022 and November 27, 2023, during which time two boreholes (designated as Boreholes CO5-4 and CO5-5) were advanced in the vicinity of Pond A and two boreholes (designated as Borehole HF-5 and HF-6) were advanced in the vicinity of Pond B. The locations of the boreholes are shown in plan on Drawing 1. These are considered dual purpose boreholes.

The boreholes were advanced using a D-50 / D-55 track-mounted drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. Traffic control was performed in accordance with the Ontario Traffic Manual Book 7 – Temporary Conditions. Borehole CO5-4 and CO5-5 were advanced using 110 mm inner diameter (ID) and 210 mm outer diameter (OD) continuous flight hollow stem augers and Boreholes HF-5 and HF-6 were advanced using 115 mm OD solid stem augers. Soil samples were typically obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in general accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). The split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions.

Water levels were observed in the open boreholes during and immediately following the drilling operations. A standpipe piezometer was installed in one borehole (Borehole CO5-4) to permit monitoring of the groundwater level. The standpipe piezometer consists of a 50 mm outer diameter Schedule 40 PVC pipe, with a slotted screen surrounded with a sand filter pack, sealed at a selected depth within the borehole. The annulus surrounding the pipe above the well screen and sand filter pack was backfilled to the ground surface with bentonite.

The boreholes, excluding Borehole CO5-4, were backfilled with bentonite upon completion of drilling operations in accordance with Ontario Regulation 903 (*Wells*), as amended. Borehole CO5-4 is to be decommissioned by the Design-Build Contractor at the time of construction.

Prior to commencement of the field work, WSP arranged for the clearance of underground utilities. The field work was supervised by a member of WSP's engineering staff, who observed the borehole drilling, in-situ testing, and

soil sampling operations, and logged the boreholes in the field. The soil samples were placed in appropriate containers, labelled, and transported to WSP's Mississauga geotechnical laboratory where the samples underwent further visual and tactile examination and geotechnical laboratory testing.

Geotechnical index testing, such as water content, Atterberg limits, and grain size distribution, was carried out on selected soil samples in accordance with MTO and/or ASTM Standards, as appropriate, and the results of which are presented in Appendix B.

The as-drilled borehole locations and the corresponding ground surface elevations were surveyed by WSP using a Trimble GPS unit. The borehole survey information, including northing/easting coordinates (reference to the NAD83 Canadian Spatial Reference System (CSRS) V6:2010 MTM Zone 10 coordinate system), latitude/longitude coordinates, and corresponding ground surface elevations (referenced to the Canadian Geodetic Vertical Datum (CGVD) 1928:1978), as well as borehole depths are provided on the borehole records in Appendix A and summarized below.

Pond ID	Borehole No.	Coordinates (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
Pond A	CO5-4	4938767.3 (44.588900)	309109.8 (-79.445721)	253.0	9.5
	CO5-5	4938765.4 (44.588883)	309160.8 (-79.445079)	249.0	11.3
Pond B	HF-5	4,938,867.1 (44.589806)	309,341.7 (-79.442803)	241.9	11.3
	HF-6	4,938,892.1 (44.590031)	309,394.1 (-79.442142)	242.0	11.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 12 lies within the Simcoe Lowlands, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). The Simcoe Lowlands consist of a series of steep sided, flat-floored valleys that were flooded by glacial lake Algonquin. The surficial soils in this area of the Simcoe Lowlands typically comprise glaciolacustrine sediments of very fine to medium-grained sand, silt and minor clay; and fluvial and glaciofluvial ice-contact sediments of fine to very coarse-grained sand, gravelly sand and gravel with minor amounts of silt, clay and flowtill. Modern alluvial deposits of clay, silt, sand gravel that may contain organics are also present.

4.2 Subsurface Conditions

The subsurface soil and groundwater conditions encountered in the boreholes advanced in the vicinity of the proposed stormwater management ponds are presented on the borehole records in Appendix A. To assist in the interpretation of the borehole records, *Method of Soil Classification, Abbreviations and Terms Used on Records of Boreholes and Test Pits* and *List of Symbols* sheets are provided in Appendix A. The geotechnical laboratory results are included in Appendix B. The results of in-situ tests as presented on the borehole records are uncorrected for overburden pressure and energy transfers. The 'N'-values are based on SPT sampling procedures carried out with a standard weight (i.e., 63.5 kg), and an automatic hammer.

The stratigraphic boundaries shown on the borehole records and on the profile shown on Drawing 1 have been inferred from observations of drilling progress, generally non-continuous sampling and in-situ testing, and therefore represent transitions between soil types rather than exact planes of geologic change. Further, subsurface conditions will vary between and beyond the borehole locations.

The subsurface soils encountered in the vicinity of the Pond A generally consist of topsoil underlain by fill, which in turn is underlain by a dense to very dense gravelly silty sand glacial till deposit at the western portion of the pond location and a very dense silt to silty sand deposit at the eastern portion of the pond location.

The subsurface soils encountered in the vicinity of the Pond B generally consist of topsoil underlain by fill, which in turn is underlain by gravelly silty sand to sandy gravel deposit, subsequently underlain by a silty sand deposit or glacial till deposit. A gravelly clayey sand deposit was encountered between the fill and the gravelly silty sand to sandy gravel deposit at the eastern portion of the pond location.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

An approximately 100 mm to 305 mm thick layer of topsoil was encountered at ground surface elevations in Boreholes CO5-4, CO5-5, HF-5, and HF-6.

4.2.2 Asphalt

An approximately 270 mm thick layer of asphalt was encountered at ground surface elevations in Borehole BH15-06.

4.2.3 Fill

An approximately 0.6 m to 6.9 m thick layer of fill was encountered underlying the topsoil / asphalt in all boreholes. The fill extends to depths of 0.7 m to 7.1 m below ground surface (Elevations 240.9 m and 250.8 m). The fill ranges in composition from clayey silt-silt to silty sand to gravelly sand. The fill contains organics in Boreholes CO5-4 and CO5-5, BH15-06, contains rock fragments in Boreholes CO5-4 and CO5-5, and observed to contain cobbles in Boreholes CO5-5 and BH15-06. Auger grinding was encountered while advancing the borehole through the fill in Borehole CO5-4, as noted on the borehole record. Based on the observed cobbles, rock fragments, and auger grinding, the presence of cobbles and boulders are anticipated within the fill.

SPT 'N'-values measured within the fill range from 1 to 101 blows per 0.3 m of penetration indicating a very loose to very dense state of compactness / stiff consistency.

As part of the previous investigation, grain size distribution testing was carried out on one sample of the fill and the results are presented on the borehole record for Borehole BH15-06 in Appendix A. As part of the current investigation, grain size distribution testing was carried out on three samples of the fill and the results are presented on Figure B-1 in Appendix B. Atterberg Limits testing was carried out on two samples of the fill and measured liquid limits of about 22% and 25%, plastic limits of about 18% and 19%, and plasticity indices of about 4% and 6%, indicating the fill ranges from a silty sand to clayey silt-silt of low plasticity. The results of the Atterberg Limit tests are presented on Figure B-2 in Appendix B.

The water contents measured on select samples of the fill range from about 4% to 22%.

4.2.4 Gravelly Clayey Sand

An approximately 3.0 m thick deposit of gravelly clayey sand was encountered below the fill in Borehole BH15-06, at a depth 7.1 m below ground surface (Elevation 244.0 m) and extended to a depth of 10.1 m below ground surface (Elevation 241.0 m). This deposit is classified as sand and gravel till on the borehole record, however, based on the current MTO soil classification system, this material is classified as gravelly clayey sand. Further, given that this material contains organics, it is inferred that this deposit is not till.

SPT-'N' values measured within the gravelly clayey sand deposit are 11 blows and 16 blows for 0.3 m of penetration, suggesting a stiff to very stiff consistency.

As part of the previous investigation, grain size distribution testing and Atterberg limit testing was carried out on one sample of the gravelly clayey sand deposit and the results are presented on the borehole record for Borehole BH15-06 in Appendix A.

Water contents measured on select samples of the gravelly clayey sand deposit were about 8% and 9%.

4.2.5 Gravelly Silty Sand to Sandy Gravel

An approximately 1.7 m to 2.7 m thick deposit of gravelly silty sand to sandy gravel was encountered below the fill in Boreholes HF-5 and HF-6, and below the gravelly clayey sand deposit in Borehole BH15-06, at depths ranging from 0.7 m to 10.1 m below ground surface (Elevations 240.9 m to 241.2 m). The gravelly silty sand to sandy gravel deposit extended to depths ranging from about 3.4 m to 11.8 m below ground surface (Elevations 238.3 m to 239.3 m). Cobbles and boulders are inferred within the deposit, based on observed auger grinding in Borehole BH15-06, as noted on the borehole record.

At Borehole HF-6, a lower sand and gravel to gravel and sand deposit was encountered below the silt to sand deposit, at a depth of 5.6 m below ground surface (Elevation 236.4 m) and extended to the borehole termination depth of 11.3 m below ground surface (Elevation 230.7 m).

SPT-'N' values measured within the gravelly silty sand to sandy gravel deposit generally range from 10 blows to greater than 100 blows for 0.3 m of penetration, indicating a compact to very dense compactness condition.

As part of the current investigation, grain size distribution testing was carried out on five samples of the gravelly silty sand to sandy gravel deposit and the results are presented on Figure B-3 in Appendix B. As part of the current investigation, Atterberg Limits testing was carried out on three samples of the gravelly silty sand to gravel and sand deposit. One sample yielded non-plastic results and the other two samples measured liquid limits of about 13% and 14%, plastic limits of about 12%, and plasticity indices of about 1% and 2% suggesting the deposit ranges from non-plastic to slightly plastic. The results of the Atterberg Limit test are presented on Figure B-4 in Appendix B.

Water contents measured on samples of the gravelly silty sand to sandy gravel range from about 5% to 13%.

4.2.6 Glacial Till

A glacial till deposit was encountered below the fill in Borehole CO5-4 and below the gravelly silty sand to sandy gravel deposit in Borehole BH15-06. The till deposit was encountered at depths of 2.2 m and 11.8 m below ground surface (Elevations 239.3 m and 250.8 m). At Borehole CO5-4, the till deposit extends to a depth of 8.4 m below ground surface (Elevation 244.6 m). At Borehole BH15-06, the till deposit extends to the borehole termination depth of 16.9 m below ground surface (Elevation 234.2 m). The glacial till ranges from silt, some gravel, some sand to gravelly silty sand. At Borehole CO5-4, the till contains rock fragments and auger grinding was observed while advancing the borehole through the till. Based on our experience with glacial till soils in the area, and the observed auger grinding and rock fragments, the presence of cobbles and boulders are anticipated within the till.

SPT-'N' values measured within the till range from 35 blows for 0.3 m of penetration to greater than 100 blows for less than 0.1 m of penetration, indicating a dense to very dense state of compactness.

As part of the previous investigation, grain size distribution testing was carried out on one sample of the till deposit and the results are presented on the borehole record for Borehole BH15-06 in Appendix A. As part of the current investigation, grain size distribution testing was carried out on two samples of the glacial till deposit and the results are presented on Figure B-5 in Appendix B. Atterberg Limits testing was carried out on two sample of the glacial

till deposit and the results are presented on Figure B-6 in Appendix B. One sample yielded non-plastic results and the other sample measured a liquid limit of 19%, a plastic limit of 15%, and a plasticity index of 4%, suggesting the till ranges from a non-plastic gravelly silty sand to a silt of low plasticity.

Water contents measured on select samples of the till deposit range from about 5% to 11%.

4.2.7 Silt to Sand

A silt to sand deposit was encountered underling the glacial till in Borehole CO5-4, underlying the fill in Borehole CO5-5, and underlying the gravelly silty sand to sandy gravel deposit in Borehole HF-5 and HF-6. The silt to sand deposit was encountered at depths ranging from 2.2 m to 8.4 m below ground surface (Elevations 238.3 m to 246.8 m). The silt to sand deposit extends to the borehole termination depths ranging from 9.5 m to 11.3 m (Elevations 230.6 m to 243.5 m). The deposit ranges in composition from silt to sand, trace to some gravel, trace to some clay.

SPT 'N'-values measured within silt to sand deposit generally range from 34 blows to over 100 blows per 0.3 m of penetration, indicating a generally dense to very dense state of compactness. SPT 'N' values of 29 blows and 88 blows per 0.3 m of penetration were measured in Borehole CO5-5 below Elevation 240 m, however, these values are likely not representative of in-situ conditions and have likely been impacted by sand heave within the borehole.

As part of the current investigation, grain size distribution testing was carried out on eight samples of the silt to sand deposit and the results are presented on Figure B-7 in Appendix B. Atterberg Limits testing was carried out on four samples of the silt to silty sand deposit. Three samples yielded non-plastic results and the other sample measured a liquid limit of about 21%, a plastic limit of 19%, and a plasticity index of about 2%, suggesting the deposit ranges from non-plastic to slightly plastic. The results of the Atterberg Limit tests are presented on Figure B-8 in Appendix B.

The water contents measured on select samples of the silt to silty sand deposit range from about 11% to 19%.

4.2.8 Groundwater Conditions

In general, the soil samples recovered from the boreholes were moist to wet. The groundwater levels were measured in the open boreholes upon completion of drilling operations. A standpipe piezometer was installed in Borehole CO5-4 to monitor the groundwater level. The groundwater level measurements and standpipe piezometer installation details and are presented below and on the borehole records.

Pond ID	Borehole No.	Water Level		Reading Type	Date
		Depth (m)	Elevation (m)		
Pond A	CO5-4	8.4	244.6	Open borehole	November 26, 2023
		7.8*	245.2*	Piezometer*	November 27, 2023*
		6.2	246.8	Piezometer	January 19, 2024
	CO5-5	7.1	241.9	Open borehole	November 24, 2023
Pond B	HF-5	0.9	241.0	Open borehole	April 29, 2022
	HF-6	0.9	241.1	Open borehole	April 29, 2022
	BH15-06	10.8	240.3	Open borehole	September 17, 2015

*Reading obtained immediately following installation of standpipe piezometer.

The groundwater level observations/measurements are subject to seasonal fluctuations and precipitation events; therefore, the groundwater level should be expected to be higher during wet periods or during any period of heavy and/or sustained precipitation.

5.0 CLOSURE

The Foundation Investigation Report was prepared by Ms. Anastasia Poliacik, P.Eng., a Senior Foundation Engineer at WSP. Mr. David Staseff, P.Eng., a Senior Principal and MTO Principal Foundations Contact with WSP conducted an independent technical and quality review of this report.

