Foundation Investigation and Design Report

HIGHWAY 17 WIDENING
AREA 1 (STA. 10+400 – 10+600), AREA 2 (STA. 11+050 – 11+160)
AREA 3 (STA. 11+525 – 11+600), AREA 4 (STA. 12+350 – 12+480)
TOWNSHIP OF DAMBROSSIO
AREA 7 (STA. 16+700 – 16+825)
TOWNSHIP OF LALIBERT
DISTRICT OF ALGOMA
BETWEEN WHITE RIVER AND WAWA, ONTARIO

AGREEMENT NO. 5017-E-0043
WORK ITEM: #5
GWP 5031-18-00

Distribution:

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GEOCRES No.: 42C-047

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February 3, 2020

Part A - Foundation Investigation Report

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District of Algoma, Between White River and Wawa, Ontario
Agreement No 5017-E-0043, Work Item: #5
G.W.P 5031-18-00

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1. Introduction

The Ministry of Transportation of Ontario Northeast Region (Ministry or MTO), plans to temporarily and permanently widen Highway 17 in Northern Ontario, and retained GHD Limited (GHD) to conduct foundation investigations at Ministry selected five (5) high fill areas. The five (5) high fill areas (Area 1, Area 2, Area 3, Area 4 and Area 7) involve local creek and lake crossings, and are located between the Towns of White River and Wawa, in the Townships of Dambrossio and Lalibert, District of Algoma, Ontario. Drawing 1 in Enclosure 1, present a site plan showing the location of these areas.

The purpose of the foundation investigations was to describe the subsurface and groundwater conditions based on borehole information and laboratory test results, and provide geotechnical recommendations for the design of embankments for temporarily and permanent road widening.

The work was conducted under a retainer agreement (Agreement No. 5017-E-0043), Work Item # 5. GHD submitted the proposal for this assignment based on the work item order received from the Ministry. The original request from the Ministry included seven (7) areas along Highway 17. However, Area 5 and Area 6 located between Area 4 and Area 7 were later removed from the scope by the Ministry. Authorization to proceed with the foundation investigation for the rest of the five areas, was obtained from Mr. Mark Winmill (Ministry's Project Manager) in an email dated August 23, 2019.

This project didn't include an environmental assessment, and the information contained herein must in no way be construed as an opinion of the environmental conditions of the project sites.

2. Site Conditions

Area 1 is located in the Township of Dambrossio, approximately 45 km north of the Town of Wawa. It represents a road embankment that stretches from Sta. 10+400 to Sta. 10+600. A permanent road widening of up to 5 m on the south side of Highway 17 was originally planned in this area. A creek flows through a corrugated steel pipe (CSP) culvert of about 1.5 m wide located at Sta. 10+422. Another CSP culvert aligned in south-north direction also exists at around Sta. 10+500. The general direction of flow in the region is from north to south, but the creeks in this area drain towards a local pond north of the highway. According to the survey data obtained from the Ministry in an email dated July 18, 2019, the existing embankment has a thickness of up to 5.5 m. The embankment fill consists of blasted rock derived from the rock cuts present on the north and south sides of the highway.

Area 2 is situated about 600 m south of Area 1, and involves a road embankment located in the Township of Dambrossio, between Sta. 11+050 and Sta. 11+600. A permanent road widening of up to 5 m on the south side of Highway 17 was originally planned in this area. Based on the survey data from the Ministry, the height of the existing embankment was reported to be 7.5 m. The creek at the middle of the road embankment, flows from northeast to southwest through a CSP culvert of about 1.5 m in width, and empty into a local lake called Lonely Lake. Bedrock outcrops were observed near the creek on the west side of the highway. Bedrock exposures are also present at the west and east ends of the embankment. However, much of the area on the west side was swampy and muddy.

Area 3 is located 500 m south of Area 2 and covers the road section from Sta. 11+525 to Sta. 11+600 in the Township of Dambrossio. The original plan indicate that the road widening in this area will be on the south side of Highway 17 and may reach a width of 5 m. According to the survey information obtained from the Ministry, the existing road embankment consists of up to 5.5 m thick blasted rock fill. A CSP culvert associated with this area is aligned in northeast to southwest direction. This culvert is about 1.5 m wide and is located in the middle of the embankment (at around Sta. 11+560). During the fieldwork, standing water was observed on the west side of the road embankment. Bedrock outcrops were observed on the northwest, southwest and southeast sides of the highway.

Area 4 refers to a lake crossing located in the Township of Dambrossio, between Sta. 12+350 and Sta. 12+480, approximately 1.7 km south of Area 3, or 2.5 km north of the intersection between Highway 17 and Highway 519. In this area, the road embankment was built by placing blasted rocks over a local lake called Desolation Lake. The thickness of the road embankment is about 5.3 m. A CSP culvert exists on the east side of the lake at Sta. 12+412, conveying water from north to south. The original plan in this area involved a road widening on the north side of the highway. Bedrock exposures exist on the west and east sides of the lake. The lake level during the fieldwork in early October was at El. 450. 8 m, and the water depth was 0.8 m at borehole locations.

Area 7 is located roughly 35 km north of the Town of Wawa, in the Township of Lalibert, between Sta. 16+700 and Sta. 16+825. In this area, permanent road widening of up to 5 m on the north side of Highway 17 was originally planned. Based on the Ministry's survey data, the road embankment is up to 14 m high and consists of blasted rocks. A creek was observed at the eastern end of the embankment flowing from southwest to northeast through a CSP culvert of about 2 m in diameter.

Highway 17 is a well-travelled route of the Trans-Canada Highway in Northern Ontario. Throughout the region, the highway passes through rugged topography with numerous road cuts and high fill embankments. Between White River and Wawa, the highway consists of two (2) travel lanes and about 2.5 m wide gravel shoulders. In Area 1, Area 2, Area 3 and Area 7, the vertical geometry of the highway is relatively flat with minor sags and crests. Local relief in these areas is less than 3 m. In Area 4, the highway makes a curve around the middle of Desolation Lake and is characterized by a higher relief. In all areas, the highway has posted speed limit of 90km/hr.

Apart from Highway 17, access to and from the region is serviced by secondary highways such as Highway 519, and local fishing and logging roads. In addition, infrequent flights to major surrounding cities can be obtained from a small airport located in the southern part of Wawa.

The land north and south of the project areas, beyond the limits of Highway 17, consists of a natural woodland and diverse landscapes marked by rocky uplands and depressions. A significant portion of the region including the project areas is occupied by a number of north-south trending lakes, ponds, swamps and wetlands. The percentage of rock outcrops is very significant as numerous rock cuts were observed along the highway. Vegetation is characterized mainly by deciduous forest consisting of white birch and spruce trees. Shrubs as well as tall grasses are also common near road embankments and on wet areas. The climate is characterized by a long cold winter and a warm summer. A freezing rain and a light snow fall has occurred during the fieldwork in early October.

Mining and mineral exploration has been a common activity throughout the region, and existing reports indicated that the area around Desolation Lake (Area 4) was once explored for gold and iron ores. However, no known quarries were observed in this area during the fieldwork.

Photo 1 to Photo 18 in Appendix A show the general site conditions and field activities.

3. Investigation Procedures

3.1 Field Investigation

The field program consisted of advancing a total of eighteen (18) boreholes and five (5) auger holes in all areas, and was conducted between September 03, 2019 and October 03, 2019. Prior to the commencement of drilling, MTO road occupancy permit were obtained and borehole locations were cleared of utilities. Access was prepared in all areas to open up the lands for foundation investigation. The access preparation involved the cutting of trees and removal of blast rocks as well as the placement of gravel on muddy and swampy grounds. Attempts were carried out to drill boreholes at or a little away from the toes of existing embankments. However, some boreholes were repositioned as necessary in consideration of surface features, the presence of communication and hydro lines, standing water and extremely wet grounds, and steep natural slopes. Boreholes through the road embankment were completed at the center of the right or left lane of the highway.

A summary of the field investigation program for each area investigated, including elevations, depths and remarks is given in Table 3.1. The logs of all boreholes are provided in Appendix B.

In Area 1, the foundation investigation involved drilling four (4) boreholes (BH-19-1-01, BH-19-1-02, BH-19-1-03, and BH-19-1-04), along the west toe of the existing embankment. The boreholes were drilled to the depth of 3.7 m to 6.8 m. To confirm the presence and depth of the bedrock, rock coring was conducted in two (BH-19-1-02 and BH-19-1-04) of these boreholes. In addition, two (2) boreholes (BH-19-1-05 and BH-19-1-06) were advanced through the left lane of the exiting road embankment. Boreholes through the embankment were drilled to the depth of 2.8 m to 5.9 m, with rock coring carried out in BH-19-1-05. Drawing 2 in Enclosure 1 presents the borehole location plan for Area 1.

In Area 2, the foundation investigation consisted of three (3) boreholes (BH-19-2-01, BH-19-2-02, and BH-19-2-03) advanced along the west toe of the existing embankment. The boreholes were drilled to the depth of 3.7 m to 5.3 m. Rock coring was initially required in one of the boreholes. However, because of the presence of shallow bedrock, rock coring was conducted in two of the boreholes (BH-19-2-01 and BH-19-2-02). No boreholes were required through the embankment. Drawing 3 in Enclosure 1 provides the location of the three (3) boreholes in the area.

The foundation investigation in Area 3 involved drilling three (3) boreholes (BH-19-3-01, BH-19-3-02, and BH-19-3-03) along the west toe of the existing embankment. The boreholes were drilled to the depth in the range of 2.5 m to 5.7 m, with rock coring conducted in BH-19-3-02. In addition, two (2) boreholes (BH-19-3-04 and BH-19-3-05) were drilled through the existing embankment to the depth of 5.9 m to 9.1 m. Drawing 4 in Enclosure 1 presents the borehole location plan for Area 3.

Table 3.1 – Field Investigation Program

Location	Sta. No	Borehole/Hand Augerhole No.	Drilling Location	Elevation[1] (m)	Depth[2] (m)	Remark
Area 1	10+400 – 10+600	BH-19-1-01	Off road	435.3	3.7	Terminated on Probable Bedrock
		BH-19-1-02		436.2	8.3	Terminated in Bedrock after Coring
		BH-19-1-03		437.2	7.9	Terminated due to Auger Refusal
		BH-19-1-04		437.7	6.8	Terminated in Bedrock after Coring
		BH-19-1-05	Embankment	441.3	5.9	Terminated in Bedrock after Coring
		BH-19-1-06		443.5	2.8	Terminated on Probable Bedrock
Area 2	11+050 – 11+160	BH-19-2-01	Off road	447.3	3.7	Terminated in Bedrock after Coring
		BH-19-2-02		446.3	5.0	Terminated in Bedrock after Coring
		BH-19-2-03		446.6	5.3	Terminated on Probable Bedrock
Area 3	11+525 – 11+600	BH-19-3-01	Off road	460.2	5.0	Terminated on Probable Bedrock
		BH-19-3-02		459.6	5.7	Terminated in Bedrock after Coring
		BH-19-3-03		460.1	2.5	Terminated on Probable Bedrock
		BH-19-3-04	Embankment	464.3	9.1	Terminated in Bedrock after Coring
		BH-19-3-05		464.6	5.9	Terminated in Probable Bedrock
Area 4	12+350 – 12+480	BH-19-4-01	In the Lake	451.3	1.8	Terminated on Probable Bedrock
		BH-19-4-02		451.3	10.8	Terminated in Bedrock after Coring
		BH-19-4-03	Off road	452.1	2.5	Terminated on Probable Bedrock
		BH-19-4-04	Embankment	455.6	13.1	Terminated due to Auger Refusal
Area 7	16+700 – 16+825	HA-19-7-01	Off road	424.0	0.8	Terminated due to Auger Refusal on Probable Bedrock
		HA-19-7-02		421.5	0.7	
		HA-19-7-03		420.5	1.9	
		HA-19-7-04		421.6	0.9	
		HA-19-7-05		420.6	0.7	

<u>Note</u> 1: Surface or Drilling Platform Elevation; 2: Depth Below Grade (m) or Drilling Platform

In Area 4, three (3) boreholes (BH-19-4-01, BH-19-4-02, and BH-19-4-03) were advanced along the east toe of the embankment, to a depth of 1.8 m to 10.8 m. Two (2) of these boreholes (BH-19-4-01 and BH-19-4-02) were completed in the lake, with rock coring conducted in BH-19-4-02. In addition, the field program involved the completion of one (1) borehole (BH-19-4-04) at the embankment to a depth of 13.1 m. Drawing 5 in Enclosure 1 presents the location of the boreholes in Area 4.