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PART B

**FOUNDATION DESIGN REPORT
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report (Part B) provides foundation engineering design recommendations for the two proposed stormwater management ponds (designated as Ponds A and B) associated with the reconstruction / reconfiguration of the Highway 11/12 (Old Barrie Road) interchange. The discussion and recommendations are based on interpretation and analysis of the factual data obtained from the boreholes and other subsurface explorations advanced during the foundation investigation(s) at this site as described in the Foundation Investigation Report (Part A of this report).

This section of the report (Part B) is intended for the use of the MTO and their procurement-ready designer for this assignment and shall not be relied upon for any other purpose or by any other parties. The discussion, recommendations and geotechnical/foundation aspects of any preliminary design or reference concept design are provided for information purposes only. Where any comments are made on construction, they are provided only to highlight those aspects which could affect the detailed design of the project. The design-build proponent(s) shall make their own interpretations based on the factual data presented in the Foundation Investigation Report (Part A of this report) and supplement with additional information as necessary, to generate and assess foundation alternatives and develop the design of the preferred alternative. The design-builder proponent is responsible for all aspects of the detailed design and construction for the preferred alternative.

6.1 General

Based on information provided by Egis in January 2024, the proposed stormwater management pond details are as summarized in the table below. Pond A is to be a wet pond and Pond B is to be a dry pond such that stormwater storage will be temporary with no permanent pool water level at Pond B.

Pond ID	Location	Approximate Existing Ground Surface Elevation (m)	Pond Base Elevation (m)	Approximate Maximum Fill / Cut (m)	Permanent Pool Elevation (m)
Pond A	Adjacent W-N Ramp	250 to 254	254.7	0.7 - 4.7 m (Fill)	255.7
Pond B	Between Hwy 12 (Old Barrie Road) and S-E/W Ramp	245 to 251	243.6 (East end) to 244.3 (West end)	1.4 – 7.4 m (Cut)	Dry

Based on the information provided by MP, it is anticipated that Pond A will primarily be founded within fill, however some minor cutting into the existing S-E/W ramp embankment will be required for construction of the pond. A proposed maintenance road extends around Pond A at Elevation 257.3 m. The Pond A side slopes are inclined at 5 Horizontal to 1 Vertical (5H:1V) from the pond base to the maintenance road and are inclined at 3H:1V above the maintenance road.

Based on the information provided by MP, Pond B will require a grade lowering of up to about 7 m depth and will include cutting into the existing S-E/W ramp embankment. The Pond B side slopes are inclined at 3H:1V.

6.1.1 Groundwater Levels

The highest groundwater levels measured in the standpipe piezometers / open boreholes located in the vicinity of Ponds A and B are at about Elevation 246.8 m and 241.1 m, as measured in January 2024 and April 2022 respectively. These groundwater elevations have been used in developing the design recommendations and construction considerations for the proposed ponds. Based on the information above, the existing groundwater level is about 7.9 m below the base of Pond A and about 2.5 m to 3.2 m below the base of Pond B.

6.2 Pond Design

6.2.1 Subgrade Conditions

All topsoil, organic material, existing fill containing excessive organics or deleterious material, and any softened/loosened soils must be completely removed from below the footprint of the pond and pond berms. Consideration can be given to leaving the existing fill in place below the pond base and pond berms, provided the fill is competent and does not contain excessive organics or deleterious materials, as determined by the geotechnical engineer / qualified person. However, if settlement sensitive structures are proposed within the pond / pond berm footprint, then it is recommended that all existing fill be removed from below the footprint of the settlement sensitive structures.

Based on the encountered subsurface conditions in the boreholes advanced in the vicinity of the ponds, excavations for removal of topsoil, organic material, and/or softened/loosened soils could extend up to the elevations and depths summarized below.

Pond	Anticipated Excavation Depth (m)	Anticipated Excavation Elevation (m)	Anticipated Subgrade Soils after Excavation	Excavation Depth Relative to Groundwater Level (m)
Pond A	2.2	246.8 – 250.8	Dense to very dense silty sand till / very dense silt	1.6 m above groundwater level
Pond B	10.1	241.0	Dense to very dense sandy gravel	0.1 m below groundwater level

After excavation to the depths / elevations noted above, the exposed soils within the footprint of the pond and pond berms should be inspected by a geotechnical engineer / qualified person (and proof-rolled by the contractor, as necessary) to identify pockets of any unsuitable soils, softened / loosened areas and/or poorly performing areas to be further subexcavated and replaced with suitable engineered fill. The excavated / sub-excavated soils shall then be replaced with engineered fill in accordance with Section 6.3.3.

6.2.2 Hydraulic Conductivity and Seepage Rate

The hydraulic conductivity of the site soils has been estimated using the Hazen method (Hazen, 1911). The Hazen method provides an estimate of hydraulic conductivity based on the grain size analyses results and the following equation:

$$K = C_H * d_{10}^2$$

Where:

K = Hydraulic conductivity (cm/s)

C_H = Empirical coefficient = 1 for sandy soils

d_{10}^2 = Diameter of the 10-percentile grain size of the material (mm²)

It is noted that the Hazen method is most applicable to well sorted fine-grained sand soils and may not necessarily be applicable to other soil types, such as the silty sand soils encountered at this site. Regardless, the calculated hydraulic conductivity values presented below are considered to be a reasonable estimate for the soil samples collected. The table below also provides the estimated seepage rate (infiltration rate) of the soils, as determined from the empirical relationship between hydraulic conductivity and infiltration rate presented in the Ontario Ministry of Municipal Affairs and Housing (OMMAH) Supplementary Guidelines to the Ontario Building Code 1997.

Pond ID	Soil Type	Hydraulic Conductivity (cm/s)	Seepage Rate (mm/hr)	Borehole No. (Sample No.)
Pond A	Silty sand fill	4×10^{-5}	35	CO5-4 (2)
	Clayey silt-silt fill	2×10^{-6}	15	CO5-5 (3)
	Silt	2×10^{-6} to 6×10^{-5}	15 to 40	CO5-5 (4, 7)
	Silty Sand	1×10^{-4} to 4×10^{-3}	45 to 125	CO5-4 (9), CO5-5 (10)
	Silt to gravelly silty sand till	2×10^{-6} to 4×10^{-5}	15 to 35	CO5-4 (5, 8)
Pond B	Silty sand fill	3×10^{-5} to 5×10^{-3}	35 to 130	HF-6 (2A), BH15-06 (6)
	Gravelly Clayey Sand	4×10^{-5}	35	BH15-06 (9)
	Gravelly silty sand	2×10^{-5} to 4×10^{-4}	30 to 70	HF-5 (2), HF-6 (3, 4)
	Silty sand	5×10^{-4}	70	HF-5 (8), HF-6 (6)
	Sand and gravel to gravel and sand	1×10^{-2}	160	HF-6 (9, 11)
	Silty sand till	2×10^{-3}	105	BH15-06 (13)

6.2.3 Pond Liners

Considering Pond A is a wet pond with a permanent pool level at Elevation 254.7 m, and the groundwater level at Pond A is about 7.9 m below the pond base, a pond liner system is required at Pond A to maintain the water level within the pond.

Considering Pond B is a dry pond and the groundwater level at Pond B is about 2.5 m to 3.2 m below the base of Pond B, a pond liner system is not required at Pond B, provided a separation barrier is not required between the natural groundwater and the stormwater contained within the pond. However, the soil at the base of Pond B will need to be protected to resist erosion from the anticipated stormwater flows as discussed in Section 6.2.5.

It is recommended that the pond liner for Pond A extend along the entire base of the pond and up the pond side slopes to the access road elevation. There are typically two liner options; a geosynthetic liner (e.g., a geosynthetic clay liner or a geomembrane) or a clay soil liner (i.e., compacted low-permeability clay soil). Given that local sources of clay soil do not appear to be readily available at the site, a geosynthetic liner is understood to be the preferred option.

The geosynthetic liner should be placed over a 150 mm thick levelling pad comprised of Concrete Sand as per OPSS.PROV 1002 (*Aggregates – Concrete*), or equivalent, as per the manufacturer's specification. Prior to the placement of the levelling pad, all topsoil, organic material, and deleterious soils should be stripped from below the levelling pad footprint. The subgrade should be checked by a geotechnical engineer / qualified personnel to identify any soft/loosened areas and/or poorly performing areas to be subexcavated and replaced with suitable engineered fill. Proof-rolling may be recommended to confirm a suitable foundation base prior to placement of the levelling pad.

The levelling pad should be placed and compacted in accordance with OPSS.PROV 206 (*Grading*) and OPSS.PROV 501 (*Compacting*), as amended by SP 105S22, compacted to 98% of the material's Standard Proctor maximum dry density. The top of the geosynthetic liner should be anchored in place by embedding it within a shallow trench, approximately 450 mm in width and 600 mm depth, that is backfilled with compacted soil. The geosynthetic liner should be provided with a minimum of 0.3 m thick cover. The cover should consist of Select Subgrade Material (SSM), with maximum particle size of 50 mm. The cover should be placed using a Low Ground Pressure (LGP) dozer, with a maximum track pressure of 35 kPa. If it is anticipated that pond cleaning will be carried out using mechanical dredging techniques, an additional 0.3 m thick layer of Granular 'B' Type II should be

placed above the soil cover. An example NSSP for installation of the geosynthetic clay liner is provided in Appendix D.

The liner construction must be conducted under full-time supervision of a qualified geotechnical engineer acting as the client representative.

6.2.4 Pond Base Stability

Hydraulic blowout is a failure of an excavation base resulting from not having enough gravitational support (weight of overlying deposits above any water bearing stratum) against the sub-artesian upward pressure heads within the underlying aquifer deposits. Given that the existing groundwater level is about 7.9 m below the base of Pond A and about 2.5 m to 3.2 m below the base of Pond B, there is negligible risk of hydraulic blowout which could lead to rupture or failure of the pond base / liner system.

6.2.5 Berms / Erosion Protection

Where grade raise is required for construction of the berms surrounding the pond, the berms shall be constructed using engineered fill in accordance with Section 6.3.3.

The requirements for the design of erosion protection measures for the inlet and outlet storm sewer pipes of the ponds should be assessed by the hydraulic design engineer based on anticipated flow velocities and design precipitation events. As a minimum, rip-rap treatment for the inlet and outlet of the storm sewer pipes should be consistent with the standard presented in OPSD 810.010 (*Rip-Rap Treatment for Sewer and Culvert Outlets*) Rip-Rap Treatment Type A, and OPSD 810.020 (*Rip-Rap Treatment for Ditch Inlets*). The rip-rap should be placed to above pipe obverts, in combination with any cut-off headwalls or clay seals. Rip-rap should be provided over the full extent of the side slopes and base grade below and adjacent to any sewer inlet and outlet locations.

The pond / berm slopes above the permanent pool elevation at Pond A, and above the pond base at Pond B, should be vegetated as soon as practicable after construction to minimize the potential for erosion due to surface water run-off, either by placement of topsoil in accordance with OPSS.PROV 802 (*Topsoil*) and seeding in accordance with OPSS.PROV 804 (*Temporary Erosion Control*) or pegged sod in accordance with OPSS.PROV 803 (*Vegetative Cover*). The soil at the pond base and interior side-slopes (i.e. anticipated wetted perimeter) should be assessed by the hydraulic engineer / pond designer to resist erosion during the design stormwater event. Vegetation may be adequate, otherwise, granular sheeting or rip-rap may be required depending on the anticipated flow velocities.

If vegetation protection is not in place before the winter season, an alternate protection, such as covering the slopes with stone or gravel sheeting or temporary erosion control blankets, is recommended to reduce the potential for remedial works required on the side slopes in the spring season prior to topsoil and seeding.

6.2.6 Global Stability

Limit equilibrium global slope stability analyses were carried out for the proposed pond / berm side slopes using the commercially available program Slide (version 9.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. The Factors of Safety (FoS) of numerous potential failure surfaces were computed to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, Φ_{gu} (i.e., $FoS = 1 / (\Psi * \Phi_{gu})$).

A target minimum FoS of 1.3 and 1.5 have been used for design of the temporary and permanent pond configurations, as per Table 6.2 of CHBDC (2019) using total stress (short-term, undrained) and effective stress (long-term, drained) conditions, respectively.

The parameters used in the global stability analysis are summarized in the table below and have been established based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990). A groundwater level at Elevation 245.6 m and 241.1 m has been used for the analysis at Pond A and B, respectively.

Pond ID	Stratigraphic Unit	γ (kN/m ³)	ϕ' (°)	S_u (kPa)
Pond A	New Engineered Fill (Select Subgrade Material)	20	33	-
	Existing Silty Sand Fill (Compact to very dense)	20	33	-
	Silt to silty sand (Very dense)	20	33	-
Pond B	Existing Silty Sand Fill (Compact to very dense)	20	33	-
	Gravelly Clayey Sand Till (Stiff to very stiff)	21	35	-
	Sandy Gravel (Dense to very dense)	21	35	-
	Silt to silty sand (Very dense)	20	33	-

The stability analyses indicate that the proposed pond side slopes have a Factor of Safety greater than 1.5 against global instability for the permanent condition. Given the non-cohesive soil conditions encountered, the temporary condition is considered not applicable. The results of the stability analyses are shown on Figures C-1 and C-2 in Appendix C.

The analyses noted above should be reviewed and confirmed by the Design-Builder, utilizing the final pond configurations. Additional analyses should be carried to incorporate the W-N Ramp embankment side slopes to the Pond A side slopes and to incorporate the proposed S-E/W Ramp embankment side slopes to the Pond B side slopes, once the embankment geometry of the ponds and ramps are confirmed.

6.3 Construction Considerations

6.3.1 Excavations

As summarized in Section 6.2.1, excavations for Pond A and B are anticipated to extend up to about 2.2 m and 10.1 m below existing ground surface, respectively. Excavations are indicated to extend through topsoil, existing fill, and gravelly clayey sand.

All 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. According to the OHSA, the overburden soils above the groundwater level are classified as Type 3 and the overburden soils below the groundwater level are classified as Type 4. Unsupported temporary excavations within Type 3 soils should be made with side slopes no steeper than 1 Horizontal: 1 Vertical (1H:1V) and unsupported temporary excavations within Type 4 soils should be made with side slopes no steeper than 3H:1V. However, depending upon the construction procedures, season of year, weather conditions and groundwater seepage conditions at the time of construction / excavation, some local flattening of the slopes may be required, especially in looser zones or where localized seepage is encountered. Permanent excavations must be in accordance with OPSS.PROV 206 (*Grading*).

All excavated material should be stockpiled away from the sides of the excavation as per the OHSA. Care must also be taken during excavation to ensure that adequate support is provided for any existing structures, roadways and underground services located adjacent the excavation. Although not anticipated, if temporary protection systems are required, they should be in accordance with OPSS.PROV 539 (*Temporary Protection Systems*) at Performance Level 2.

6.3.2 Groundwater / Surface Water Control During Construction

Excavation for Pond A is not anticipated to extend below the groundwater level. As such, pumping from properly filtered sump pumps within the excavations would be adequate to control any water seepage into the excavations during the construction of the ponds. Further, no groundwater control measures are expected to be required during the fully drained scenario for Pond A maintenance operations.

Based on the water level measured inside the open borehole upon completion of drilling at Borehole HF-6 (located about 30 m from Pond B), excavations for Pond B are anticipated to extend about 0.1 m below the groundwater level. Therefore, dewatering may be required to allow for construction (i.e. excavations, placement of engineered fill and compaction) in dry conditions. Dewatering may require installation of a series of perimeter well points, water wells, or an eductor system. The groundwater level should be drawn down to at least 1 m below the base of the excavation over the entire pond footprint. It is recommended that the Design-Builder carry out further assessment of the groundwater level at the location of Pond B to confirm the need for dewatering at this location.

Water takings in excess of 50,000 L/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater and stormwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MECP Environmental Activity and Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking and a Section 53 approval for discharge of water to the environment. A "Water Taking Plan" and a "Discharge Plan" are required by the MECP if water is taken in accordance with an EASR. In all cases, discharge under the EASR must be in accordance with a Discharge Plan (to be developed by a qualified professional). The Contractor will be responsible for obtaining any required discharge approvals and EASR registration. A Category 3 PTTW would be required for water takings in excess of 400,000 L/day.

Should dewatering be required, dewatering must be in accordance with OPSS.PROV 517 (Dewatering) and MTO's SP 517F01 (Temporary Flow Passage System). A sample SP is provided in Appendix D. Given the limited dewatering efforts anticipated to be required for construction at this site and the predominately cohesionless soils, a preconstruction survey is not considered to be required this work. It is recommended that the design engineer have a minimum of 5 years' experience is designing systems of similar nature and scope to the required work and therefore the fill-in in Table A of Special Provision 517F01 should indicate "Yes" for the "Design Engineer Requirements". The remaining fill-ins of Special Provision 517F01 should be provided by the drainage engineer.

Surface water should be directed away from the excavation areas to prevent ponding and/or flowing water that could result in disturbance and loosening/softening of the subgrade soils.

6.3.3 Engineered Fill

The existing site soils (anticipated to be available from excavation of Pond B and/or other cut areas at the site) that do not contain topsoil, organics, or any other deleterious materials can be reused on site as engineered fill, provided they satisfy the gradation of OPSS.PROV 1010 Select Subgrade Material (SSM). Based on the measured natural water contents, the existing site soils are generally at or above their estimated optimum water contents for compaction and therefore soil "wetting" will likely not be required; however, some drying may be necessary to

achieve the required compaction levels. Alternatively, imported materials meeting the required of OPSS.PROV 1010 (Aggregates) Granular 'B' Type I or Select Subgrade Material (SSM) may be used for engineered fill.

Following proof-rolling and approval of the subgrade, engineered fill should be placed in accordance with OPSS.PROV 206 (*Grading*) and OPSS.PROV 501 (*Compacting*), as amended by SP 105S22, compacted to 98% of the material's Standard Proctor maximum dry density.

The final surface of engineered fill should be protected as necessary from construction traffic and should be sloped to provide positive drainage for surface water during the construction period.

6.3.4 Obstructions

The Design-Build Contractor should be alerted to the potential presence of cobble and boulder obstructions within the existing fill and native soils as noted on the borehole records. The potential presence of cobble and boulder obstructions has been inferred based on the presence of rock fragments within the collected soil samples and several instances of auger grinding and split-spoon refusal. Further, glacially derived till deposits, such as those encountered at this site, should be expected to contain coarse gravel, cobbles and/or boulders. Note that the extent and depth of the cobble and boulder obstructions may vary beyond and between the borehole locations.

The presence of obstructions (i.e., cobbles and/or boulders) may affect excavation operations for pond construction. The Design-Build Contractor must be prepared with suitable equipment and procedures to remove/penetrate through any obstructions that may be encountered during construction.

6.3.5 Piezometer Decommissioning

A piezometer was installed in Borehole CO5-4 to monitor the groundwater levels at the location of Pond A. The piezometer has been left in place to allow the Design-Build Contractor to obtain additional groundwater level readings during the detailed design stage and prior to / during construction. The existing standpipe piezometer shall be decommissioned by the Design-Build Contractor according to applicable law (O. Reg. 903, as amended) following the initial works.

6.3.6 Additional Work

It is recommended that the Design-Build Contractor carry out further assessment of the subsurface conditions, including the groundwater level, at the location of Pond B. The assessment should confirm the presence / depth of deleterious soils (i.e., soft / loose soils and/or soils containing organic material) below the pond base, which would require sub-excavation. In addition, the assessment should determine the groundwater level at the pond location to confirm the need for dewatering during construction of Pond B.

The global stability analyses carried out of the stormwater management ponds should be reviewed and confirmed by the Design-Builder, utilizing the final pond configurations. Additional analyses should be carried to incorporate the W-N Ramp embankment side slopes to the Pond A side slopes and to incorporate the proposed SE/W Ramp embankment side slopes to the Pond B side slopes once the embankment geometry of the ponds and ramps are confirmed.

6.0 Closure

The Foundation Design Report was prepared by Ms. Anastasia Poliacik, P.Eng., a Senior Geotechnical Engineer with WSP. Mr. David Staseff, P.Eng., a Senior Principal and MTO Principal Foundations Contact with WSP conducted an independent technical and quality review of this report.

Signature Page

WSP Canada Inc.



Anastasia Poliacik, P.Eng.
Senior Geotechnical Engineer



David Staseff, P.Eng.
Senior Principal, MTO Foundations Designated Contact

AMP/DS/al

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REFERENCES

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- Canadian Standard Association (CSA) Group. *Canadian Highway Bridge Design Code (CHBDC (2019)) and Commentary on CAN/CSA-S6-14*.
- 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.
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- OMMAH, 1997. Ontario Ministry of Municipal Affairs and Housing, Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario.

Ontario Provincial Standard Drawing (OPSD)

- | | |
|--------------|--|
| OPSD 810.010 | General Rip-Rap Layout for Sewer and Culvert Outlets |
| OPSD 810.020 | General Rip-Rap Layout for Ditch Inlets |

Ontario Provincial Standard Specifications (OPSS)

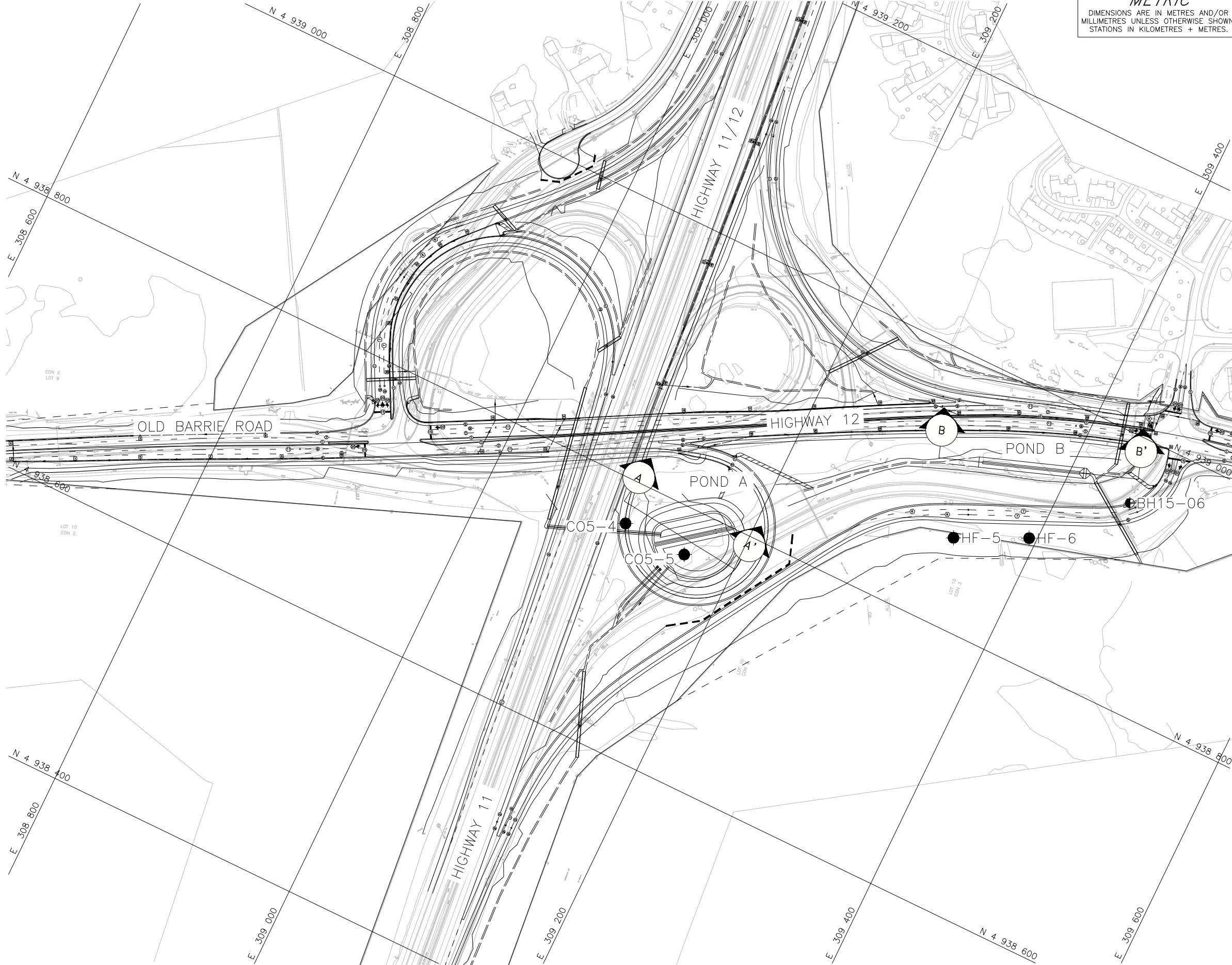
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|----------------|---|
| OPSS.PROV 206 | Construction Specification for Grading |
| OPSS.PROV 501 | Construction Specification for Compacting |
| OPSS.PROV 539 | Construction Specification for Temporary Protection System |
| OPSS.PROV 517 | Construction Specification for Dewatering |
| OPSS.PROV 802 | Construction Specification for Topsoil |
| OPSS.PROV 803 | Construction Specification for Vegetative Cover |
| OPSS.PROV 804 | Construction Specification for Temporary Erosion Control |
| OPSS.PROV 1002 | Material Specification for Aggregates – Concrete |
| OPSS.PROV 1010 | Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material |

Special Provision

- | | |
|--------|----------------------------|
| 105S22 | Amendment to OPSS.PROV 501 |
| 517F01 | Amendment to OPSS.PROV 517 |

Ontario Water Resources Act

- Ontario Regulation 903 Wells (as amended)



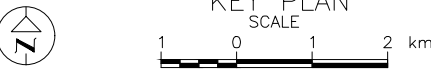
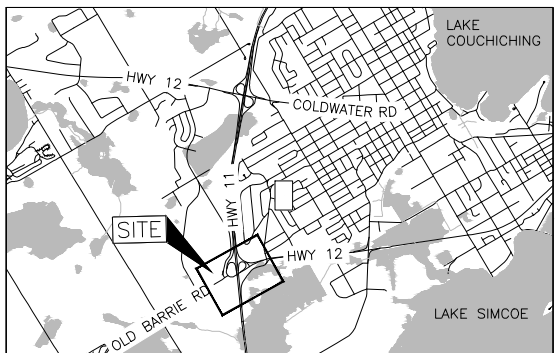
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STATIONS IN KILOMETRES + METRES.

CONT No. .
GWP No. 2129-18-00



HIGHWAY 11/12 INTERCHANGE
STORMWATER MANAGEMENT PONDS
BOREHOLE LOCATION PLAN

SHEET



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation (Geocres. No. 31D-647)

BOREHOLE CO-ORDINATES NAD 83 MTM ZONE 10			
No.	ELEVATION	NORTHING	EASTING
BH15-06	251.1	4938950.1	309452.2
CO5-4	253.0	4938767.3	309109.8
CO5-5	249.0	4938765.4	309160.8
HF-5	241.9	4938867.1	309341.7
HF-6	242.0	4938892.1	309394.1



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.

REFERENCE

Design plan provided in digital format by McIntosh Perry, drawing file no. 197147-c2_hwy011_dph-ncp-interim.dwg, received November 30, 2023.
Drainage plan in digital format by McIntosh Perry, drawing file no. x_drainage_plan.dwg, received November 30, 2023.
Base plan provided in digital format by McIntosh Perry, drawing file no. x_197147_BASE.dwg, received May 19, 2021.