In Area 7, a total of five (5) hand augerholes (HA-19-7-01, HA-19-7-02, HA-19-7-03, HA-19-04, and HA-19-7-05) were completed along the east toe of the existing embankment, with the purpose of confirming the bedrock surface. The depth of these hand augerholes ranged from 0.8 m to 1.9 m. Drawing 6 in Enclosure 1 present the location plan for the hand augerholes.

Drilling started in Area 1 using an Acker Soil Sentry track-mounted drilling rig equipped with hollow stem augers. Rock coring was carried out using swivel-type double tube core barrels. The drilling equipment was owned and operated by a specialist drilling contractor known as Tatry Environmental Drilling (Tatry) of Timmins, Ontario. In the time between September 03, 2019 and September 13, 2019, Tatry completed four (4) boreholes in Area 1, and was later replaced by Marathon Underground Constructors Corp. (Marathon) of Greely, Ontario. The remaining boreholes in Area 1 and all boreholes in Area 2, Area 3 and Area 4 were completed by Marathon using CME 55 track mounted drilling rig equipped with hollow stem augers and capable of rock coring.

In Area 4, drilling in the lake was conducted by Fordia Explo Limited Access portable rig mounted on a boat. The rig is designed for shallow depth geotechnical investigation in areas where access is limited, and can be used for auger, rotary, or percussive drilling and rock coring. The boat was placed in the lake with the support of an excavator and tied to trees to obtain balance against current. A sediment curtain (silt fence) was used to prevent the release of silt and drill cuttings into the lake.

All access preparation, track and barge-mounted drilling, soil and rock sampling, and backfilling of boreholes was conducted under the full time supervision of a GHD field technician.

During drilling, representative soils samples were obtained at selected intervals from all boreholes, using a 50 mm outer-diameter (OD) split-barrel sampler advanced in accordance with the Standard Penetration Test (SPT) procedures described in the ASTM D1586. The drill rigs were equipped with 63.5 kg cathead automatic hammer calibrated to fall freely through 760 mm. The results of the SPTs are reported as “N” values on borehole logs. In addition to soils samples, NQ size (47.6 mm) rock cores were obtained from boreholes completed in Area 1, Area 2, Area 3 and Area 4.

Soil samples obtained from boreholes were inspected in the field immediately upon retrieval for type, texture, and color. All samples were sealed in plastic jars and transported to the GHD laboratory in Whitby and Waterloo for further visual examination and geotechnical laboratory tests.

Rock cores were described based on color, hardness, degree of fracturing, weathering, core recovery (REC), rock quality designation (RQD) and were placed in core boxes for additional description and laboratory testing. The core boxes were labeled with project name, area or location of drilling, borehole number, core run lengths and depth, date of coring, and name of drilling contractor.

The groundwater condition at borehole locations were observed and recorded during drilling by visual examination of soil samples, sampler and drill rods. Upon completion of drilling, groundwater levels were measured in open boreholes. In addition, a total of four (4) monitoring wells were installed in Area 1 (BH-19-1-01), Area 2 (BH-19-2-03), Area 3 (BH-19-3-03) and Area 4 (BH-19-4-03) to conduct short term static water level measurements. The monitoring wells were 50 mm PVC standpipes with 1.5 m screen, and were backfilled in the upper part with sand and bentonite.

Upon completion of drilling, all boreholes drilled on land were backfilled with a mixture of auger cuttings, and sealed at the top with bentonite in accordance with the MTO guidelines and Ministry of Environment, Conservation and Parks (MOECP) O.Reg. 903 for borehole abandonment. The boreholes in the lake were grouted with cement and sealed with bentonite to a depth of about 1 m.

Surveying of the as-drilled borehole locations was conducted by Callon Dietz Inc. (Callon Dietz) of Sudbury, Ontario under contract to GHD. In the lake in Area 4, surveying of boreholes was carried out during drilling, once the boat was tied and stabilized. Callon Dietz provided coordinates in MTM NAD 83 Northing and Easting. The coordinates shown on borehole logs and any part of this report correspond to these northings and eastings. All elevations given in this report are geodetic.

3.2 Laboratory Program

Geotechnical laboratory testing was performed at GHD's laboratory located in Whitby, Ontario. The laboratory program consisted of moisture content, Atterberg Limits and grain size analyses. For this assignment, moisture content tests were carried out on all samples collected from all boreholes. Atterberg Limits tests were conducted on two (2) samples, and gradation analyses (sieve and hydrometer) were performed on thirty two (32) representative soil samples.

Table 3.2 provides a summary of the laboratory program carried out for this project.

All routine laboratory tests were conducted in accordance with MTO procedures, which follow American Society for Testing Materials (ASTM) standards. The results of the moisture content, Atterberg Limits tests and gradation analyses are provided on the corresponding borehole logs provided in Appendix B. The results of the Atterberg Limits tests and gradation analyses are also presented graphically on plasticity charts and gradation curves given in Appendix C.

In addition to routine soil laboratory tests, a total of three (3) unconfined compressive strength (UCS) tests were conducted on rock cores collected from boreholes drilled in Area 1, Area 3 and Area 4. The UCS test results are provided on corresponding borehole logs and also given in Appendix D.

Table 3.2 - Laboratory Analysis Program

Location	Borehole No.	Sample No.	Depth[1] (m)	Type of Laboratory Test			
Area 1	BH-19-1-01	SS3	1.5 - 2.1			Hydrometer	
	BH -19-1-01	SS4	2.3 – 3.1		Sieve		
	BH-19-1-02	SS6	4.5 – 5.0			Hydrometer	
	BH -19-1-03	SS8	5.4 - 5.9		Sieve		
	BH -19-1-03	SS9	6.1 - 6.7		Sieve		
	BH -19-1-03	SS10	7.7 - 7.9		Sieve		
	BH -19-1-04	SS4	2.3 - 2.9		Sieve		
	BH -19-1-04	SS5	3 - 3.7		Sieve		
	BH-19-1-05	SS1	0.2 - 0.8			Hydrometer	
	BH -19-1-06	SS1	0.2 – 0.8			Hydrometer	
	BH-19-1-06	SS2	0.8 - 1.4			Hydrometer	
Area 2	BH -19-2-01	SS1	0 - 0.3		Sieve		
	BH-19-2-02	SS1	0 – 0.6		Sieve		
		SS2	0.6 - 1.1			Hydrometer	
		Rock Core	4.4 – 4.6				UCS
	BH-19-2-03	SS4	3 - 3.7	Atterberg Limit		Hydrometer	
		SS5	3.8 - 4.4			Hydrometer	
Area 3	BH-19-3-01	SS5	3.1 – 3.6	Atterberg Limit			
	BH-19-3-01	SS6	3.8 - 4.4			Hydrometer	
	BH-19-3-02	SS3	1.5 - 2.1			Hydrometer	
	BH-19-3-03	SS2	0.8 - 1.4			Hydrometer	
	BH -19-3-03	SS4	2.3 - 2.5		Sieve		
	BH -19-3-04	SS2	3.8 - 4.4		Sieve		
	BH-19-3-04	SS3	4.6 - 5.2		Sieve		
	BH-19-3-04	Rock Core	5.9 – 6.2				UCS
	BH-19-3-05	SS1	0.2 - 0.8			Hydrometer	
	BH -19-3-05	SS2	0.8 - 1.2		Sieve		
Area 4	BH-19-4-02	SS6	6.6 - 7.2			Hydrometer	
		Rock Core	7.6 – 7.8				UCS
	BH-19-4-03	SS3	1.5 - 2.1			Hydrometer	
		SS4	2.3 - 2.5			Hydrometer	
	BH-19-4-04	SS1	0.2 - 0.8			Hydrometer	
	BH-19-4-04	SS3	7.6 – 8.2		Sieve		
	BH-19-4-04	SS4	9.1 – 9.7		Sieve		
Area 7	BH-19-7-03	SS1	1.1 - 1.4			Hydrometer	
	BH-19-7-05	SS1	0.4 - 0.7			Hydrometer	

Note 1: Depth Below Grade (m) or Drilling Platform

4. Regional Physiography and Geology

Based on publicly available geological reports and maps, the project areas lie in the Superior Province of the Canadian Shield. The Superior Province is divided into various regionally extensive sub-provinces. The project areas are located in the Wawa sub-province. The physiography is represented by the Wawa-Abitibi Terrane. Topography in the region is generally rugged and hilly, and is strongly controlled by the nature of the underlying bedrock. Topographic highs correspond to bedrock outcrops resistant to weathering and erosion. Weak rocks and recent materials gave rise to lower elevations.

The bedrock geology is composed predominantly of irregularly distributed Archean greenstone belts (Michipicoten, Gamitagama, and Mishibishu) surrounded by granitic bodies of various compositions and sizes, with smaller mafic intrusive rocks locally present. The Michipicoten greenstone belt is a structurally and stratigraphically complex assemblage of volcanic, sedimentary and intrusive rocks, metamorphosed to greenschist facies and localized amphibolite facies. Relative to smaller greenstone belts, the grade of metamorphism in the Michipicoten greenstone belt was low.

Most parts of the project areas are rocky and consisted of gently dipping rock outcrops. The rocks are characterized by well-developed white and dark bands of minerals, foliations, and discontinuous quartz veins. Northwest and northeast trending Faults are common in the region. In addition, the metasedimentary and metavolcanic rocks are locally intruded by a series of diabase dykes.

Any existing overburden in the region consists of a thin layer of discontinuous glaciolacustrine deposits and recent organic, alluvial and fluvial materials. Glaciofluvial outwash deposits are confined in topographically low areas along bedrock valleys. In all project areas, the main overburden materials encountered above the bedrocks, were mainly peat and sand and gravel deposits.

5. Subsurface Conditions

Details of the subsurface conditions encountered at the project areas are graphically presented on individual soil strata profiles provided on Drawings 2 to 5 in Enclosure 1, and on the logs in Appendix B. The profiles represent a transition from one soil type to another along the lines of cross sections, and should not be inferred to represent an exact plane of geological or stratigraphic change, as the subsurface and groundwater conditions may vary between and beyond the borehole locations.

The following sections provide descriptions of the major soil strata, and subsurface and groundwater conditions encountered at Area 1, Area 2, Area 3, Area 4 and Area 7.

5.1 Area 1

The subsurface investigation in Area 1 involved drilling four (4) boreholes (BH-19-1-01, BH-19-1-02, BH-19-2-03 and BH-19-2-04) close to the toe of the road embankment. An additional two (2) boreholes (BH-19-1-05 and BH-19-1-06) were also completed through the embankment. In summary, the stratigraphy along the toe of the embankment consisted of 0.2 m to 5.3 m thick peat, underlain by layers of silty sand, sand and gravel, and sandy gravel materials. The thickness of the granular materials beneath the peat ranged from 0.7 m to 2.8 m. Following the granular materials, a greenstone bedrock was encountered. At the road embankment, an asphalt layer was underlain by granular fill of sand and gravel. The fill was underlain by 1.4 m to 2.1 m thick rock fill consisting blasted and smaller size rocks. A bedrock was encountered beneath the rock fill. The following sections provide the descriptions of the soil, rock and groundwater conditions encountered in Area 1.

5.1.1 Peat

A layer of peat was encountered in BH-19-1-01, BH-19-1-02, BH-19-1-03 and BH-19-1-04. Peat is normally formed from the remains of grasses, sedges and bog mosses. The peat was dark brown in color, amorphous and spongy, and wet. In BH-19-1-02, sandy fractions were visible in retrieved samples and the peat was described as amorphous to sandy peat. The thickness of the peat ranged from 0.6 m in BH-19-1-02 to 5.3 m in BH-19-1-03, suggesting the presence of a thicker organic deposit in the southeastern part of the road embankment, extending to El. 431.9 m.

The moisture content of samples of the peat ranged from 170% to 724%. High moisture content values were obtained in samples taken from the lower portion of BH-19-1-03.