REVISION			
NO.	DATE	BY	REVISION
Geocres No. .			
HWY. 11 AND 12		PROJECT NO. 19135676	DIST. .
SUBM'D. MH	CHKD. MH	DATE: 04/23/2024	SITE:
DRAWN: DD/ZS/SA	CHKD. AMP	APPD. DS	DWG. 1

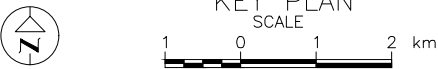
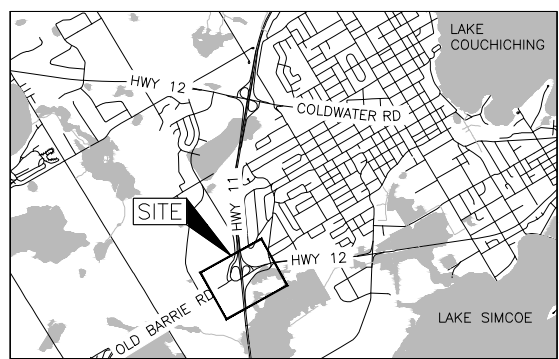
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DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. .
GWP No. 2129-18-00

HIGHWAY 11/12 INTERCHANGE
STORMWATER MANAGEMENT PONDS

SOIL STRATA

SHEET



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation (Geocres. No. 31D-647)
- ⊥ 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 January 19, 2024
- ≡ WL upon completion of drilling

BOREHOLE CO-ORDINATES NAD 83 MTM ZONE 10

No.	ELEVATION	NORTHING	EASTING
BH15-06	251.1	4938950.1	309452.2
C05-4	253.0	4938767.3	309109.8
C05-5	249.0	4938765.4	309160.8
HF-5	241.9	4938867.1	309341.7
HF-6	242.0	4938892.1	309394.1

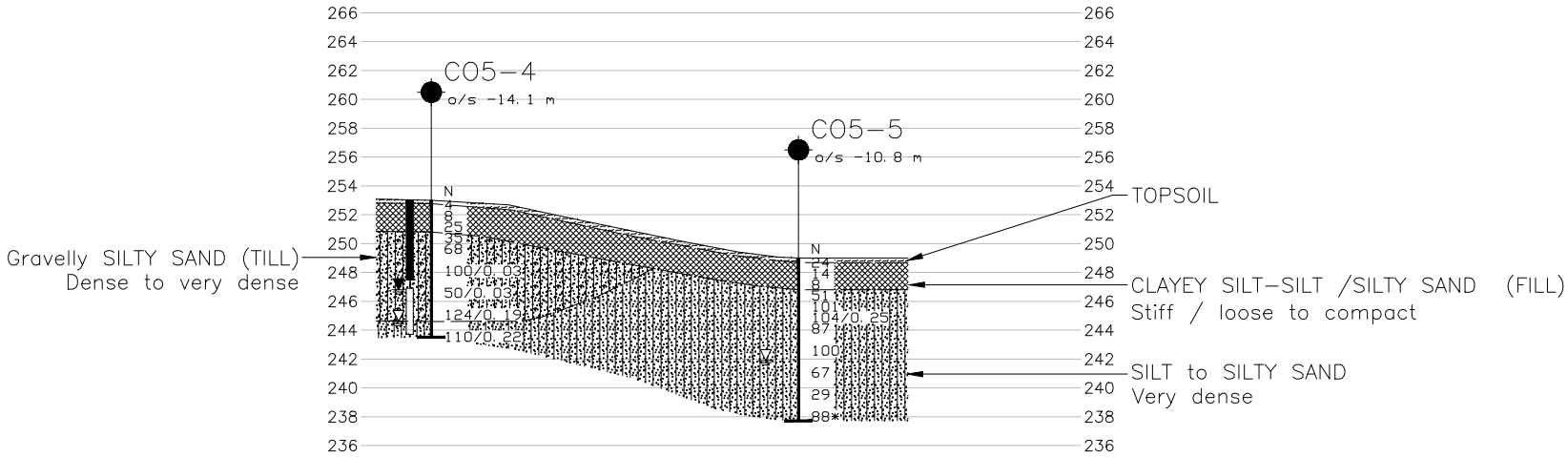
NOTES

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.

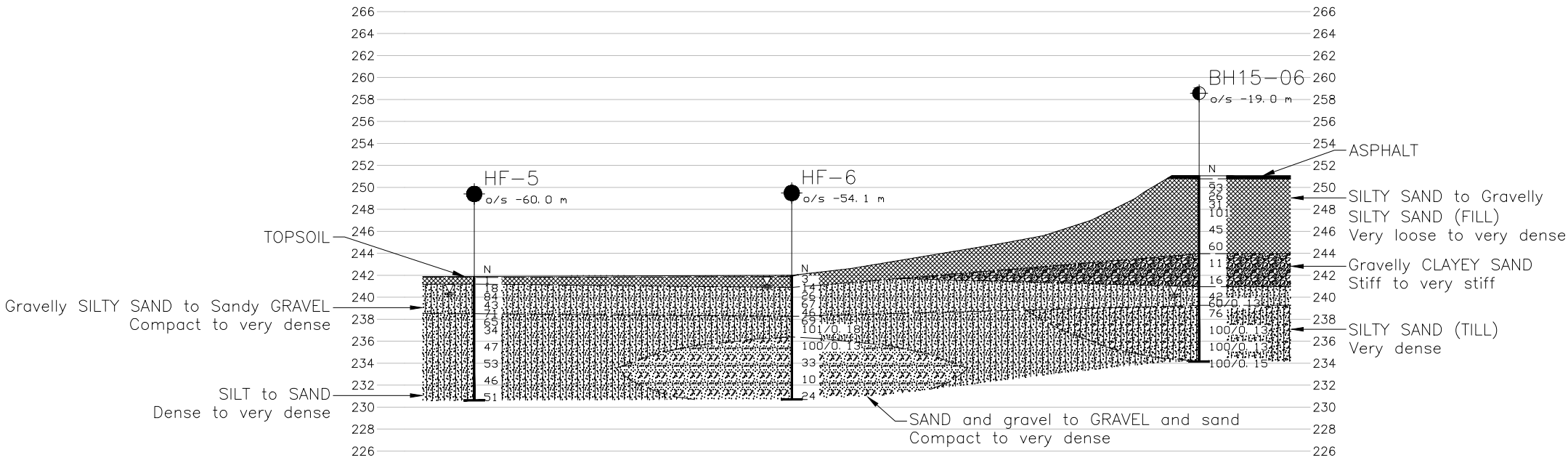
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

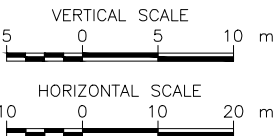
Design plan provided in digital format by McIntosh Perry, drawing file no. 197147-c2_hwy011_dph-ncp-interim.dwg, received November 30, 2023.
Drainage plan in digital format by McIntosh Perry, drawing file no. x_drainage_plan.dwg, received November 30, 2023.
Base plan provided in digital format by McIntosh Perry, drawing file no. x_197147_BASE.dwg, received May 19, 2021.



PROFILE A-A'
POND A



PROFILE B-B'
POND B



NO.	DATE	BY	REVISION
Geocres No.			
HWY. 11 AND 12			PROJECT NO. 19135676
SUBM'D. MH	CHKD. MH	DATE: 04/23/2024	SITE:
DRAWN: DD/ZS/SA	CHKD. AMP	APPD. DS	DWG. 2

APPENDIX A

Record of Borehole Sheets

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

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

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

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

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Cone Penetration Test (CPT)

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

Dynamic Cone Penetration Resistance (DCPT); N_d:

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

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

SOIL TESTS

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

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

COARSE-GRAINED SOILS

Compactness¹

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

1. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

2. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{\alpha(e)}$	secondary compression index
C_{α}	rate of secondary compression
$C_{\alpha(e)}$	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

[illegible]

PROJECT <u>13-1111-0026</u>			RECORD OF BOREHOLE No BH15-06				2 OF 2 METRIC	
W.P. <u>11-20002</u>			LOCATION <u>N 4938950.1; E 309452.2</u>				ORIGINATED BY <u>DM</u>	
DIST <u>CENTRAL</u> HWY <u>12</u>			BOREHOLE TYPE <u>200 mm Diameter Hollow Stem Augers</u>				COMPILED BY <u>NL</u>	
DATUM <u>GEODETIC</u>			DATE <u>September 16 and 17, 2015</u>				CHECKED BY <u>JMAC</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W		
							<div style="display: flex; justify-content: space-between; font-size: small;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between; font-size: x-small;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>					WATER CONTENT (%)				
							<div style="display: flex; justify-content: space-between; font-size: x-small;"> 20 40 60 80 100 10 20 30 </div>					<div style="display: flex; justify-content: space-between; font-size: x-small;"> W_p W W_L </div>				
	--- CONTINUED FROM PREVIOUS PAGE ---															
	SILTY SAND to SAND, some gravel, trace clay (TILL) Very dense Brown to grey Moist to wet		12	SS	76		239									
							238									
			13	SS	100/0.13		237									
			14	SS	100/0.13		236									
							235									
234.2			15	SS	100/0.15											
16.9	END OF BOREHOLE															
	Note: 1. Water level at a depth of 10.8 m below ground surface in augers (Elev. 240.3 m) upon completion of drilling.															

SUD-MTO 001 131110026.GPJ GAL-MISS.GDT 15/04/16 DATA INPUT:



PROJECT 19135676			RECORD OF BOREHOLE No C05-4			SHEET 1 OF 1			METRIC														
G.W.P. 2129-18-00			LOCATION N 4938767.3; E 309109.8 MTM NAD 83 ZONE 10 (LAT. 44.588900; LONG. -79.445721)			ORIGINATED BY KR																	
DIST Central HWY 11/12			BOREHOLE TYPE 110 mm ID / 210 mm OD Hollow Stem Augers			COMPILED BY ML																	
DATUM Geodetic			DATE November 24 to 27, 2023			CHECKED BY AMP																	
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																		
253.0	GROUND SURFACE																						
0.0	TOPSOIL (225 mm)																						
0.2	SILTY SAND (SM), trace gravel, trace clay, containing organics and rock fragments (FILL) Loose to compact Brown Moist - Augers grinding above 1.5 m depth		1	SS	4																		
			2	SS	8																		
			3	SS	25																		
250.8	Gravelly SILTY SAND (SM), trace clay, containing rock fragments (TILL) Dense to very dense Brown Moist to wet - Augers grinding between 3.8 m and 4.1 m depth.		4	SS	35																		
2.2			5	SS	68																		
			6	SS	100/0.03																		
			7	SS	50/0.03																		
246.1	SILT (ML) some gravel, some sand, some clay (TILL) Very Dense Brown Moist		8	SS	24/0.19																		
6.9																							
244.6	SILTY SAND (SM), trace clay Very dense Brown Moist		9	SS	10/0.22																		
8.4																							
243.5	END OF BOREHOLE																						
9.5	NOTES: 1. Water level measured in open borehole at a depth of about 8.4 m below ground surface (Elevation 244.6 m) upon completion of drilling (November 26, 2023). 3. Water measured in piezometer as follows: Date Depth (m) Elev. (m) 27-Nov-23 7.8 245.2 19-Jan-24 6.2 246.8																						



PROJECT		19135676		RECORD OF BOREHOLE		No C05-5		SHEET 1 OF 1		METRIC			
G.W.P.		2129-18-00		LOCATION		N 4938765.4; E 309160.8 MTM NAD 83 ZONE 10 (LAT. 44.588883; LONG. -79.445079)		ORIGINATED BY		KR			
DIST		Central HWY 11/12		BOREHOLE TYPE		110 mm ID / 210 mm OD Hollow		COMPILED BY		ML			
DATUM		Geodetic		DATE		November 23 to 24, 2023		CHECKED BY		AMP			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W		
249.0	GROUND SURFACE												
0.0	TOPSOIL (305 mm)												
248.7			1	SS	24								
248.3	SILTY SAND (SM), some gravel, some clay, containing cobbles (FILL)		2	SS	14								
0.7	Compact Brown Moist												
246.8	CLAYEY SILT-SILT (CL-ML) some sand, trace gravel, containing organics and rock fragments (FILL)		3	SS	8								
2.2	Stiff Brown Moist to wet		4	SS	51								
	SILT (ML), trace to some sand, trace to some clay, trace gravel		5	SS	101								
	Very dense Brown Moist to wet		6	SS	104/0.25								
			7	SS	87								
243.4	SILTY SAND (SM), trace clay												
5.6	Very dense Brown Moist to wet		8	SS	100								
	- Wet below 7.6 m depth		9	SS	67								
	- About 0.3 m of sand heave was measured inside the augers after drilling to 9.1 m depth		10	SS	29*								
	- About 1.1 m of sand heave was measured inside the augers after drilling to 10.7 m depth		-	-	88*								
237.7	END OF BOREHOLE												
11.3													
NOTES:													
1.*Indicates "N" value may be disturbed due to sand heave.													
2. Water level measured inside augers at a depth of 11.3 m below ground surface (Elev. 237.7 m) upon completion of drilling.													
3. Water level and cave measured at depths of 7.1 m and 7.7 m below ground surface, (Elev. 241.9 m and 241.3 m), respectively, upon completion of drilling and removal of augers.													



SUD-MTO 001 \\GOLDER.GPS\COMPLEX\DATA\OFFICE\ONTARIO\CLIENTS\MTOWHY_11&12_OLD_BARRIE_RD_TO_COLDWATER_RD.GPJ GAL-MISS.GDT 24-1-22

PROJECT 19135676		RECORD OF BOREHOLE No. HF-5				1 OF 1 METRIC							
G.W.P. 2129-18-00		LOCATION N 4938867.1; E 309341.7 NAD83 MTM ZONE 10 (LAT. 44.58980565; LONG. -79.44280319)				ORIGINATED BY ML							
DIST Central HWY 11/12		BOREHOLE TYPE 115 mm O.D. Solid Stem Augers				COMPILED BY MH							
DATUM Geodetic		DATE April 28 and 29, 2022				CHECKED BY MH/AMP							
SOIL PROFILE			SAMPLES			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	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL	
241.9	GROUND SURFACE												
0.0	TOPSOIL (100 mm)												
241.2	SILTY SAND (SM), trace gravel (FILL) Very loose Brown Moist		1	SS	1		241					25 46 24 5	
0.7	Gravelly SILTY SAND (SM), trace clay, containing rock fragments Compact to very dense Brown Moist		2	SS	18		240						
	- Augers grinding below 2.3 m depth		3	SS	84		239						
			4	SS	43		238						
238.5	SILTY SAND (SM), trace to some gravel, trace clay Dense to very dense Brown Moist		5A	SS	71		237						
3.4	- Augers grinding from 3.4 m to 4.4 m depth		5B	SS	65		236						
			6	SS	65		235						
			7	SS	34		234						
			8	SS	47		233						
			9	SS	53		232						
233.2	SAND (SP-SM), some gravel, trace silt Dense Brown Moist		10	SS	46		231					14 78 7 1	
231.7	SILTY SAND (SM), trace to some gravel Dense Brown		11	SS	51								
230.6	END OF BOREHOLE												
11.3	NOTES: 1. Water level measured at a depth of 0.9 m below ground surface (Elev. 241.0 m) upon completion of drilling. 2. Borehole caved to a depth of 3.0 m below ground surface (Elev. 238.9 m) upon completion of drilling.												