5.1.2 Silty Sand

A silty sand layer with varying amounts of silt was encountered below the peat in BH-19-1-01. The silty sand contained some gravel, cobbles and organic matter, was dark brown in color and wet. The thickness of this silty sand was 0.7 m, extending to El. 433.9 m.

The SPT "N"-value within this layer was 34 blows/300 mm, suggesting the presence of "dense" soil.

The moisture content of the silty sand deposit was 25%.

5.1.3 Sand with Gravel /Sand and Gravel/ Sandy Gravel

A layer of sand with gravel was encountered in BH-19-1-01 at a depth of 0.2 m, and extended to the full depth of investigation of 3.7 m (El. 431.6 m). In BH-19-1-02 and BH-19-1-03, the layer was encountered beneath the peat, and was described to be sand and gravel. The gravel proportion increased further in BH-19-1-04 and the layer was described as sandy gravel. In all boreholes, the layer consisted of trace silt, trace clay and cobbles, and was brown to grey in color and wet. The thickness of this layer ranged from 1.4 m in BH-19-1-04 to 3.8 m in BH-19-1-02.

The SPT "N"-values in the sand with gravel /sand and gravel/ sandy gravel layer ranged from 15 blows/300 mm to greater than 100 blows/300 mm, indicating a "compact" to "very dense" state of relative density. In BH-19-1-02, drilling in the upper part of the sand and gravel layer was found to be difficult because of the presence of a higher proportion of gravel and cobbles.

The moisture content tests carried out on samples yielded values ranging from 5% to 35%. High values of moisture content were obtained from samples taken in the upper part of BH-19-1-03.

The sieve analyses of samples taken from BH-19-1-01, BH-19-1-03, and BH-19-1-04 indicated a composition of 23% - 88% gravel, 12% - 68% sand, and 0% - 9% fines (silt and clay). Hydrometer analysis of a sample from BH-19-1-01 resulted in 30% of gravel, 57% of sand, 11% of silt, and 2% of clay. The grain size distribution curves are provided on Fig. No. GSA-1-1 in Appendix C.

5.1.4 Sand with Silt

A sand with silt deposit consisting of trace gravel and trace clay was encountered in BH-19-1-02 beneath the sand and gravel layer. This deposit was grey in color, wet and had a thickness of 0.6 m.

The SPT "N"-value within the sand with silt deposit was documented to be more than 100 blows/300 mm, and the deposit was described to be "very dense".

The moisture content of a sample of the silty sand was 17%.

A grainsize analysis of a sample of the silty sand resulted in a composition of 9% gravel, 66% sand, 23% silt and 2% clay. The grain size distribution curve is given on Fig. No. GSA-1-2 in Appendix C.

5.1.5 Asphalt

A layer of asphalt was encountered in BH-19-1-05 and BH-19-1-06 drilled through the embankment of Highway 17. The asphalt thickness at borehole locations ranged from 200 mm to 230 mm.

5.1.6 Fill

A fill material was encountered beneath the asphalt layer. The fill consisted of sand with gravel to sand and gravel, with trace silt and trace clay. Cobbles and a higher proportion of gravel were also encountered in the upper part of the fill. The fill was brown in color and was dry to moist at the time of drilling. The thickness of the granular fill ranged from 0.6 m to 1.2 m, extending to El. 440.5 m.

The SPT "N" values within the fill were in the range of 18 blows/300 mm to 37 blows/300 mm, indicating a "compact" to "dense" in-situ state of relative density. Because of the higher proportion of gravel, the SPT "N" values in the upper part of the fill were higher than in the lower part.

Moisture content tests conducted on samples of the fill gave values from 4% to 13%. The high moisture content values correspond to the lower part of the fill with an increased amount of silt.

The grainsize analyses of representative samples from BH-19-1-05 and BH-19-1-06 resulted in a composition of 28% - 48% gravel, 48% - 61% sand, 7% - 10% silt, and 1% - 2% clay. The grain size distribution curves are given on Fig. No. GSA-1-3 in Appendix C.

5.1.7 Rock Fill

A rock fill was encountered beneath the granular fill in BH-19-1-05 and BH-19-1-06. Because of difficult drilling conditions and auger grinding, the casing was inserted through the rock fill without sampling and in-situ testing. Generally, field observations indicated that the embankment fill consisted of all sizes of rocks including blasted rocks obtained from the surrounding outcrops. The thickness of the rock fill ranged from 1.4 m in BH-19-1-06 to 2.1 m in BH-19-1-05, extending to El. 438.3 m.

No samples were retrieved from the rock fill for laboratory testing.

5.1.8 Bedrock

Rock outcrops are present both on the southwest and southeast ends of the road embankment. The presence of a bedrock was confirmed through diamond coring in BH-19-1-02 and BH-19-1-04. The bedrock was encountered at a depth of 5.0 m (El. 431.2 m) in BH-19-1-02 and 3.7 m (El. 438.3 m) in

BH-19-1-04, and coring was carried out to a depth of 8.3 m (El. 427.9 m) and 6.8 m (El. 430.9 m), respectively. A bedrock was also encountered in BH-19-1-05 advanced through the road embankment. The depth of the bedrock in this borehole was 2.9 m (El. 438.3 m), and rock coring was carried out for a total length of 3 m, or up to the depth of 5.9 m (El. 435.3 m).

The bedrock encountered in all three boreholes was described to be Greenstone. It was dark green to black in color, fine to medium grained, weakly to moderately foliated, and consisted of quartz and plagioclase crystals in a fine matrix of dark minerals. The rock cores were slightly weathered and moderately to well fractured, and were medium to hard (based on geological hammer and knife test).

Measured Core Recovery (REC) was 88% to 100% in BH-19-1-02, 100% in BH-19-1-04, and 99% to 100% in BH-19-1-05. The corresponding Rock Quality Designation (RQD) ranged from 51% to 85% in BH-19-1-02, 88% to 96% in BH-19-1-04, and 72% to 73% in BH-19-1-05. Based on the RQD values, the rock mass quality was described to be “fair” to “excellent”.

Photo 19 to Photo 21 given in Appendix D, show the rock cores from BH-19-1-02, BH-19-1-04 and BH-19-1-05. The corresponding rock core descriptions are provided in Table D1 in Appendix D.

5.1.9 Groundwater

Groundwater was encountered in BH-19-1-02, BH-19-1-03 and BH-19-1-04 during drilling. The depth of the groundwater level in these boreholes upon completion of drilling varied from 0.15 m to 0.38 m. The level of groundwater was also measured in a well installed in BH-19-1-01 on October 2, 2019, 27 days after the completion of the borehole. The results indicated a static water level of 1.5 m.

It should be noted that the depth of the groundwater level and its gradient of flow in the area may be influenced by the placement of the road embankment, the surrounding landscape, the rock outcrops, underlying geology, and the water levels in creeks, local lakes and marshy grounds, and may fluctuate because of seasonal changes, periods of precipitation, and temperature. As such, the groundwater level may differ at any time throughout the year and during construction.

Groundwater data is presented on individual borehole logs provided in Appendix B.

5.2 Area 2

The subsurface investigation in Area 2 involved three (3) boreholes (BH-19-2-01, BH-19-2-02, and BH-19-2-03) completed close to the toe of the road embankment. Based on the borehole information, the subsurface in the southwestern part of the road embankment consisted of sand with gravel, and gravelly sand materials with traces of organic topsoil. In the southeastern part of the embankment, a peat was encountered beneath the ground surface followed by layers of silt and silty sand materials. The granular materials are underlain by a greenstone bedrock. The following sections provide the descriptions of soils, rocks and groundwater conditions encountered in Area 2.

5.2.1 Peat

A layer of peat was encountered at the top of the subsurface in BH-19-2-03 drilled in the southeastern part of the embankment. The peat was dark brown in color and wet. In the top part, undecomposed large pieces of wood as well as trace sand and gravel were observed. In the lower part, the peat became amorphous and spongy and was extremely wet. The thickness of the peat was 3 m.

The moisture content of samples of the peat ranged from 47% to 617%. High moisture content values were obtained in samples taken from the lower portion of BH-19-2-03.

5.2.2 Gravelly Sand / Sand with Gravel

A layer of gravelly sand and sand with gravel was encountered immediately beneath the ground surface in BH-19-2-01 and BH-19-2-02. The gravelly sand consisted of some topsoil, trace organic matter, and trace to some fines. In the lower portion of BH-19-2-02, the proportion of gravel increased, and the layer was described as sand and gravel. The gravelly sand was dark brown to brown in color and wet. Its thickness ranged from 0.3 m in BH-19-2-01 to 1.4 m in BH-19-2-02.

The SPT "N" values within the gravelly sand or sand with gravel was in the range of 3 blows/300 m to more than 100 blows/300 m, indicating a "loose" to "very dense" relative density.

Moisture content tests carried out on retrieved samples yielded 43% to 92%.

The sieve analyses indicated a composition of 26% - 28% gravel, 61% - 70% sand, and 4% - 11% fines. The grain size distribution curve is given on Fig. No. GSA-2-1 in Appendix C.

5.2.3 Silt

A layer of silt consisting of trace gravel, trace sand, trace clay, and organic matter was encountered in BH-19-2-03 beneath the peat deposit. The silt was grey in color, wet, and had a thickness of 0.8 m, extending to the depth of 3.8 m (El. 442.8 m).

The SPT "N"-value was 1 blows/300 mm, and the deposit was described to be "very soft".

The moisture content of a sample of the silt was reported to be 91%.

An Atterberg Limit test performed on this silt yielded negligible liquid limit, indicating that the soil is non-plastic. A hydrometer test on the same sample resulted in a composition of 1% gravel, 7% sand, 89% silt and 3% clay. The grain size distribution curve is given on Fig. No. GSA-2-2 in Appendix C.

5.2.4 Sand with Silt

A sand with silt deposit was encountered underneath the silt layer in BH-19-2-03. The silty sand consisted of trace to some gravel, trace clay, and was dark brown in color and wet. The thickness of this silty sand deposit was 1.5 m, extending to the depth of 5.3 m (El. 441.2 m) in BH-19-2-03.

The SPT "N"-values ranged from 12 blows/300 mm to as high as 100 blows/300 mm, suggesting "compact" to "very dense" soil relative density.

The moisture content test of retrieved samples resulted in 18.7% to 28.4%.

The grainsize analysis of a sample taken at a depth of 4 m (El. 442.5 m) from BH-19-2-03, yielded a composition of 10% gravel, 61% sand, 27% silt, and 2% clay. The corresponding grain size distribution curve is presented on Fig. No. GSA-2-3 in Appendix C.

5.2.5 Bedrock

Rock outcrops are present both on the southwest and southeast ends of the road embankment. The existence of a bedrock was confirmed through diamond coring in BH-19-2-01 and BH-19-2-02. The bedrock was encountered at a depth of 0.3 m (El. 447.0 m) in BH-19-2-01 and 1.4 m (El. 444.8 m) in BH-19-2-02. Coring was done for a length of 3.4 m in BH-19-2-01 (to El. 443.7 m). In BH-19-2-02, rock coring extended for a length of 3.6 m or to the depth of 5 m (El. 441.3 m).

The bedrock encountered in the two boreholes was described to be Greenstone, and was the same as the type of rock exposed at road cuts and surface exposures in the area. It was dark green to black in color, fine to medium grained, weakly to moderately foliated, and consisted of quartz and plagioclase crystals in a fine matrix of dark minerals. The rock cores were slightly weathered and moderately to highly fractured, and were medium to hard (based on geological hammer and knife test).

Measured Core Recovery (REC) in BH-19-2-01 ranged from 98% to 100%. In BH-19-2-02, the REC was in the range of 93% to 100%. The Rock Quality Designation (RQD) values ranged from 67% to 81% in BH-19-2-01, and 33% to 90% in BH-19-2-02. Based on the RQD values, the quality of the rock mass near BH-19-2-01 was described to be “fair” to “excellent”, while the quality of the rock mass around BH-19-2-02 was “poor” to “good”. Especially, the core run corresponding to the upper part of the bedrock in BH-19-2-02 was highly fractured, and indicated a “very poor” rock mass quality.