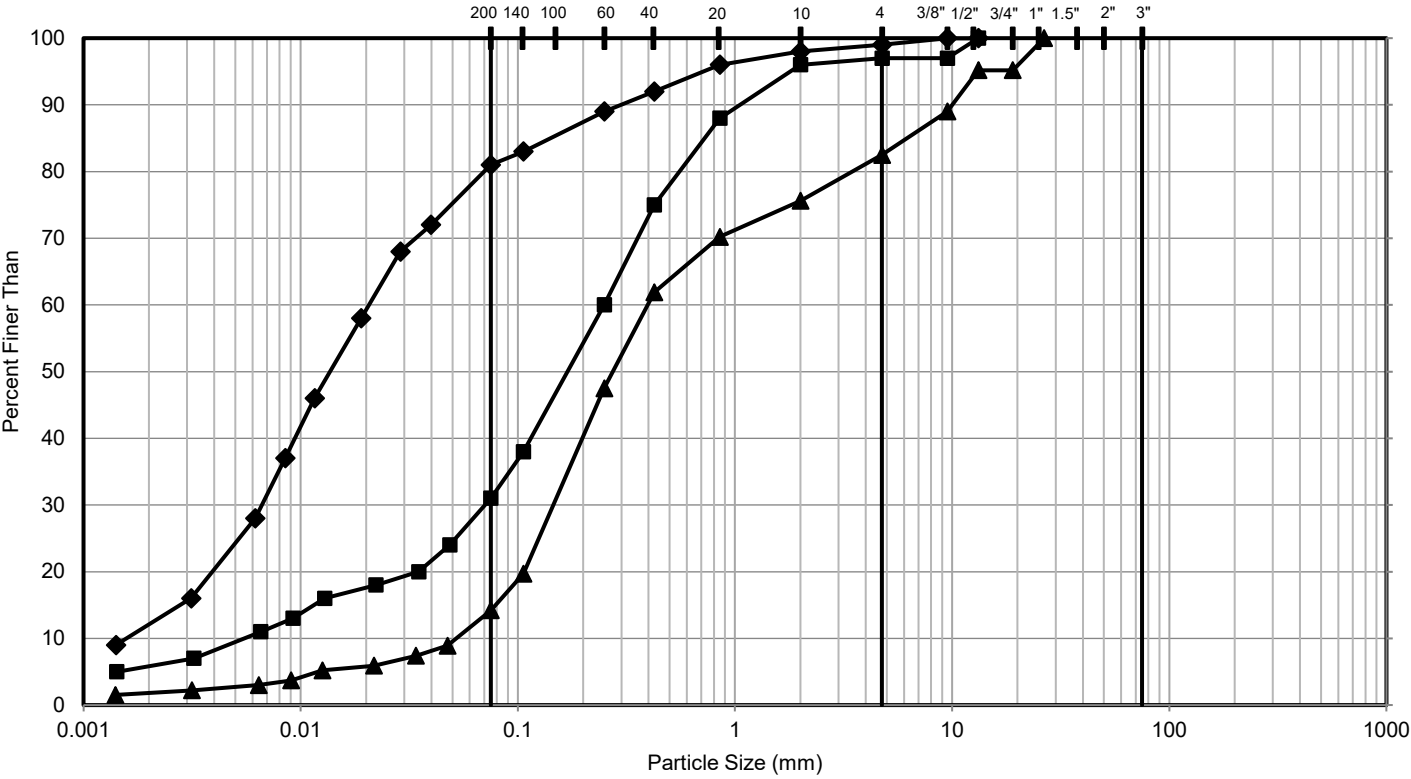


PROJECT		19135676		RECORD OF BOREHOLE No. HF-6				1 OF 1 METRIC																			
G.W.P.		2129-18-00		LOCATION				N 4938892.1; E 309394.1 NAD83 MTM ZONE 10 (LAT. 44.5900307; LONG. -79.44214207)		ORIGINATED BY		MTI															
DIST		Central		HWY		11/12		BOREHOLE TYPE				115 mm O.D. Solid Stem Augers				COMPILED BY		ML									
DATUM		Geodetic		DATE		April 29, 2022		CHECKED BY				MH/AMP															
SOIL PROFILE				SAMPLES				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		GROUND WATER CONDITIONS		ELEVATION SCALE		SHEAR STRENGTH kPa		W _p W W _L		WATER CONTENT (%)		γ		GR SA SI CL	
242.0		0.0		GROUND SURFACE				1A		SS		3				241										18 68 12 2	
240.9		1.1		TOPSOIL (100 mm)				1B		SS		14				240										40 40 17 3	
				SILTY SAND (SM), trace gravel (FILL)				2A		SS		26				239										32 41 23 4	
				Very loose to compact				2B		SS		46				238										3 72 23 3	
				Brown												237											
				Moist												236											
				Gravelly SILTY SAND (SM) to SILTY SAND (SM) and gravel, trace clay, containing rock fragments				3		SS		67				235										36 55 7 2	
				Compact to very dense				4		SS		10				234											
				Brown to grey				5		SS		101/0.18				233											
				Moist to wet				6		SS		100/0.13				232											
				- Auger grinding at 1.5 m depth				7		SS		101/0.18				231										48 44 6 2	
238.3		3.7		SILTY SAND (SM), trace gravel, trace clay				8		SS		100/0.13															
				Very dense																							
				Brown																							
				Moist																							
236.4		5.6		SAND (SP-SM) and gravel to GRAVEL (GP-GM) and sand, trace silt, trace clay				9		SS		33															
				Compact to very dense				10		SS		10															
				Brown to grey																							
				Moist to wet				11		SS		24															
230.7		11.3		END OF BOREHOLE																							
				NOTE:																							
				1. Water level measured at a depth of 0.9 m below ground surface (Elev. 241.1 m) upon completion of drilling on April 29, 2022.																							

APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	CO5-4	2	0.8 - 1.4	252.2 to 251.6
◆	CO5-5	3	1.5 - 2.1	247.5 to 246.9
▲	HF-6	2A	0.8 - 1.1	241.2 to 240.9

CLIENT

MCINTOSH PERRY / MINISTRY OF TRANSPORTATION
ONTARIO (MTO)

CONSULTANT



YYYY-MM-DD 2024-01-23

DESIGNED ML

PREPARED ML

REVIEWED AMP

APPROVED DS

PROJECT

HIGHWAY 11 / 12 (OLD BARRIE ROAD)
STORMWATER MANAGEMENT PONDS A AND B

TITLE

Grain Size Distribution - SILTY SAND (SM) / CLAYEY
SILT-SILT (CL-ML) (FILL)

PROJECT NO.

19135676

CONTROL

0

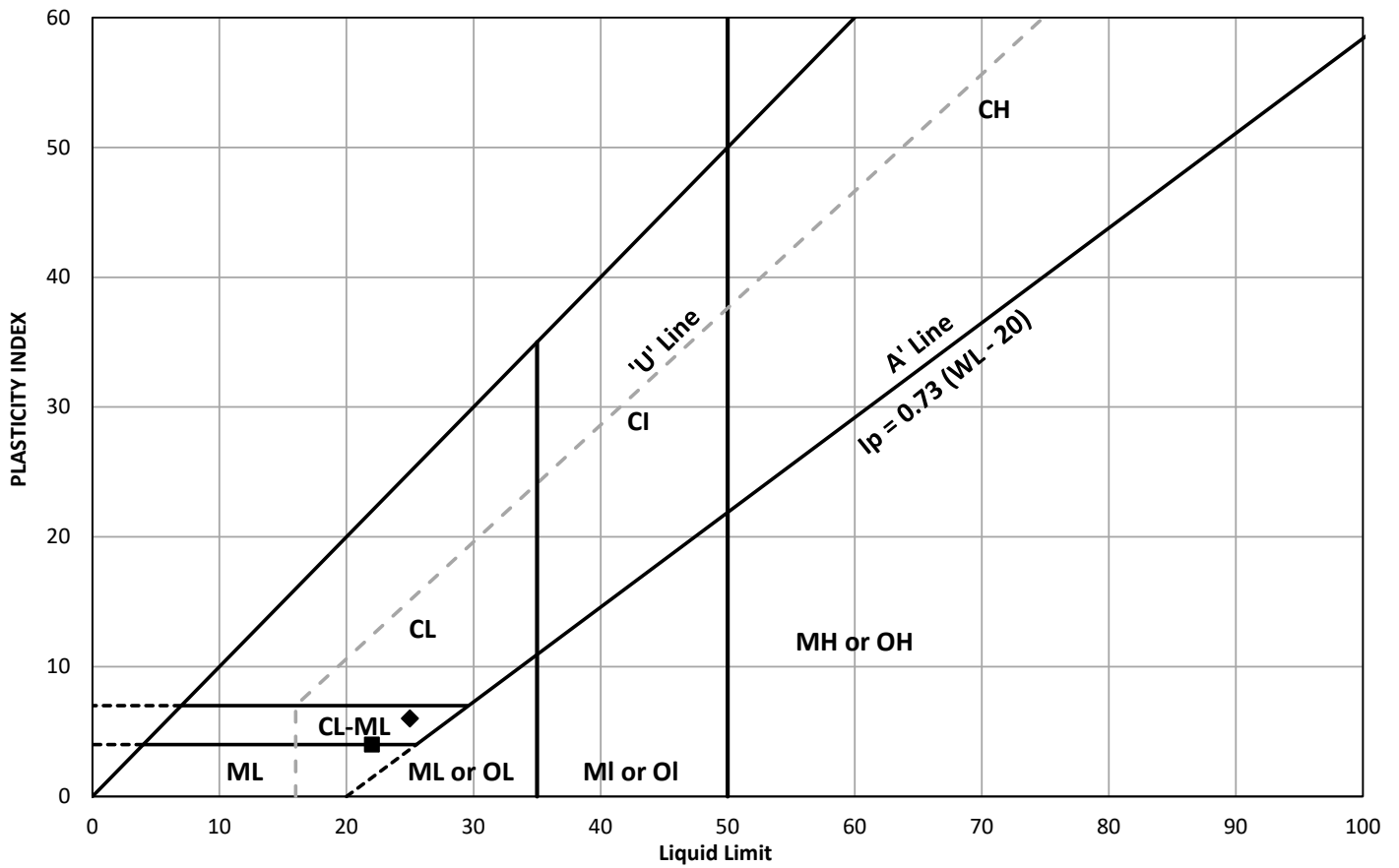
REV.

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FIGURE

B-1


PLASTICITY CHART



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
■	CO5-4	2	0.8 - 1.4	12.8	22	18	4	-1.30
◆	CO5-5	3	1.5 - 2.1	21.7	25	19	6	0.45

CLIENT
MCINTOSH PERRY / MINISTRY OF TRANSPORTATION ONTARIO (MTO)

PROJECT
HIGHWAY 11 / 12 (OLD BARRIE ROAD)
STORMWATER MANAGEMENT PONDS A AND B



CONSULTANT

YYYY-MM-DD

2024-01-23

DESIGNED

ML

PREPARED

ML

REVIEWED

AMP

APPROVED

DS

TITLE

Plasticity Chart - SILTY SAND (SM) / CLAYEY SILT-SILT (CL-ML) (FILL)

PROJECT NO.

CONTROL

REV.

FIGURE

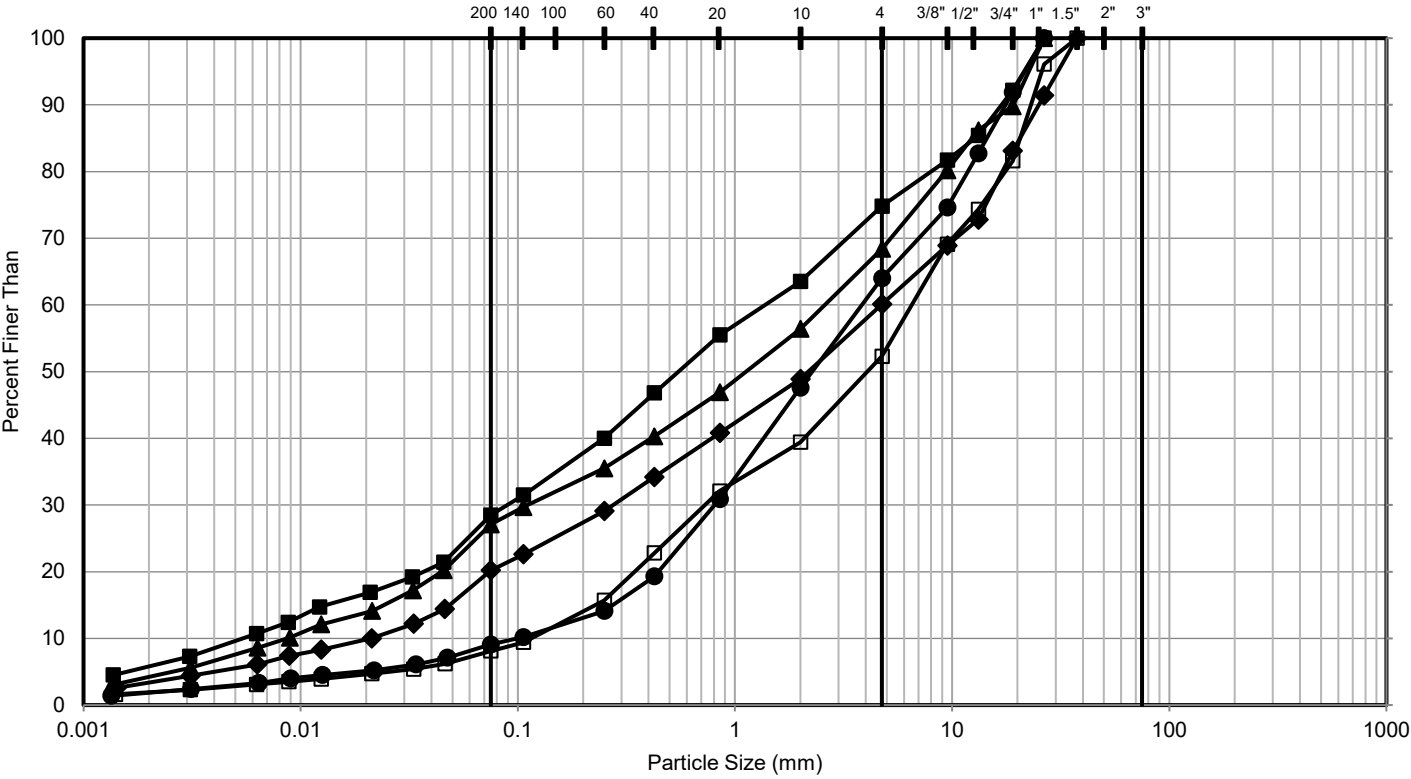
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B-2

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HF-5	2	0.8 - 1.4	241.1 to 240.5
◆	HF-6	3	1.5 - 2.1	240.5 to 239.9
▲	HF-6	4	2.3 - 2.9	239.7 to 239.1
●	HF-6	9	7.6 - 8.2	234.4 to 233.8
□	HF-6	11	10.7 - 11.3	231.3 to 230.7

CLIENT

MCINTOSH PERRY / MINISTRY OF TRANSPORTATION
ONTARIO (MTO)

CONSULTANT



YYYY-MM-DD 2024-01-23

DESIGNED ML

PREPARED ML

REVIEWED AMP

APPROVED DS

PROJECT

HIGHWAY 11 / 12 (OLD BARRIE ROAD)
STORMWATER MANAGEMENT PONDS A AND B

TITLE

Grain Size Distribution - Gravelly SILTY SAND (SM) to
GRAVEL (GP-GM) and sand

PROJECT NO.