A UCS test was conducted on a rock core taken at a depth of 4.4 m (El. 441.9 m) from BH-19-2-02. The results of the USC test indicated a compressive strength of 199.3 MPa.

Photo 22 and Photo 23 in Appendix D, show the rock cores from BH-19-2-01 and BH-19-2-02. The rock core descriptions (Table D2) and details of the USC test are also given in Appendix D.

5.2.6 Groundwater

Groundwater was encountered in BH-19-2-02 during drilling. The depth of the groundwater level in this borehole upon completion of drilling was 0.15 m. The groundwater level was also measured in a monitoring well installed in BH-19-2-03 on October 2, 2019, about 3 days after the completion of the borehole. The results indicated a stabilized water level of 0.9 m.

It should be noted that the depth of the groundwater level and its gradient of flow in the area may be influenced by the placement of the road embankment, the surrounding landscape, the rock outcrops, underlying geology, and the water levels in creeks, local lakes and marshy grounds, and may fluctuate because of seasonal changes, periods of precipitation, and temperature. As such, the groundwater level may differ at any time throughout the year and during construction.

Groundwater data is presented on individual borehole logs provided in Appendix B.

5.3 Area 3

The subsurface investigation in Area 3 involved drilling three (3) boreholes (BH-19-3-01, BH-19-3-02, and BH-19-3-03) close to the toe, and two boreholes (BH-19-3-04 and BH-19-3-05) through the road embankment. Based on the information from these boreholes, the subsurface in the southwestern part of the embankment consisted of peat, silty clay and sand. In the southeastern part, a 1.5 m silty sand with gravel was encountered beneath the ground surface followed by gravelly sand materials. At the road, the asphalt layer was underlain by a fill composed of sand and gravel (sandy gravel). Beneath the granular fill, the embankment is composed of rock fill. Both the granular materials at the toe and the rock fill at the embankment are underlain by a greenstone bedrock. The following sections provide the descriptions of soils, rocks and groundwater conditions encountered in Area 3.

5.3.1 Peat

A layer of peat was encountered in BH-19-3-01 and BH-19-1-02 immediately below the ground surface. The peat was dark brown in color, amorphous and spongy, and wet. The thickness of the peat ranged from 1.4 m in BH-19-3-02 to 3 m in BH-19-3-01.

The moisture content of samples of the peat ranged from 148% to 588%. High moisture content values were obtained in samples taken from the lower portion of BH-19-3-01.

5.3.2 Silty Clay

A silty clay deposit was encountered underneath the peat in BH-19-3-01. The silty clay contained organic matter, and was grey in color and wet. The thickness of this deposit was 0.7 m, extending to the depth of 3.7 m (El. 456.5 m).

The SPT "N"-value within the silty clay deposit was negligible or "Weight of the Hammer (WH)" suggesting a "very soft" soil consistency.

The moisture content of the silty clay deposit was 144%.

Atterberg Limit tests performed on a sample of the silty clay gave a liquid limit of 64 and a plastic limit of 53, resulting in a plasticity index of 11. The results of these tests are reported on the Unified Soil Classification System plasticity chart presented on Fig. No. PC-3-1 in Appendix C. Based on the results, the deposit may be classified as organic silty clay of high compressibility (OH).

5.3.3 Sand with Gravel

A layer of sand with gravel was encountered in BH-19-3-01, at a depth of 3.7 m (El. 456.5 m), and extending to the full depth of investigation of 5.0 m (El. 455.7 m). The sand layer consisted of some silt and trace clay, and was grey in color and wet.

The SPT "N"-value in the upper part this deposit was 10 blows/300 mm, indicating "compact" state of relative density.

The moisture content of a retrieved sample from this deposit was 12.7%.

The grainsize analysis of the same sample gave rise a composition of 21% gravel, 62% sand, 16% silt and 1% clay. The grain size distribution curve is given on Fig. No. GSA-3-1 in Appendix C.

5.3.4 Silty Sand with Gravel

A silty sand with gravel deposit was encountered underneath the peat in BH-19-3-02 and immediately below the ground surface in BH-19-3-03. The deposit consisted of trace clay and organic matter, and was dark grey in color and wet. Its thickness ranged from 0.9 m in BH-19-3-02 to 1.5 m in BH-19-3-03, extending to the depth of 2.3 m (El. 457.3 m) and 1.5 m (El. 458.6 m), respectively.

The SPT "N"-values within the silty sand ranged from 4 blows/300 mm to 18 blows/300 mm, suggesting "very loose" to "compact" soil relative density.

The moisture content of samples from the silty sand was in the range of 14.6% to 23%.

The grainsize analyses of representative samples taken from both BH-19-3-02 and BH-19-3-03, resulted in a composition of 23% - 25% gravel, 46% - 49% sand, 24% - 30% silt, and 1% - 2% clay. The corresponding grain size distribution curve is presented on Fig. No. GSA-3-2 in Appendix C.

5.3.5 Gravelly Sand

A layer of gravelly sand was encountered beneath the silty sand in BH-19-3-03. The gravelly sand consisted of trace fines, and was dark brown in color and wet. It thickness was 1 m, extending to the full depth of investigation of 2.5 m (El. 457.7 m).

The SPT "N" values within this layer ranged from 17 blows/300 m to as high as 100 blows/300 m, indicating a "compact" to "very dense" relative density.

Moisture content tests carried out on retrieved samples yielded 26% to 103%.

The sieve analysis of a representative sample yielded a composition of 36% gravel, 57% sand, and 7% fines. The grain size distribution curve is given on Fig. No. GSA-3-3 in Appendix C.

5.3.6 Asphalt

A layer of asphalt was encountered in BH-19-3-04 and BH-19-3-05 drilled through the road embankment. The asphalt thickness at borehole locations was 180 mm.

5.3.7 Fill

A fill material was encountered beneath the asphalt layer. The fill consisted of gravelly sand to sand and gravel, with some silt, trace clay and cobbles. It was brown to grey in color and was dry to moist. The thickness of the fill ranged from 0.8 m to 1 m, extending to El. 463.4 m.

The SPT "N" values within the fill were in the range of 21 blows/300 mm to 46 blows/300 mm, indicating a "compact" to "dense" in-situ state of relative density.

Moisture content tests conducted on samples of the fill gave values from 4% to 7%.

The grainsize analyses of samples from BH-19-3-05 resulted in a composition of 33% - 72% gravel, 24% - 53% sand, 11% silt, and 3% clay. The corresponding grain size distribution curves are presented on Fig. No. GSA-3-4 in Appendix C.

5.3.8 Rock Fill

A rock fill was encountered beneath the granular fill in BH-19-3-04 and BH-19-3-05. No sampling and in-situ testing was conducted in the rock fill. Generally, field observations indicated that the rock fill consisted of all sizes of rocks including blasted rocks from the surrounding outcrops. The thickness of the rock fill ranged from 3.0 m in BH-19-3-04 to 3.1 m in BH-19-3-05, extending to El. 460.3 m.

No samples were retrieved from the rock fill for laboratory testing. ,

5.3.9 Bedrock

Rock outcrops are present both on the southwest and southeast ends of the road embankment. The presence of bedrock was confirmed through diamond coring in BH-19-3-02 and BH-19-3-04. The bedrock was encountered at a depth of 2.3 m (El. 457.3 m) in BH-19-3-02 and 5.8 m (El. 458.5 m) in BH-19-3-04. Coring was carried out for a length 3.4 m in BH-19-3-02 (El. 453.9 m). In BH-19-3-04, the rock coring extended for a length of 3.3 m or to the depth of 9.1 m (El. 455.2 m). A bedrock was also encountered in BH-19-3-05 at a depth of 4.3 m (El. 460.3 m), although the rock coring in this borehole was limited to 1.8 m and couldn't be used to confirm the presence of bedrock.

The bedrock encountered in all boreholes was described to be Greenstone. It was dark green to black in color, fine to medium grained, weakly to moderately foliated, and consisted of quartz and plagioclase crystals in a fine matrix of dark minerals. The rock cores were slightly weathered and moderately to highly fractured, and were medium to hard (based on geological hammer and knife test).

Measured Core Recovery (REC) was 100% both in BH-19-3-02 and BH-19-3-04, indicating less degree of weathering. The RQD values in BH-19-3-02 ranged from 83% to 100%. In BH-19-3-04, the RQD values were between 68% and 90%. Based on the RQD values, the quality of the rock mass near BH-19-3-02 was described to be “good” to “excellent”, while the quality of the rock mass beneath the road embankment or around BH-19-3-04 was “fair” to “good”.

A UCS test was conducted on a rock core taken at a depth of 5.9 m (El. 458.6 m) from BH-19-3-04. The results of the USC test indicated a compressive strength of 78.6 MPa.

Photo 24 and Photo 25 in Appendix D, show the rock cores from BH-19-3-02 and BH-19-3-04. The rock core descriptions (Table D3) and details of the USC test are also given in Appendix D.

5.3.10 Groundwater

Groundwater was encountered in BH-19-3-01 and BH-19-3-02 during drilling. The depth of the groundwater level in these boreholes upon completion of drilling was 0.15 m. The groundwater level was also measured in a monitoring well installed in BH-19-3-03 on October 2, 2019, about 4 days after the completion of the borehole. The results indicated a stabilized water level of 1.3 m.

It should be noted that the depth of the groundwater level and its gradient of flow in the area may be influenced by the placement of the road embankment, the surrounding landscape, the rock outcrops, underlying geology, and the water levels in creeks, local lakes and marshy grounds, and may fluctuate because of seasonal changes, periods of precipitation, and temperature. As such, the groundwater level may differ at any time throughout the year and during construction.

Groundwater data is presented on individual borehole logs provided in Appendix B.

5.4 Area 4

The subsurface investigation in Area 4 involved two (2) boreholes (BH-19-4-01 and BH-19-4-02) in the lake, one (1) borehole (BH-19-4-03) close to the toe, and one (1) borehole (BH-19-3-04) at the road embankment. Based on the information from the boreholes, the subsurface in the lake consisted of a peat layer underlain by silt, sandy silt and gravelly sand deposits. In the northeast part, a 1.4 m peat was encountered followed by silty sand and sandy silt deposits. At the road, the asphalt layer was underlain by a fill composed of gravelly sand. Beneath the fill, the embankment is composed of rock fill. The lower part of the stratigraphy consisted of a greenstone bedrock. The following sections provide the descriptions of soils, rocks and groundwater conditions encountered in the area.

5.4.1 Peat

A layer of peat was encountered immediately below the lake bottom in BH-19-4-02 and the ground surface in BH-19-1-03. The peat was dark brown in color, amorphous and spongy, and wet. The thickness of the peat ranged from 4.1 m in BH-19-3-02 to 1.4 m in BH-19-3-03.

The moisture content of samples of the peat ranged from 107% to 233%. High moisture content values were obtained in samples taken from the lower portion of BH-19-4-02.

5.4.2 Silty Sand to Sandy Silt

A layer of silty sand to sandy silt consisting of trace to some gravel and trace clay was encountered in BH-19-4-03 beneath the peat deposit. The sandy silt deposit with organic matter was also encountered immediately below the lake bottom in BH-19-4-01. The silty sand to sandy silt layer was dark brown to grey in color and wet. Its thickness in BH-19-4-01 was 0.5 m, extending to the full depth of investigation of 1.8 m from the top of the drilling platform (El. 449.5 m). In BH-19-4-03, the silty sand was 1.1 m thick, extending to the depth of 2.5 m (El. 449.6 m).

The SPT “N”-values within this deposit in BH-19-4-03 ranged from 23 blows/300 mm to over 100 blows/300 mm, and the deposit was described to be “compact” to “very dense”.

The moisture contents of samples of the silty sand ranged from 12% to 16%.

The grainsize analyses of samples taken from BH-19-4-03, yielded a composition of 10% - 25% gravel, 39% sand, 34% - 43% silt, and 2% clay. The corresponding grain size distribution curve is presented on Fig. No. GSA-4-1 in Appendix C.

5.4.3 Silt

A silt deposit was encountered underneath the peat in BH-19-4-02. The silt consisted of some sand, trace clay, and organic matter, and was dark brown in color and wet. The thickness of this silt deposit was 0.8 m, extending to the depth of 7.2 m from the drilling platform (El. 444.2 m).

The SPT “N”-value within this deposit was 7 blows/300 mm, suggesting “loose” soil relative density.

The moisture content of a retrieved sample was 89%.

The grainsize analysis of the same sample gave rise a composition of 0% gravel, 11% sand, 84% silt and 5% clay. The grain size distribution curve is shown on Fig. No. GSA-4-2 in Appendix C.

5.4.4 Asphalt

A layer of asphalt was encountered in BH-19-4-04 drilled through the embankment of Highway 17. The asphalt thickness at borehole location was 200 mm.

5.4.5 Fill

A fill material consisting of gravelly sand with trace to some silt, trace clay and trace cobbles was encountered beneath the asphalt layer. The fill was brown in color and dry. The thickness of the granular fill was 1.2 m, extending to El. 454.2 m.

The SPT “N” value within the fill was 72 blows/300 mm, indicating a “very dense” state of compaction.

Moisture content tests conducted on a sample of the fill yielded 4.5%.

The grainsize analysis of the same sample resulted in a composition of 32% gravel, 57% sand, 10% silt, and 1% clay. The grain size distribution curve is given on Fig. No. GSA-4-3 in Appendix C.

5.4.6 Rock Fill

A rock fill was encountered beneath the granular fill in BH-19-4-01. Because of difficult drilling conditions and auger grinding, the casing was inserted through the rock fill without sampling and in-situ testing. Generally, field observations indicated that the embankment fill consisted of all sizes of rocks including blasted rocks obtained from the surrounding outcrops. The thickness of the rock fill was 5.3 m, extending to the depth of 6.7 m from the top of the asphalt layer (El. 448.9 m).

No samples were retrieved from the rock fill for laboratory testing.

5.4.7 Sandy Gravel to Gravel

A layer of sandy gravel to gravel was encountered beneath the rock fill in BH-19-4-04. The sandy gravel to gravel layer consisted of cobbles, and was dark brown in color and moist. Trace organic matter was also obtained in the upper part of this layer, immediately beneath the rock fill. It thickness was 4.6 m, extending to the depth of 11.3 m (El. 444.3 m).

The SPT "N" values within this layer ranged from 13 blows/300 m to as high as 100 blows/300 m, indicating a "compact" to "very dense" relative density.

Moisture content values of samples ranged from negligible to 8.6%.

The sieve analyses of samples yielded 66% - 99% gravel, 1% - 30% sand, and 0% - 4% fines. The grain size distribution curves are presented on Fig. No. GSA-4-4 in Appendix C.

5.4.8 Bedrock

Rock outcrops are present both on the northwest and northeast ends of the road embankment or the lake shore. The presence of bedrock was confirmed through diamond coring in BH-19-4-02. The bedrock was encountered at a depth of 7.2 m from the drilling platform (El. 444.2 m). Rock coring in this borehole was carried out for a length 3.6 m (El. 440.5 m). A highly fractured bedrock was also encountered in BH-19-4-04 at a depth of 11.3 m (El. 444.3 m), although the rock coring in this borehole was limited to 1.8 m and could not be used to confirm the presence of a bedrock.

The bedrock encountered in BH-19-4-02 was described to be Greenstone. It was dark green to black in color, fine to medium grained, weakly to moderately foliated, and consisted of quartz and plagioclase crystals in a fine matrix of dark minerals. The rock cores were slightly weathered and moderately to highly fractured, and were medium to hard (based on geological hammer and knife test).

Measured Core Recovery (REC) in BH-19-4-02 was 100%. The corresponding RQD values were computed to be between 87% and 100%. Based on the RQD values, the quality of the rock mass in the lake near BH-19-3-02 was described to be "good" to "excellent".

A UCS test was conducted on a rock core taken at a depth of 7.6 m (El. 443.7 m) from BH-19-4-02. The results of the USC test indicated a compressive strength of 119.5 MPa.

Photo 26 in Appendix D, show the rock cores from BH-19-4-02. The corresponding rock core descriptions (Table D4) and details of the USC test are also given in Appendix D.

5.4.9 Groundwater

The lake level was at El. 450.8 both in BH-19-4-01 and BH-19-4-02. A monitoring well was installed in BH-19-4-03 to measure stabilized groundwater level. The groundwater level measured in this well on October 2, 2019, 2 days after the completion of the borehole, showed a water level at a depth of 0.8 m (El. 450.7 m), slightly lower than the elevation of the water surface in the lake.

It should be noted that the depth of the groundwater level and its gradient of flow in the area may be influenced by the placement of the road embankment, the surrounding landscape, the rock outcrops, underlying geology, and the water levels in creeks, local lakes and marshy grounds, and may fluctuate because of seasonal changes, periods of precipitation, and temperature. As such, the groundwater level may differ at any time throughout the year and during construction.

Groundwater data is presented on individual borehole logs provided in Appendix B.

5.5 Area 7

The subsurface investigation in Area 7 involved completing five (5) hand auger holes (HA-19-7-01, HA-19-7-02, HA-19-7-03, HA-19-7-04, and HA-19-7-05) close to the toe of the road embankment to confirm the depth of the bedrock surface. The hand augers were advanced to a maximum depth of 1.9 m (El. 418.6 m). No coring was carried out in Area 7 to conform the presence of bedrcok. The subsurface within the depth of investigation consisted of a peat and topsoil underlain by sandy silt, and sand and gravel. All hand augers reached refusal on probable bedrock. The following sections provide the descriptions of soils and groundwater conditions in the area.

5.5.1 Peat / Topsoil

A layer of peat was encountered immediately below the ground surface. In HA-19-7-01, a topsoil was encountered below the ground surface. The peat was dark brown in color, amorphous and spongy, and wet. The thickness of the peat ranged from 0.2 m in HA-19-7-05 to 0.9 m in HA-19-7-03, extending to El. 419.6 m. The topsoil in HA-19-7-01 was 0.1 m thick. The topsoil consisted of some sand and trace gravel, and organic matter, and was dark brown in color and wet.

5.5.2 Sandy Silt / Silt with Sand

A layer of sandy silt /silt with sand/ consisting of trace to some gravel and trace clay was encountered beneath the peat or topsoil deposits. The sandy silt / silt with sand layer was brown to grey in color and moist to wet. Its thickness ranged from 0.2 m in HA-19-7-01 to 0.7 m in HA-19-04.

The moisture contents of auger (grab) samples of the sandy silt ranged from 30% to 36%.

The grainsize analyses of samples taken from HA-19-7-03 and HA-19-7-05, yielded a composition of 17% - 21% gravel, 11% - 25% sand, 54% - 64% silt, and 4% clay. The corresponding grain size distribution curve is presented on Fig. No. GSA-7-1 in Appendix C.

5.5.3 Sand and Gravel

A layer of sand and gravel was encountered beneath the sandy silt. The sand and gravel layer consisted of cobbles, trace fines, and was brown to grey in color and moist to wet. It thickness ranged from 0.3 m to 0.5 m, extending to the depth of 1.9 m in HA-19-7-03 (El. 418.6 m).

5.5.4 Groundwater

Groundwater was observed during the fieldwork. The depth of the groundwater level upon completion of the hand augers was observed to vary from 0.3 m to 0.5 m.

Groundwater data is presented on individual logs provided in Appendix B.

6. Closure

The fieldwork was supervised by Mr. Joe Lin, Mr. Jun Wang and Mr. Moe Nasir under the direction of Mr. Lul Yimam, PhD. P.Eng. The drilling equipment were supplied and operated by Tatry Environmental Drilling of Timmins, Ontario and Marathon Underground Constructors Corp. of Greely, Ontario. Routine laboratory tests of soil samples were conducted at the GHD laboratory in Whitby, Ontario. Complex laboratory tests of rock cores were carried out at GHD's MTO accredited High Complexity material testing laboratory in Waterloo, Ontario. Surveying of the as-drilled borehole locations was conducted by Callon Dietz Inc. (Callon Dietz) of Sudbury, Ontario.

This report was prepared by Mr. Lul Yimam, PhD, P.Eng., Senior Geotechnical/Foundation Engineer and MTO Key Personnel. Mr. Nyle McIlveen. P.Eng. conducted an independent review of the report.

Sincerely,

GHD

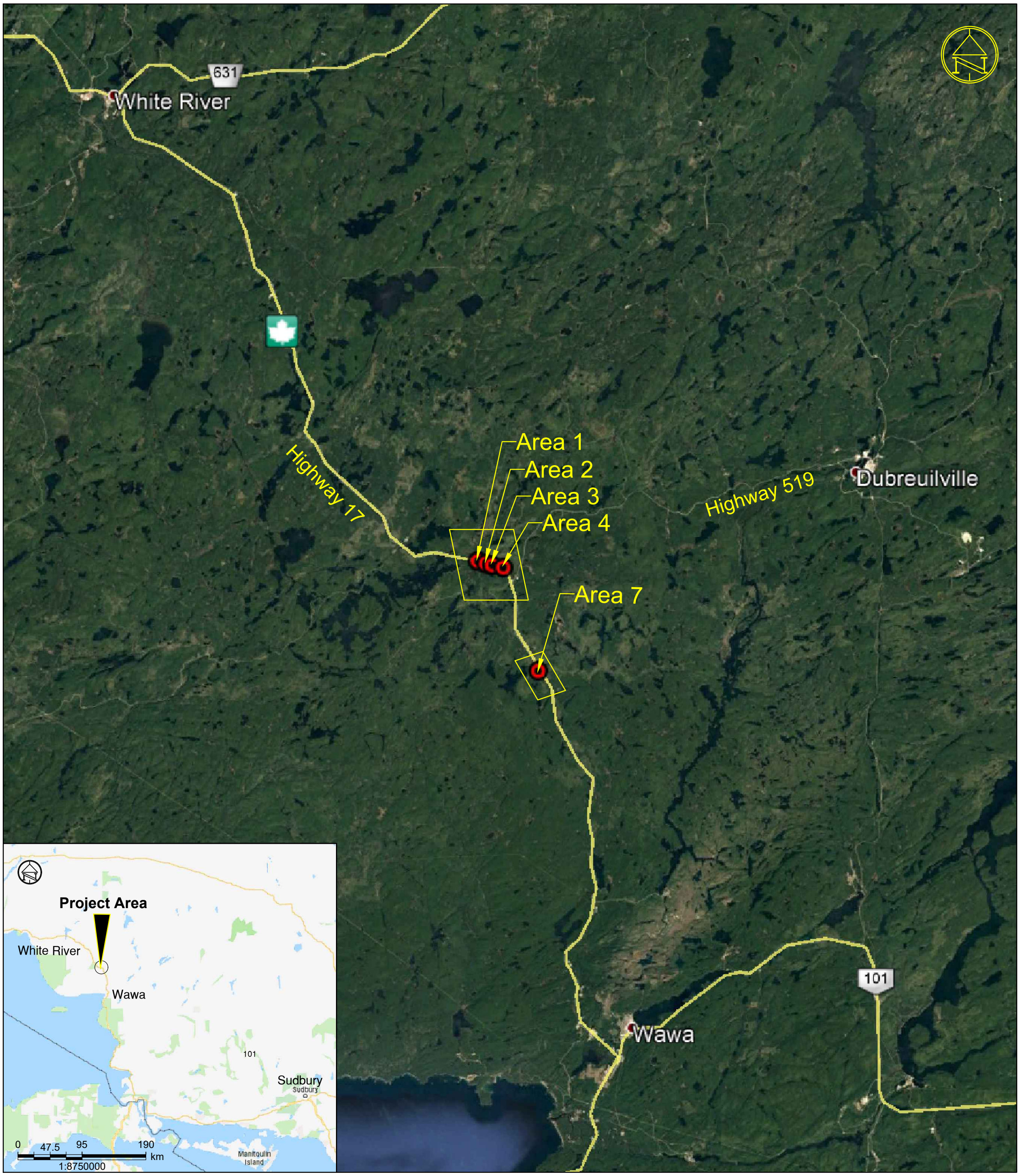
LICENSED PROFESSIONAL ENGINEER
L. A. YIMAM
100500748
Feb 3/2020
PROVINCE OF ONTARIO

Lul Yimam, PhD., P. Eng.
Project Manager and Senior Geotechnical/Foundation Engineer
MTO Key Personnel