19135676

CONTROL

0

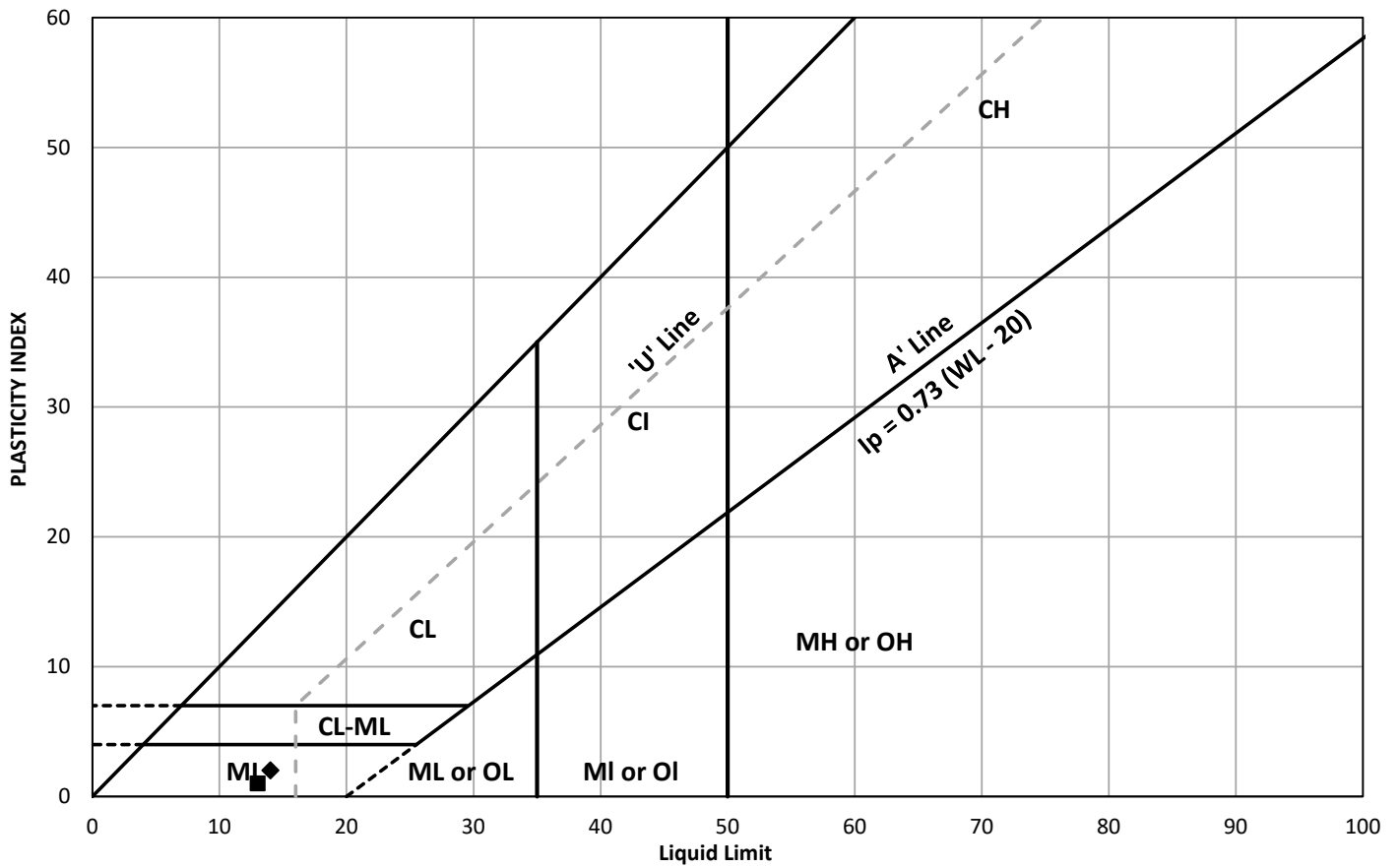
REV.

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FIGURE

B-3

PLASTICITY CHART



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
■	HF-5	2	0.8 - 1.4	8	13	12	1	-4.00
◆	HF-6	4	2.3 - 2.9	5.3	14	12	2	-3.35

CLIENT
MCINTOSH PERRY / MINISTRY OF TRANSPORTATION ONTARIO (MTO)

PROJECT
HIGHWAY 11 / 12 (OLD BARRIE ROAD)
STORMWATER MANAGEMENT PONDS A AND B

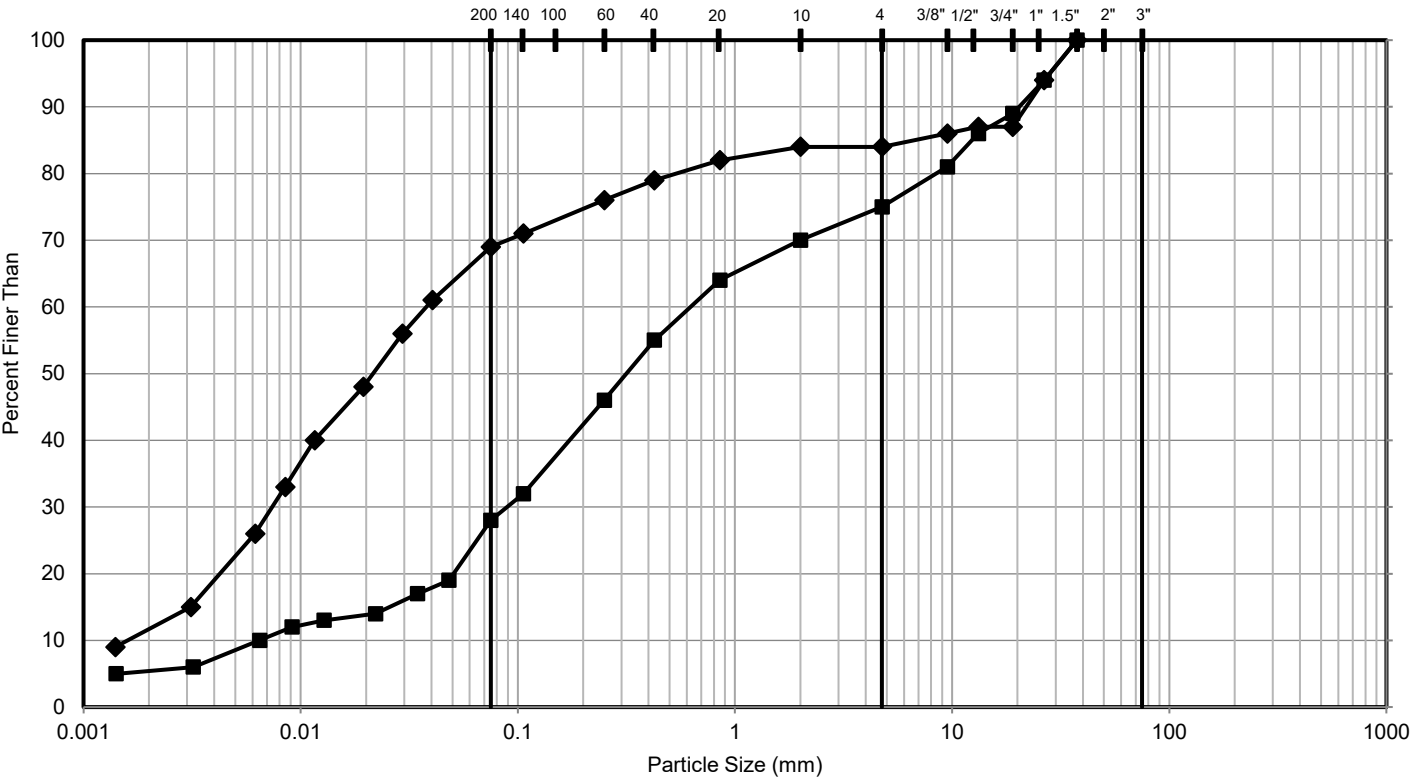


CONSULTANT	YYYY-MM-DD	2024-01-23
DESIGNED		ML
PREPARED		ML
REVIEWED		AMP
APPROVED		DS

TITLE
Plasticity Chart - Gravelly SILTY SAND (SM)

PROJECT NO.	CONTROL	REV.	FIGURE
19135676	0	0	B-4

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	CO5-4	5	3.0 - 3.7	249.9 to 249.3
◆	CO5-4	8	7.6 - 8.0	245.4 to 245.0

CLIENT

MCINTOSH PERRY / MINISTRY OF TRANSPORTATION
ONTARIO (MTO)

CONSULTANT



YYYY-MM-DD 2024-01-23

DESIGNED ML

PREPARED ML

REVIEWED AMP

APPROVED DS

PROJECT

HIGHWAY 11 / 12 (OLD BARRIE ROAD)
STORMWATER MANAGEMENT PONDS A AND B

TITLE

Grain Size Distribution - SILT (ML) to Gravelly SILTY SAND (SM) (TILL)

PROJECT NO.

19135676

CONTROL

0

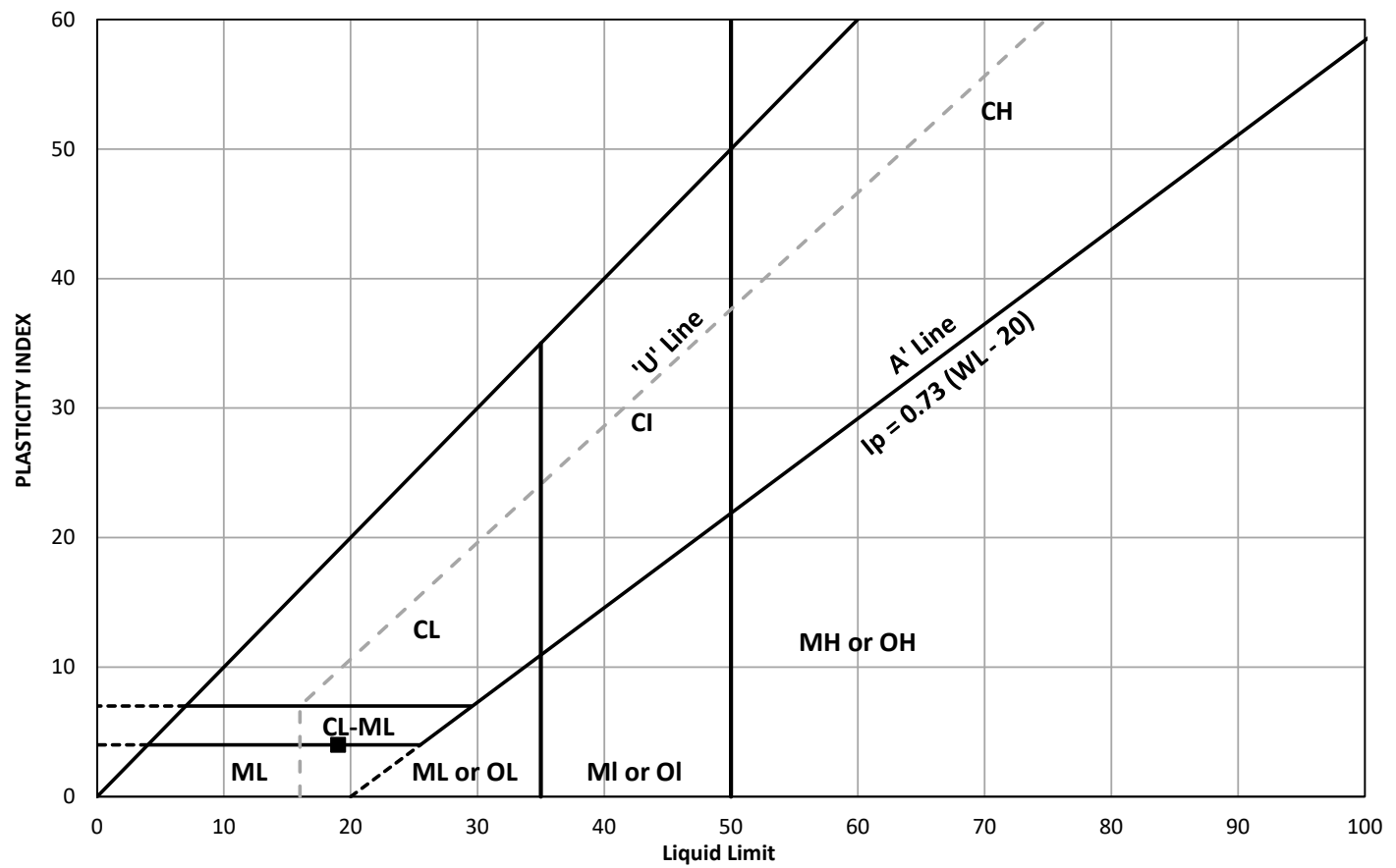
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FIGURE

B-5


PLASTICITY CHART



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
■	CO5-4	8	7.6 - 8.0	11.1	19	15	4	-0.98

CLIENT
MCINTOSH PERRY / MINISTRY OF TRANSPORTATION ONTARIO (MTO)

PROJECT
HIGHWAY 11 / 12 (OLD BARRIE ROAD)
STORMWATER MANAGEMENT PONDS A AND B



CONSULTANT

YYYY-MM-DD

2024-01-23

DESIGNED

ML

PREPARED

ML

REVIEWED

AMP

APPROVED

DS

TITLE

Plasticity Chart - SILT (ML) (TILL)

PROJECT NO.

19135676

CONTROL

0

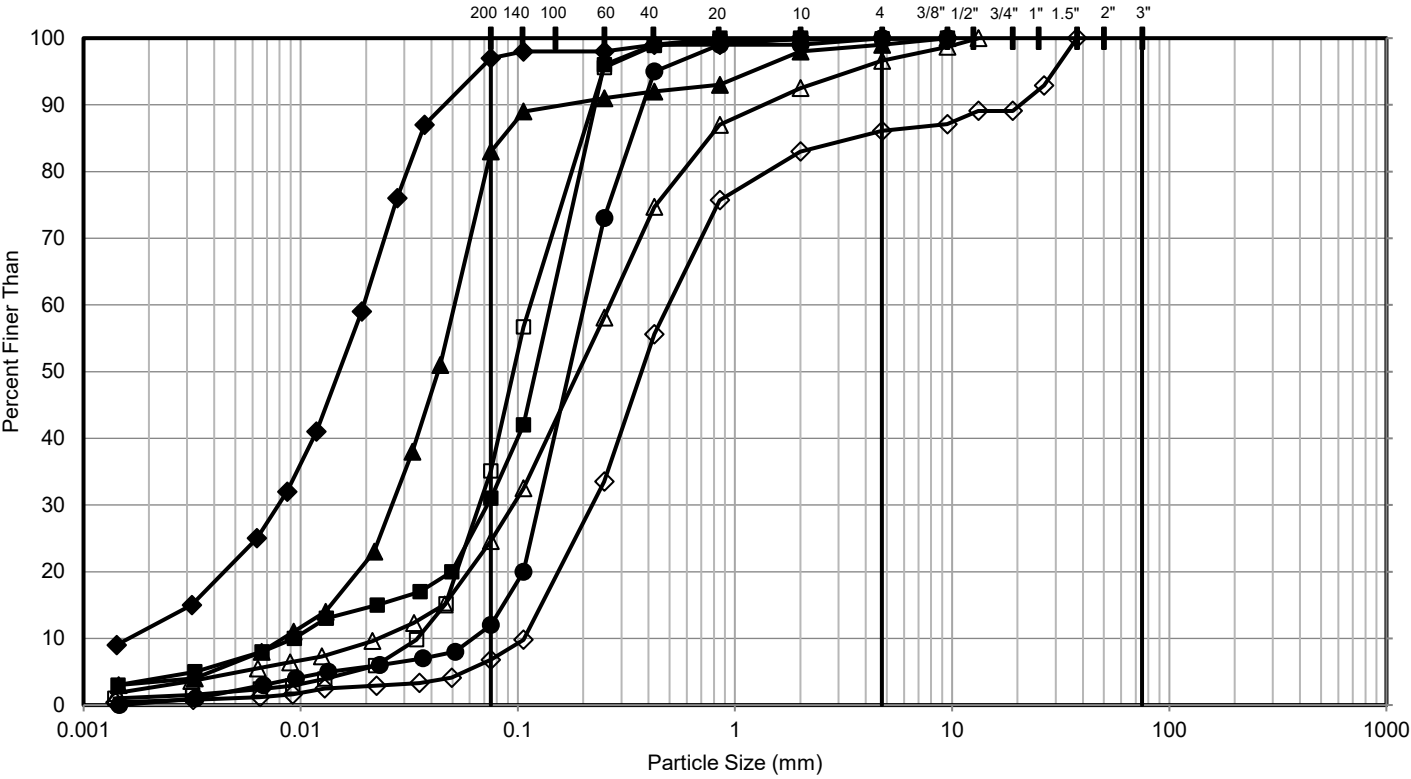
REV.

0

FIGURE

B-6

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	CO5-4	9	9.1 - 9.5	243.8 to 243.5
◆	CO5-5	4	2.3 - 2.9	73.6 to 73.0
▲	CO5-5	7	4.6 - 5.2	71.3 to 70.7
●	CO5-5	10	9.1 - 9.8	66.8 to 66.2
□	HF-5	8	6.1 - 6.7	235.8 to 235.2
◇	HF-5	10	9.1 - 9.8	232.8 to 232.2
△	HF-6	6	3.8 - 4.4	238.2 to 237.6

CLIENT

MCINTOSH PERRY / MINISTRY OF TRANSPORTATION
ONTARIO (MTO)

CONSULTANT



YYYY-MM-DD 2024-01-23

DESIGNED ML

PREPARED ML

REVIEWED AMP

APPROVED DS

PROJECT

HIGHWAY 11 / 12 (OLD BARRIE ROAD)
STORMWATER MANAGEMENT PONDS A AND B

TITLE

Grain Size Distribution - SILT (ML) to SAND (SP-SM)

PROJECT NO.

19135676

CONTROL

0

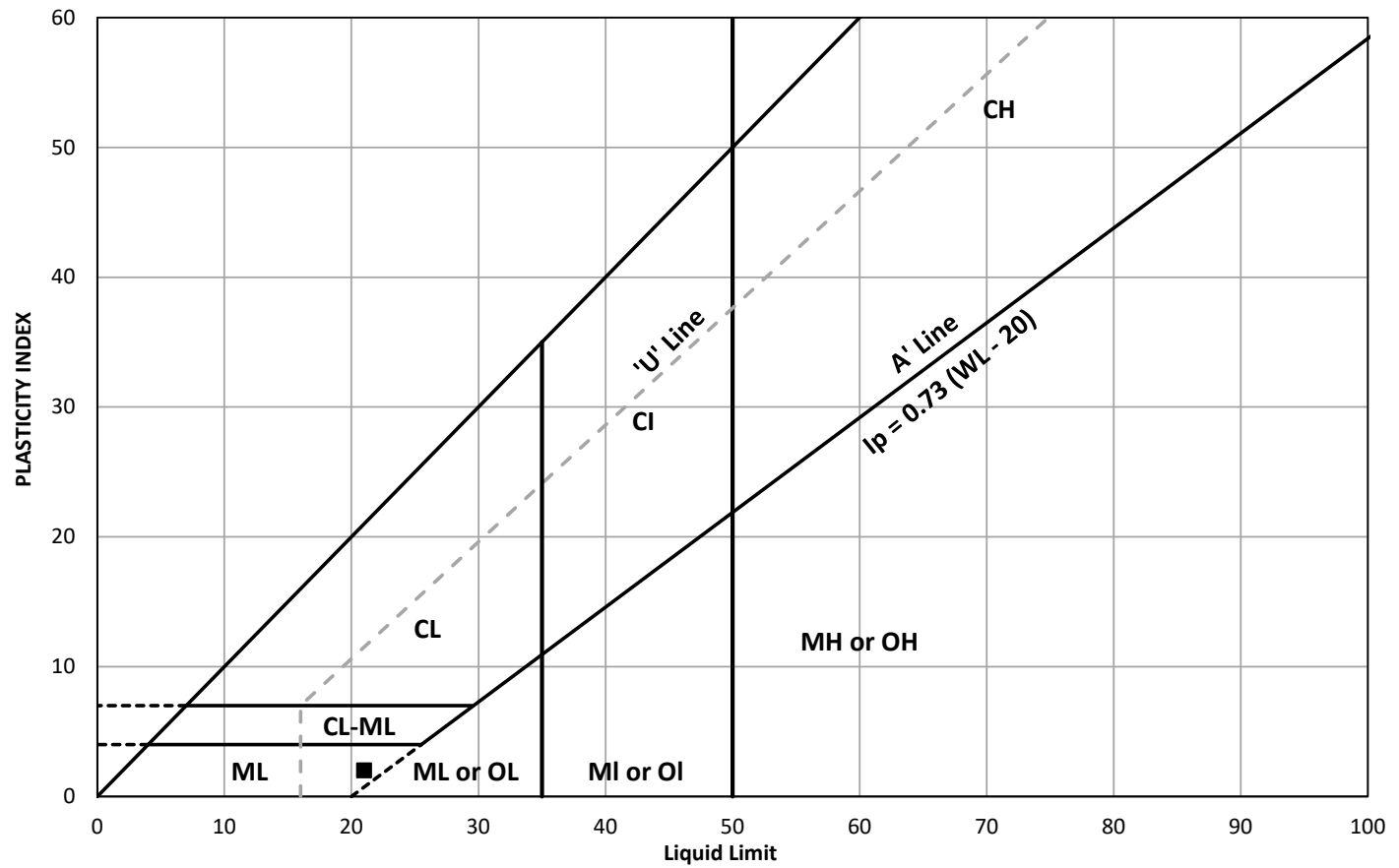
REV.

0

FIGURE

B-7

PLASTICITY CHART



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
■	CO5-5	4	2.3 - 2.9	18.2	21	19	2	-0.40

CLIENT		
MCINTOSH PERRY / MINISTRY OF TRANSPORTATION ONTARIO (MTO)		
CONSULTANT	YYYY-MM-DD	2024-01-23
	DESIGNED	ML
	PREPARED	ML
	REVIEWED	AMP
	APPROVED	DS

PROJECT			
HIGHWAY 11 / 12 (OLD BARRIE ROAD) STORMWATER MANAGEMENT PONDS A AND B			
TITLE			
Plasticity Chart - SILT (ML)			
PROJECT NO.	CONTROL	REV.	FIGURE
19135676	0	0	B-8

APPENDIX C

Global Stability Figures

GLOBAL STABILITY ANALYSIS - POND A

Drained Analysis

Figure C-1

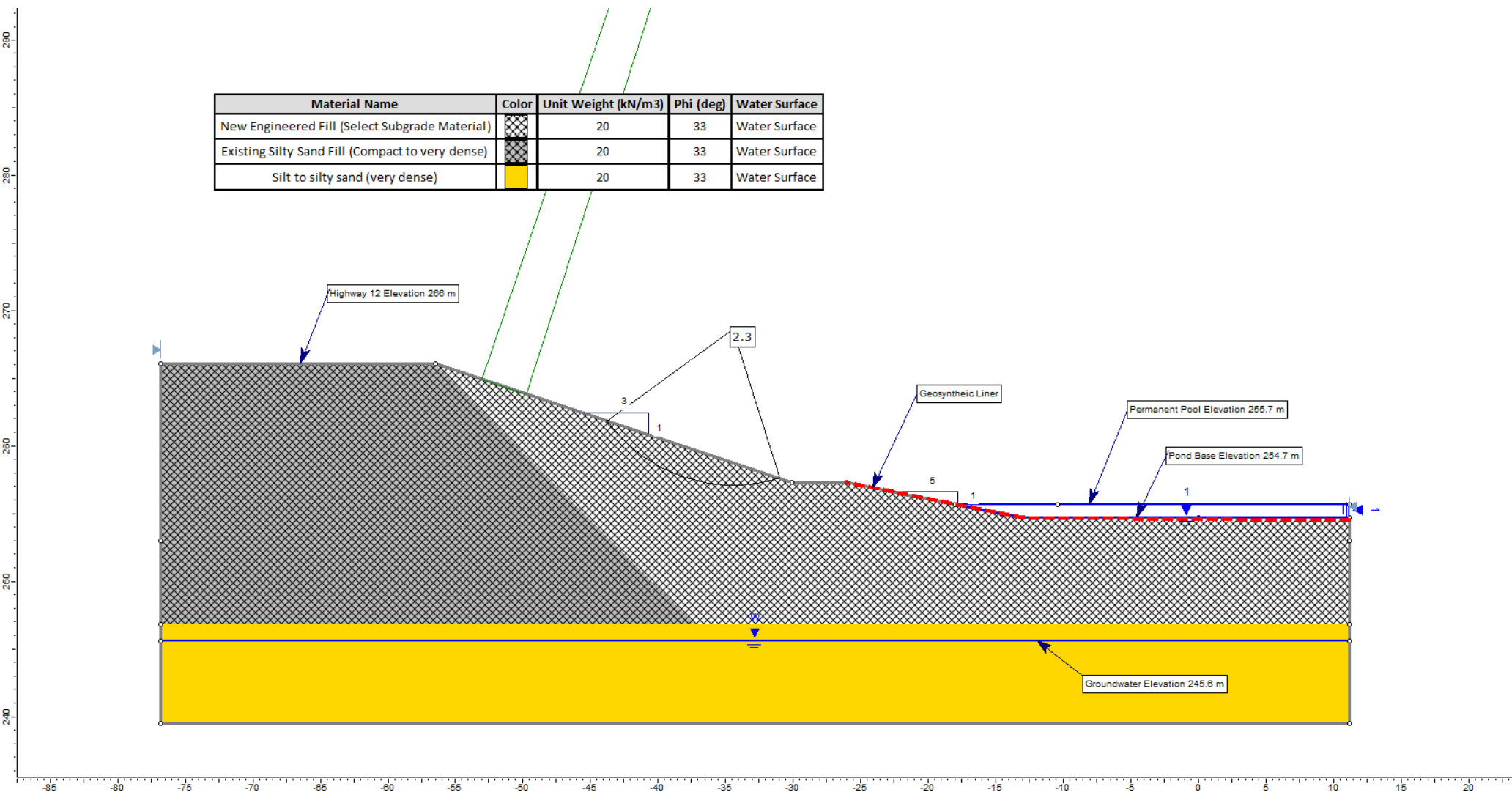


Figure shows all failure surface with Factor of Safety <1.5

GLOBAL STABILITY ANALYSIS - POND B
Drained Analysis

Figure C-2

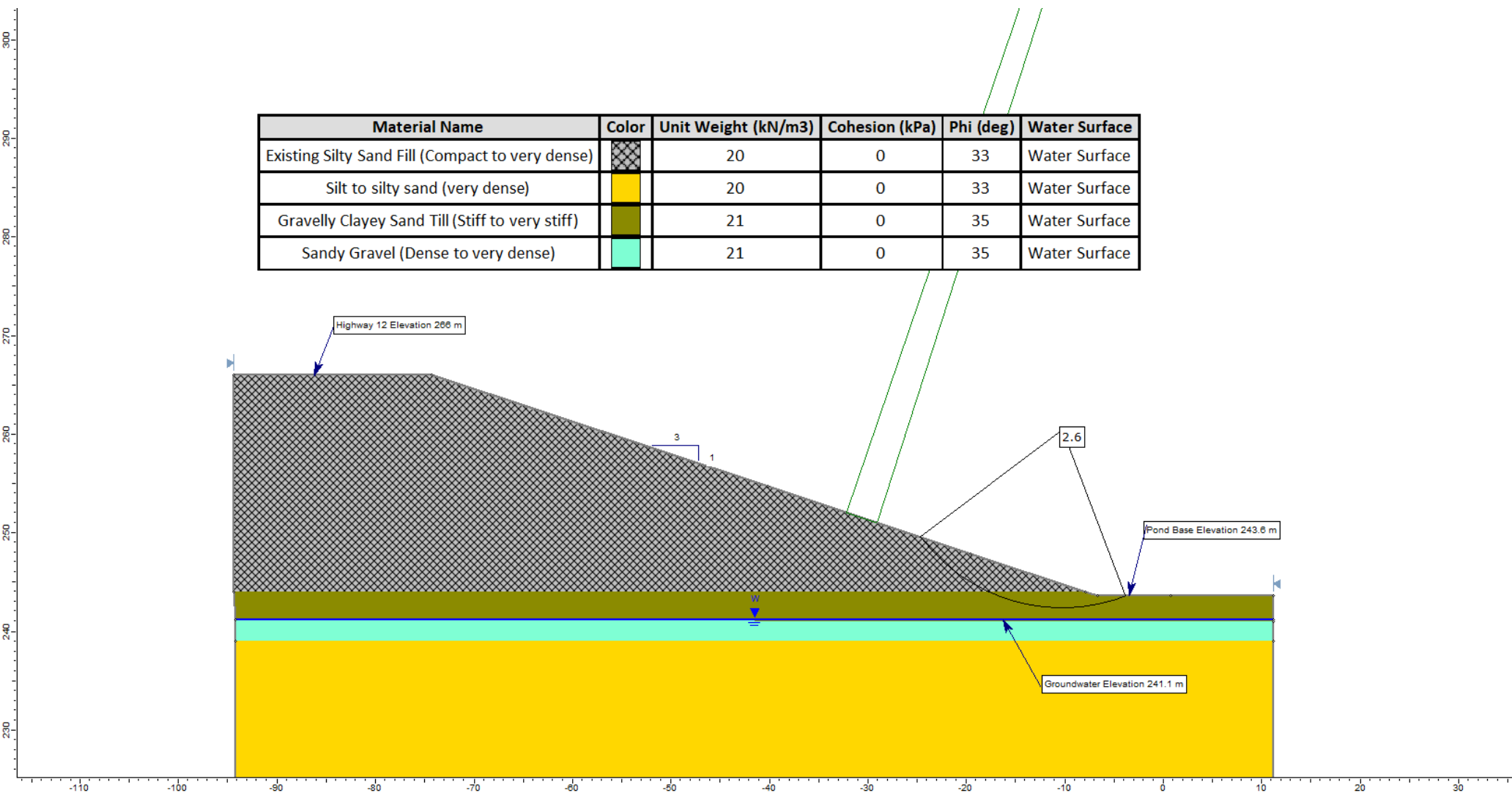


Figure shows all failure surfaces with Factor of Safety <1.5

APPENDIX D

**Non-Standard Special Provisions /
Special Provisions**

GEOSYNTHETIC CLAY LINER

Special Provision

November 2020

1.0 SCOPE

This specification describes the requirements for the manufacturing, supply and installation of a reinforced Geosynthetic Clay Liner (GCL) in conjunction with the required excavation and fill placement as detailed in the Contract Documents.