LICENSED PROFESSIONAL ENGINEER
N. C. McILVEEN
Feb 3/20
PROVINCE OF ONTARIO

Nyle McIlveen, P.Eng.
Principal

Enclosure 1
Site Plan, Borehole Location Plan, and Soil Strata



LEGEND: Project Area

REFERENCE: Google Earth Pro 7.3.2.5776

Agreement No.	5017-E-0043	Work Item: # 5	
GWP:	5031-18-00	Hwy No.: 17	
Township:	Dambrossio & Lalibert		
District:	Algoma		
Drawn:	AW	Project No.:	11185012 (500)
Checked:	LY	Date:	2020.01.17
Reviewed:	LY	Revision:	
Approved:	NM	Drawing:	1

GHD

Site Plan

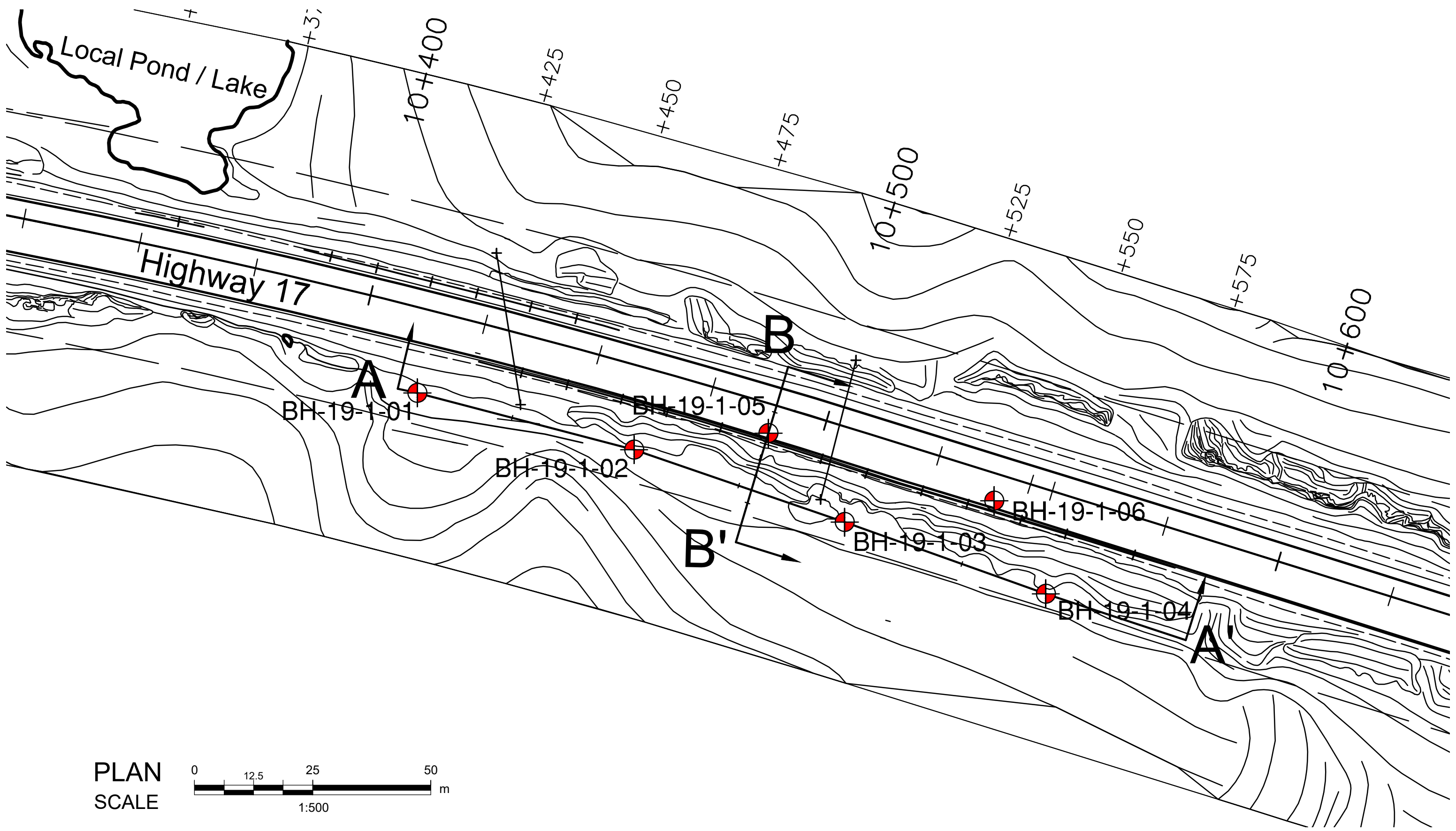
Foundation Investigation and Design

Hwy 17 Widening, Between White River and Wawa

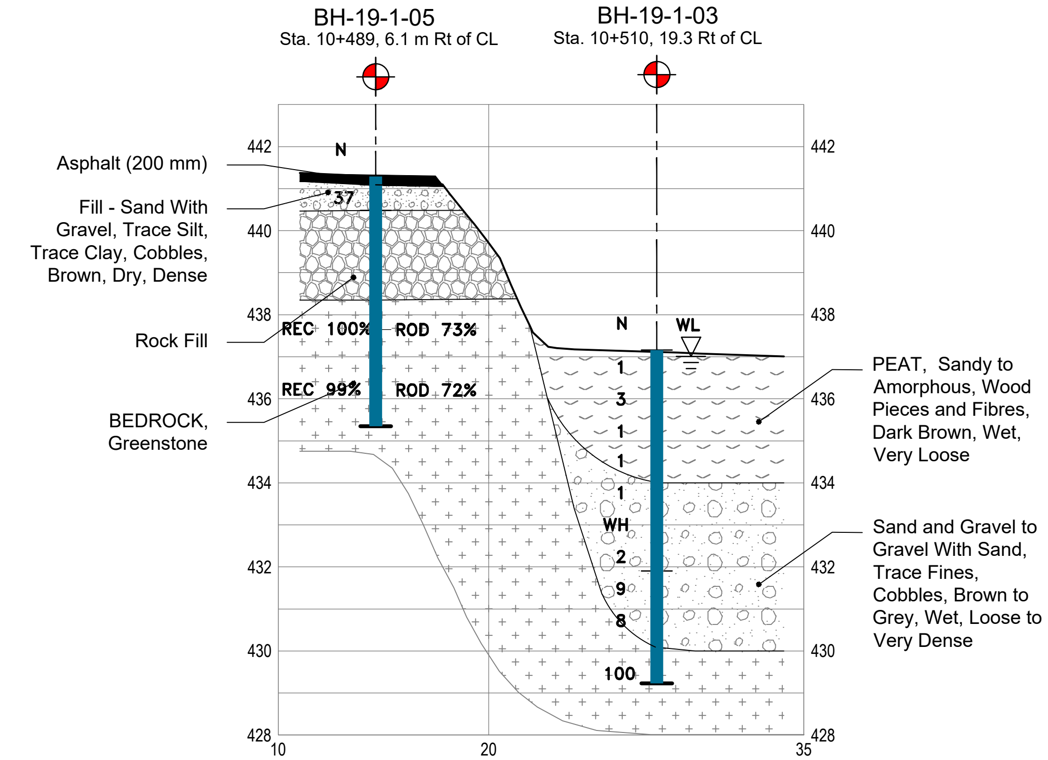
CONT No.:
Agreement No.: 5017-E-0043
Work Item No.: 5
G.W.P. No.: 5031-18-00

Borehole Location Plan and Soil Strata

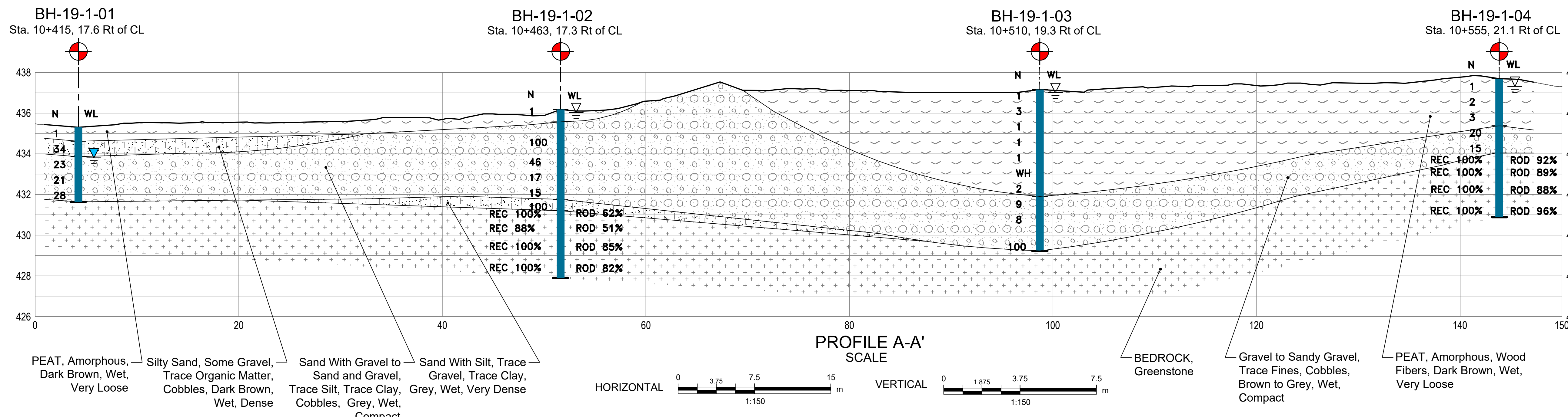
Sheet



PLAN
SCALE 1:500



PROFILE B-B'
SCALE
HORIZONTAL 1:120
VERTICAL 1:120



PROFILE A-A'
SCALE
HORIZONTAL 1:150
VERTICAL 1:150

DIST OF ALGOMA
TOWNSHIP OF DAMBROSSIO

KEY PLAN
SCALE 1:3700000

LEGEND

Borehole

Water Level (WL) Measured

Water Level (WL) Observed

BH No.	Northing (m)	Easting (m)	Elevation (m)
BH-19-1-01	5351072.8	236575.9	435.3
BH-19-1-02	5351060.8	236621.7	436.2
BH-19-1-03	5351045.4	236666.3	437.2
BH-19-1-04	5351030.3	236708.8	437.7
BH-19-1-05	5351064.2	236650.1	441.3
BH-19-1-06	5351050.0	236697.9	443.5

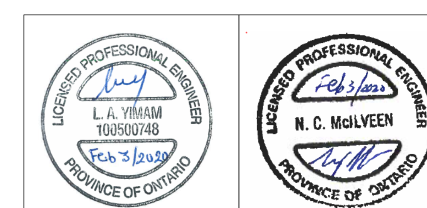
GHD Ref. 11185012 - 500

-Notes-

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF THE REPORT AND RECORD OF BOREHOLE LOGS.
2. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
3. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.



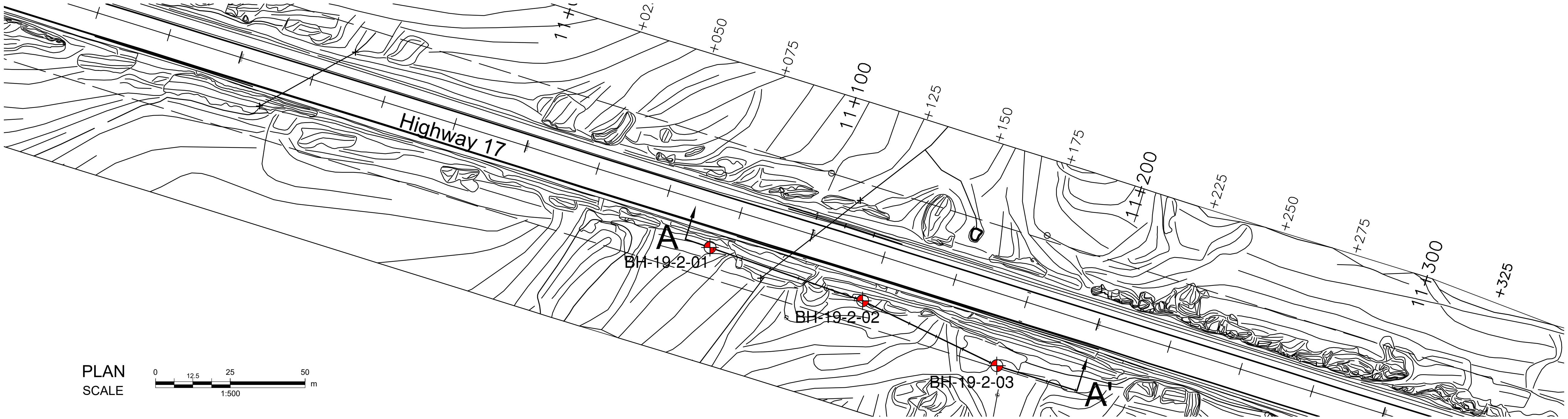
REF MTO DRAWING: SURVEY CROSS SECTIONS
DATE RECEIVED: JULY 18, 2019

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No.: 42C-047

SUBM'D:	LY	HWY No.:	17	DATE:	2020.02.03
DRAWN:	AW	DIST:	Algoma	DWG:	2
CHECKED:	LY	SITE:	Area 1		
APPROVED:	NM				

CONT No.: Agreement No.: 5017-E-0043 Work Item No.: 5 G.W.P. No.: 5031-18-00	
Borehole Location Plan and Soil Strata	Sheet



PLAN
SCALE 0 12.5 25 50 m 1:500

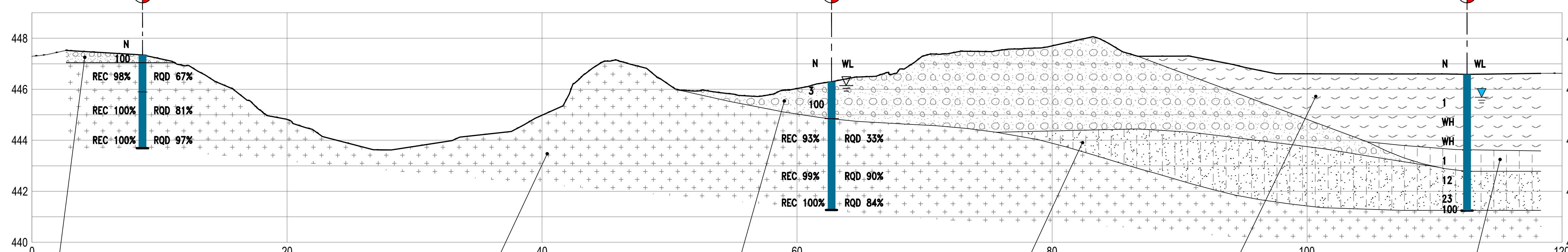
DIST OF ALGOMA
TOWNSHIP OF DAMBROSSIO

KEY PLAN SCALE 0 75 150 300 km 1:3700000

BH-19-2-01
Sta. 11+69, 14.0 Rt of CL

BH-19-2-02
Sta. 11+123, 15.8 Rt of CL

BH-19-2-03
Sta. 11+172, 23.2 Rt of CL



Gravelly Sand, Some Fines, Some Topsoil, Trace Organic Matter, Dark Brown, Moist, Very Dense

BEDROCK, Greenstone

Sand With Gravel to Sandy Gravel, Trace Fines, Trace Organic Matter, Dark Brown to Brown, Wet, Loose to Very Dense

Sand With Silt, Trace to Some Gravel, Trace Clay, Dark Brown, Wet, Compact to Very Dense

PEAT, Trace Sand, Trace Gravel, Wood Pieces, Amorphous, Dark Brown, Wet, Very Loose

Silt, Trace Gravel, Trace Sand, Trace Clay, Organic Matter, Grey, Wet, Very Soft

PROFILE A-A'
SCALE

HORIZONTAL 0 3.75 7.5 15 m 1:120

VERTICAL 0 1.875 3.75 7.5 m 1:120

LEGEND

- Borehole
- Water Level (WL) Measured
- Water Level (WL) Observed

BH No.	Northing (m)	Easting (m)	Elevation (m)
BH-19-2-01	5350884.3	237201.2	447.3
BH-19-2-02	5350866.4	237252.2	446.3
BH-19-2-03	5350844.7	237297.0	446.6

GHD Ref. 11185012 - 500

-Notes-

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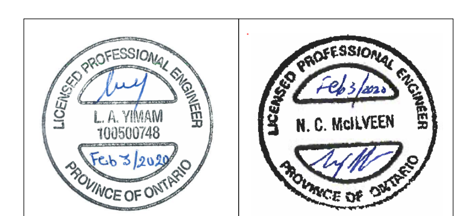
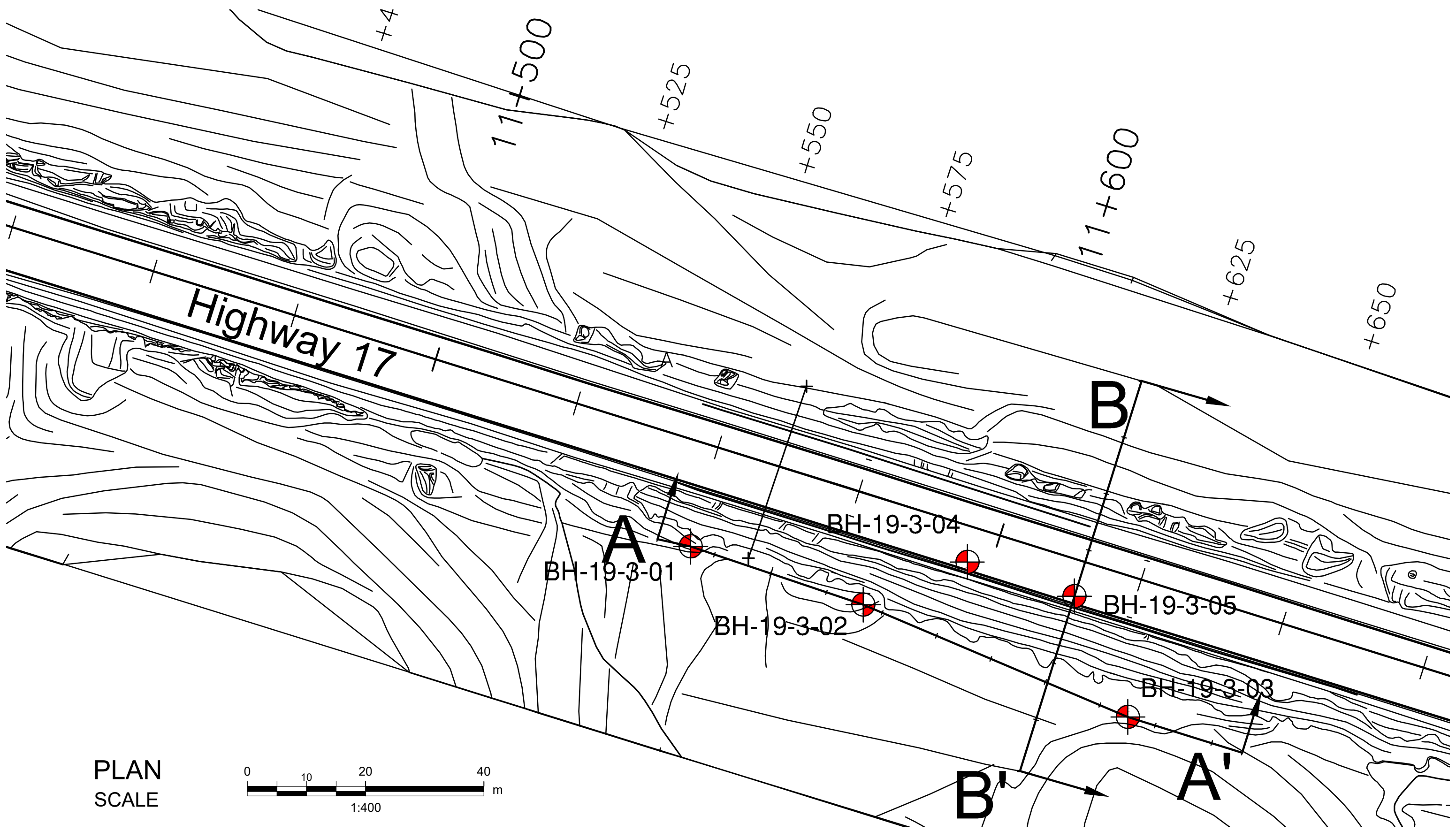


REF MTO DRAWING: SURVEY CROSS SECTIONS
DATE RECEIVED: JULY 18, 2019

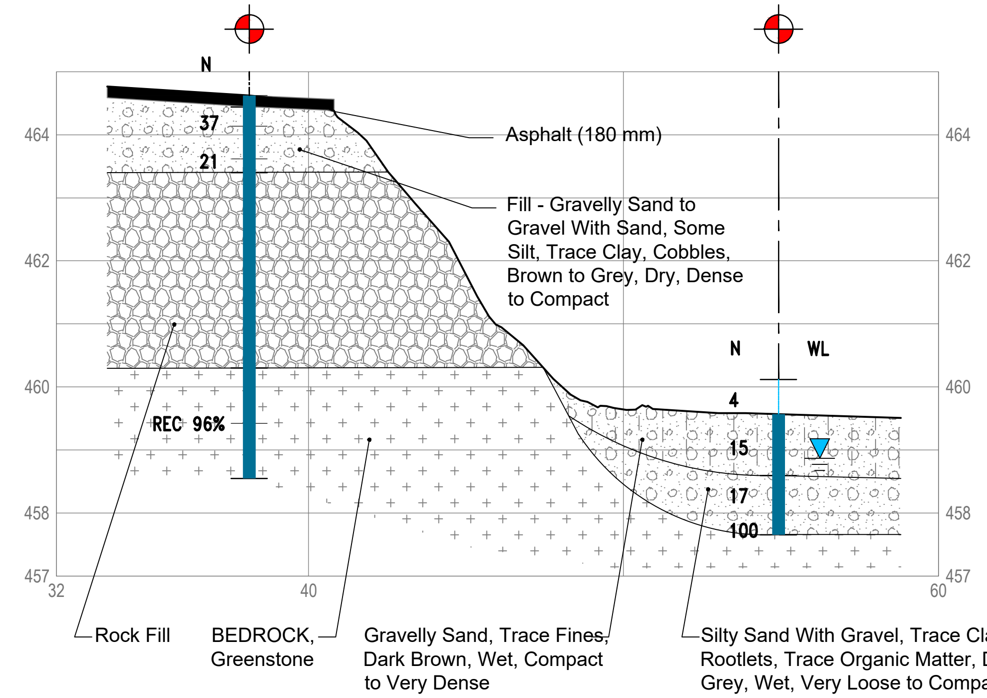
REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No.: 42C-047

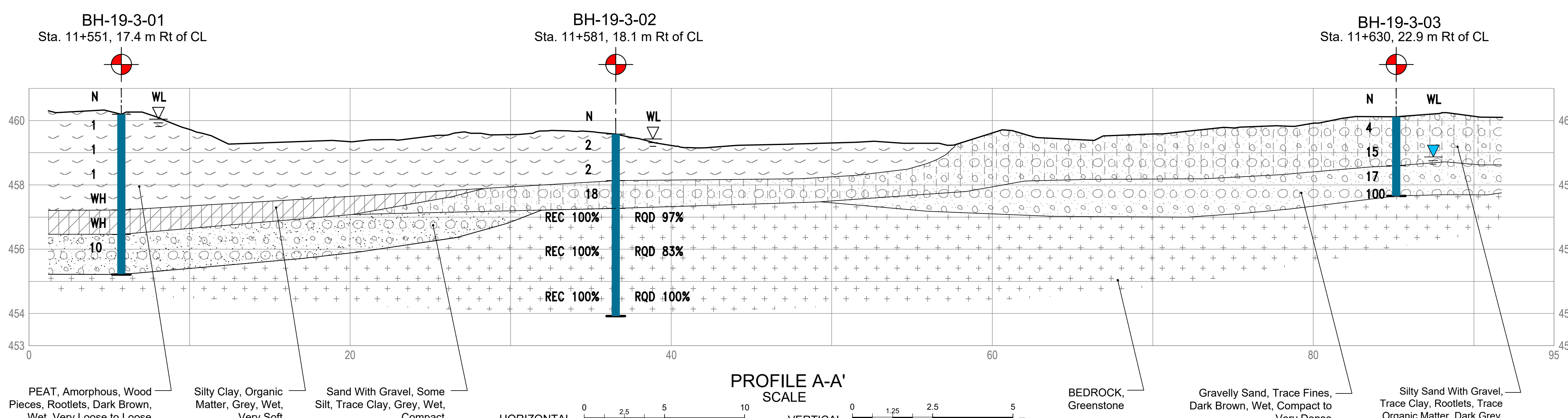
SUBM'D:	LY	HWY No.:	17	DATE:	2020.02.03
DRAWN:	AW	DIST:	Algoma	DWG:	3
CHECKED:	LY	SITE:	Area 2		
APPROVED:	NM				