2.0 REFERENCES

American Society for Testing and Materials (ASTM)

- ASTM D 4632, Standard Test Method for Grab breaking Load and Elongation of Geotextiles
- ASTM D 4643, Determination of Moisture Content of Soil by the Microwave Oven Method
- ASTM D 5261, Test Method for Measuring Mass per Unit Area of Geotextiles
- ASTM D 5887, Measurement of Index Flux through Saturated Geosynthetic Clay Liner Specimens Using a Flexible Wall Permeameter
- ASTM D 5890, Standard Test Method for Swell Index of Clay Mineral Component of Geosynthetic Clay Liners
- ASTM D 5891, Standard Test Method for Fluid Loss of Clay Component of Geosynthetic Clay Liners
- ASTM D 5993, Standard Test Method for Measuring Bentonite Mass per Unit Area of Geosynthetic Clay Liners
- ASTM D 6768, Standard Test Method for Tensile Strength of Geosynthetic Clay Liners

3.0 DEFINITIONS – N/A

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Submission Requirements

4.01.01 Working Drawings

At least three (3) weeks prior to the use of the geosynthetic clay liner, the Contractor shall submit to the Contract Administrator six (6) copies of the Working Drawings and a method statement signed and sealed by the design Engineer and design-check Engineer.

4.01.02 Quality Test Certificates

Prior to installation of the geosynthetic clay liner, the Contractor shall submit quality test certification for each production lot supplied from a laboratory accredited by the Standards Council of Canada. The quality test certificates shall demonstrate compliance with all requirements of this special provision.

4.01.03 Delivery, Storage, Handling, and Protection Procedure

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the geosynthetic clay liner manufacturer's requirement.

A Manufacturer's Certificate of Conformance and a separate report shall be submitted to the Contract Administrator at least three (3) weeks prior to the delivery of each geosynthetic clay liner.

This report shall include the following information:

1. Name of the manufacturer
2. Product name
3. Roll numbers and identification
4. Mill test data
5. Sampling procedures and frequency
6. Results of quality control tests including description of test methods

Upon request, documentation describing the manufacturer's Quality Control program shall be made available to the Contract Administrator.

The delivery of each geosynthetic clay liner, shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

4.02 As-Built Drawings

As-built drawings shall be submitted to the Contract Administrator in a reproducible format prior to Contract Completion.

The as-built drawings shall be dated and bear the seal and signature of the design check Engineer and design Engineer

5.0 MATERIALS

The geosynthetic clay liner shall meet the requirements of Table 1.

The geotextile components shall be non-woven, needle punched and woven polypropylene or polyester material with Typical and Minimum Average Roll Values (MARV) meeting or exceeding the criteria specified Table 1.

The bentonite shall consist of montmorillonite (sodium bentonite).

The geosynthetic clay liner product shall retain their structure during handling, placement and long-term service.

The geosynthetic clay liner shall be resistant to acid and alkali action, micro-organisms and insects and ultra violet degradation.

The geosynthetic clay liner shall be supplied in rolls of minimum 4.5 metre continuous width.

The minimum roll length shall be equal to the Manufacturer's standard minimum length.

During shipping and on-site storage, the geosynthetic clay liner shall be protected against exposure from sun; moisture, contamination by mud, dust, dirt; puncture; tearing and any other damaging or deleterious conditions.

Each geosynthetic clay liner roll shall have waterproof labels in two separate locations, which contains the following information:

- Manufacturer's name,
- Production Identification,
- Lot Number,
- Roll Number,
- Roll Weight, and
- Roll Dimensions.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 General

The geosynthetic clay liner shall be installed and covered as specified in the Contract Documents.

7.01.01 Placement of the Geosynthetic Clay Liner

The placement of the geosynthetic clay liner (GCL) shall be undertaken under the supervision of the Contractor's Engineer.

The manufacturer's representative shall be on site to oversee installation of the geosynthetic clay liner at the commencement of the installation.

7.02 Delivery, Storage and Handling

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall handle the geosynthetic clay liner in such a manner as to avoid damage.

The Contractor shall protect the geosynthetic clay liner from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

7.03 Foundation Excavation / Dewatering

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the pond base / side slope elevation shall be sub-excavated and replaced with Granular A or Granular B Type I or II, or SSM material meeting the requirements of OPSS.PROV 1010.

The Contractor shall maintain the subgrade surface in suitable condition in accordance with OPSS.PROV 206 throughout the installation period.

Where required, a dewatering and/or temporary flow passage system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the pond base / side slopes, permit the replacement of any sub-excavated soils, placement of levelling pad / geosynthetic clay liner / cover in the dry. The groundwater level should be drawn down to at least 1 m below the base of the excavation over the entire pond footprint. Working Drawings for the dewatering system to be prepared by a specialist dewatering company shall be in accordance with OPSS.PROV 517 as amended by Special Provision 517F01. The dewatering system shall be constructed and operated according to the Working Drawings. Unwatering shall be carried out as necessary.

Activation and deactivation of the dewatering and temporary flow passage system, if applicable, shall be in accordance with OPSS.PROV 517.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be in accordance with OPSS.PROV 805. Measures not specified in OPSS.PROV 805 shall be carried out according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

7.04 Installation of Geosynthetic Clay Liner

The Contractor shall:

1. Install the geosynthetic clay liner as indicated by the manufacturer and as detailed on the Contract Documents.
2. Place panels from the lowest elevation towards the highest elevation.
3. Overlap all geosynthetic clay liner panels. Along the width of the mat, the overlap of side joints shall be a minimum of 300 mm, or as specified by the manufacturer. The edges of the geosynthetic clay liner panels should be adjusted to smooth out any wrinkles or creases, to maximize contact with the underlying panel.
4. Remove any soil or other deleterious material present in the overlap zone.
5. Place or pour a fillet of bentonite or other sealing material recommended by the manufacturer and acceptable to the Contract Administrator, in a continuous manner along the overlap zone at a rate of at least 1800 grams per lineal metre (0.25 pounds per lineal foot) to seal the overlaps. The bentonite used in the overlap areas shall meet the specifications for the bentonite used in manufacture of the geosynthetic clay liner as specified.
6. Cut the geosynthetic clay liner using a utility blade in a manner recommended by the manufacturer and exercise due care to prevent damage to any underlying or adjacent liner system components during cutting.
7. Replace or properly repair any geosynthetic clay liner damaged by stones or other foreign objects, or installation activities.
8. Repair any holes or tears in the geosynthetic clay liner by placing a geosynthetic clay liner patch over the hole, overlapping the edges of the hole or tear by at least 600 mm in all directions.

Bentonite shall be applied between the geosynthetic clay liner and the patch in the overlap area, as per the manufacturer's specifications. Patches shall not be nailed or stapled.

9. Remove any soil or other material which may have penetrated the torn geosynthetic clay liner.
10. Place only the amount of geosynthetic clay liner which can be covered with earth material within the same day.
11. Install the geosynthetic clay liner in a way that reduces the potential for hydration of the mat prior to completion of construction of the overlying cover soil.
12. Remove the geosynthetic clay liner and replace with new material if it becomes hydrated before the overlying earth material is placed.
13. In the presence of wind, sufficiently weight all geosynthetic clay liner with sandbags or the equivalent. Install such sandbags during placement and maintain in place until replaced with cover material.
14. Provide temporary cover on the geosynthetic clay liner as necessary to prevent movement of the geosynthetic clay liner both in storage and as placed due to windy conditions
15. Geosynthetic clay liner shall not be installed in standing water, snow or ice.
16. Geosynthetic clay liner shall not be installed during precipitation, high winds or other conditions that may cause rapid hydration of or damage to the geosynthetic clay liner.
17. The top of the geosynthetic clay liner should be anchored in place by embedding it within a shallow trench, approximately 450 mm in width and 600 mm depth, that is backfilled with compacted soil.
18. The geosynthetic liner should be provided with a minimum of 0.3 m thick soil cover.
19. The soil cover should consist of Select Subgrade Material (SSM), with maximum particle size of 50 mm.
20. Cover material shall be installed in such a manner that equipment does not drive directly on the liner material. The 0.3 m soil cover directly above the liner should be placed using a Low Ground Pressure (LGP) dozer, with a maximum track pressure of 35 kPa.
21. If it is anticipated that pond cleaning will be carried out using mechanical dredging techniques, an additional 0.3 m thick layer of Granular 'B' Type II should be placed above the soil cover. The Granular 'B' Type II should be placed and track-packed using a dozer.

7.05 Permanent Cover Materials

A Request to Proceed shall be submitted to the Contract Administrator prior to the placement of any permanent cover materials.

Permanent cover materials shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT

Measurement is by Plan Quantity, as may be revised by Adjusted Plan Quantity, in square metres following the contours of the subgrade.

10.0 BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment, and material to do the work.

TABLE 1 MINIMUM REQUIRED PROPERTIES		
Property	ASTM Test Method	Specified Value
<u>Geotextiles</u>		
Upper Non-Woven Cover:		
Mass/Unit Area	D 5261	200 g/m ² MARV
Lower Woven Carrier:		
Mass/Unit Area	D 5261	105 g/m ² MARV
<u>Bentonite</u>		
Swell Index	D 5890	Minimum 20 ml/2 g
Moisture Content	D 4643	Maximum 12 %
Fluid Loss	D 5891	Maximum 20 ml
Material (sodium bentonite)	XRD	90% montmorillonite
<u>GCL Product</u>		
Bentonite Mass/Unit Area	D 5993	Minimum 3,600 g/m ²
Tensile Strength	D 6768	Minimum 5kN/m MARV
Grab Tensile Strength *	D 4632	93N min
Index Flux	D 5887	Maximum 1x10 ⁻⁸ m ³ /m ² /s
Permeability	D 5887	Maximum 5x10 ⁻⁹ cm/s

*Modified ASTM D 4632 to use a 100 mm wide grip.

TABLE 2 REQUIRED PRE-SHIPING TESTING OF GCL		
Property	ASTM Test Method	Specified Frequency
Geotextile Mass/Unit Area	D 5261	1 per 4,000 m ²
Swell Index; Fluid Loss	D 5890, D 5891	1 per shipment or per 50,000 kg max.
Moisture Content	D 4643	1 per 4,000 m ²
Bentonite Mass/Unit Area	D 5993	1 per 4,000 m ²
GCL Tensile	D 6768	1 per 10,000 m ²
GCL Grab Tensile	D 4632	1 per 10,000 m ²

GCL Index Flux	D 5887	1 per 10,000 m ²
Permeability	D 5887	1 per 10,000 m ²

DEWATERING SYSTEM - Item No.
TEMPORARY FLOW PASSAGE SYSTEM - Item No.

Special Provision No. 517F01

July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

Section 517.01 of OPSS 517 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the design, operation, and removal of a dewatering or temporary flow passage system or both to control water during construction, and the control of the water prior to discharge to the natural environment and sewer systems.

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Subsection 517.04.01 of OPSS 517 is amended by deleting the first paragraph in its entirety and replacing it with the following:

A dewatering or temporary flow passage system or both shall be designed to control water at the locations specified in the Contract Documents and at any other location where a system is necessary to complete the work. The design of the system shall be sufficient to permit the work at each location to be carried out as specified in the Contract Documents.

Subsection 517.04.01 of OPSS 517 is further amended by deleting the second last paragraph in its entirety and replacing it with the following:

Temporary flow passage systems shall be designed, as a minimum, for a 2 year design storm return period and groundwater discharge, except for the work specified in Table A. For the work specified in Table A, the temporary flow passage system shall be designed, as a minimum, for the design storm return period specified in Table A and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

Table A

IDF Curve Location	Latitude: 43 57' 15" N		Longitude: 79 34' 14" W			
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m³/s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
Pond B	5					Yes
Dewatering Systems						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)					Design Engineer Requirements (Note 1)
Pond B	N/A					Yes
Note:						
1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.						
2. “N/A” indicates a preconstruction survey is not required.						

NOTES TO DESIGNER:

Designer Fill-in for Table A:

- | | |
|------|--|
| * | Enter the latitude and longitude co-ordinates of the IDF Curve as obtained using the MTO IDF Curve Look up Tool. Create additional tables, as necessary, if more than one (1) IDF curve was used on the contract (i.e. on a very long contract there may be two IDF curves used to better represent rainfall events for two (2) different sections of the contract). |
| ** | Fill-in site name, work, and station reference as appropriate for the dewatering system and/or temporary flow passage system item locations. |
| *** | For temporary flow passage system item locations, fill-in the minimum design storm return period for the site based on MTO Drainage Design Standard TW-1. |
| **** | For temporary flow passage system item locations, fill-in the design flow rate estimates for the various return periods. |

***** Insert “Yes” when recommended by the Foundation Engineer. Insert “No” otherwise.

***** Fill-in the required distance for preconstruction survey if recommended by the Foundation Engineer. Fill-in “N/A” if not recommended.

Table A (Sample)

IDF Curve Location	Latitude: 44.974844		Longitude: -79.769339			
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m³/s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
Woods Creek Culvert Rehabilitation	2	0.7	3.5	7.5	10.9	N/A
Site 32-145 Robbs Creek Culvert Replacement	10	1.6	7.6	17.4	25.2	Yes
Dewatering Systems						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)					Design Engineer Requirements (Note 1)
Site 32-145 Robbs Creek Culvert Replacement	300					Yes
Note:						
1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.						
2. “N/A” indicates a preconstruction survey is not required.						

WARRANT: Always with these tender items.



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