PLAN
SCALE
0 10 20 40 m
1:400



PROFILE B-B'
SCALE
HORIZONTAL 0 1.25 2.5 5 m 1:80
VERTICAL 0 0.625 1.25 2.5 m 1:80



PROFILE A-A'
SCALE
HORIZONTAL 0 2.5 5 10 m 1:100
VERTICAL 0 1.25 2.5 5 m 1:100

NOTES:

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REF MTO DRAWING: SURVEY CROSS SECTIONS
DATE RECEIVED: JULY 18, 2019

CONT No.:
Agreement No.: 5017-E-0043
Work Item No.: 5
G.W.P. No.: 5031-18-00

Borehole Location Plan and Soil Strata

Sheet

DIST OF ALGOMA
TOWNSHIP OF DAMBROSSIO

KEY PLAN
SCALE
0 75 150 300 km
1:3700000

LEGEND

Borehole
Water Level (WL) Measured
Water Level (WL) Observed

BH No.	Northing (m)	Easting (m)	Elevation (m)
BH-19-3-01	5350737.6	237660.3	460.2
BH-19-3-02	5350727.8	237689.4	459.6
BH-19-3-03	5350708.8	237734.2	460.1
BH-19-3-04	5350735.0	237707.0	464.3
BH-19-3-05	5350729.2	237725.2	464.6

GHD Ref. 11185012 - 500

-Notes-

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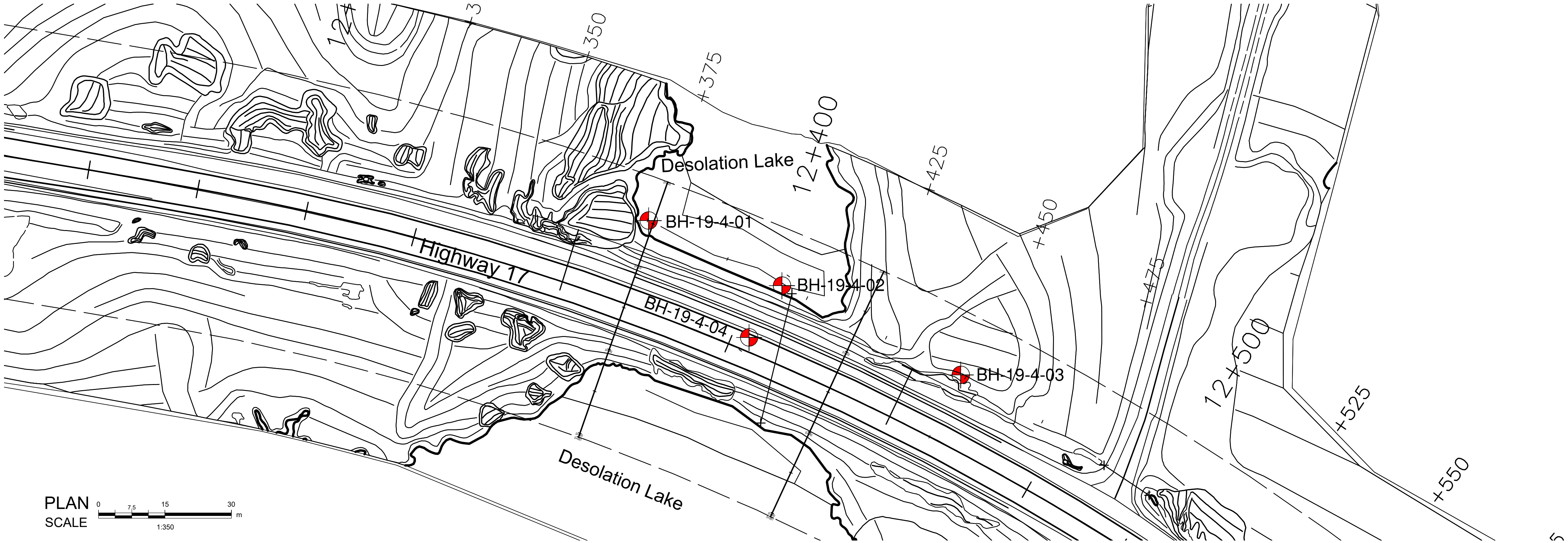
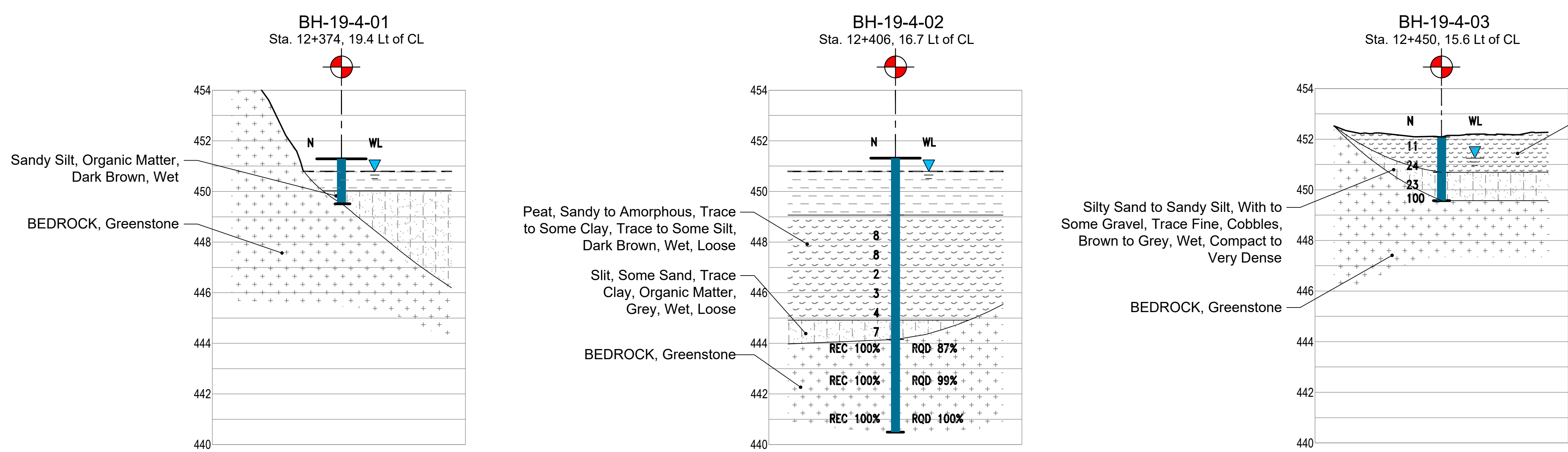
REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No.: 42C-047

SUBM'D:	LY	HWY No.:	17	DATE:	2020.02.03
DRAWN:	AW	DIST:	Algoma	DWG:	4
CHECKED:	LY	SITE:	Area 3		
APPROVED:	NM				





PROFILE
SCALE

HORIZONTAL 1:140

VERTICAL 1:140

NOTES:

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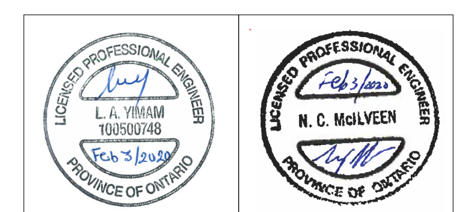

REF MTO DRAWING: SURVEY CROSS SECTIONS
DATE RECEIVED: JULY 18, 2019

CONT No.:
Agreement No.: 5017-E-0043
Work Item No.: 5
G.W.P. No.: 5031-18-00

Borehole Location Plan and Soil Strata

Sheet

DIST OF ALGOMA
TOWNSHIP OF DAMBROSSIO

KEY PLAN
SCALE 1:3700000

LEGEND

Borehole

Water Level (WL) Measured

Water Level (WL) Observed

BH No.	Northing (m)	Easting (m)	Elevation (m)
BH-19-4-01	5350576.1	238469.6	450.0
BH-19-4-02	5350561.4	238499.7	449.1
BH-19-4-03	5350541.3	238540.1	452.1
BH-19-4-04	5350549.7	238492.3	455.6

GHD Ref. 11185012 - 500

-Notes-

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS			
	DATE	BY	DESCRIPTION

GEOCRES No.: 42C-047

SUBM'D:	LY	HWY No.:	17	DATE:	2020.02.03
DRAWN:	AW	DIST:	Algoma	DWG:	5
CHECKED:	LY	SITE:	Area 4		
APPROVED:	NM				

CONT No.:
Agreement No.: 5017-E-0043
Work Item No.: 5
G.W.P. No.: 5031-18-00

Borehole Location Plan and Soil Strata

Sheet

DIST OF ALGOMA
TOWNSHIP OF LALIBERT

SCALE

KEY PLAN 0 75 150 300 km 1:5700000

LEGEND

Augerhole

BH No.	Northing (m)	Easting (m)	Elevation (m)
AH-19-7-01	5343192.6	240876.9	424.0
AH-19-7-02	5343177.4	240886.8	421.5
AH-19-7-03	5343167.7	240891.1	420.5
AH-19-7-04	5343157.7	240897.4	421.6
AH-19-7-05	5343145.7	240906.8	420.6

GHD Ref. 11185012 - 500

-Notes-

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

0 10 20 40 m

1:300

Notes:

1. DIMENSIONS ARE IN METERS/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METERS.

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No.: 42C-047

SUBM'D:	LY	HWY No.:	17	DATE:	2020.02.03
DRAWN:	AW	DIST:	Algoma	DWG:	6
CHECKED:	LY	SITE:	Area 7		
APPROVED:	NM				

Appendix A
Site Photographs

Photo 1 – Highway 17 - Area 1 Access Preparation - Looking West

Photo 2 – Highway 17 – Area 2 Access Preparation – Looking West

Photo 3 – Highway 17 - Area 3 Site Condition - Looking West

Photo 4 – Highway 17 – Area 4 Site Conditions - Looking East

Photo 5 – Highway 17 – Area 7 Site Conditions - Looking West

Photo 6 – Highway 17 - Area 1 – Drilling at BH-19-1-01 - Looking West

Photo 7 – Highway 17 - Area 1 – Drilling at BH-19-1-03 - Looking West

Photo 8 – Highway 17 - Area 1 – Drilling at BH-19-1-04

Photo 9 – Highway 17 - Area 2 – Drilling at BH-19-2-02

Photo 10 – Highway 17 - Area 2 – Drilling Setup at BH-19-2-03

Photo 11 – Highway 17 - Area 3 – Drilling Setup at BH-19-3-03

Photo 12 – Highway 17 - Area 3 – Drilling at BH-19-3-05 – Looking West

Photo 13 – Highway 17 - Area 4 – Boat Unloading and Setup – Looking East

Photo 14 – Highway 17 - Area 4 – Drilling Setup at BH-19-4-01 – Looking East

Photo 15 – Highway 17 - Area 4 – Drilling at BH-19-4-02

Photo 16 – Highway 17 - Area 4 – Drilling at BH-19-4-03

Photo 17 – Highway 17 - Area 4 – Drilling at BH-19-4-04 – Looking West

Photo 18 – Highway 17 - Area 7 – Hand Auger

Appendix B
Borehole Logs

Notes on Borehole and Test Pit Reports

Soil Description:

Each subsurface stratum is described using the following terminology. The relative density of granular soils is determined by the Standard Penetration Index ("N" value), while the consistency of clayey sols is measured by the value of undrained shear strength (Cu).

Soil Classification			
Clay (CL)	< 0.002 mm		
Silt (SI)	0.002 to 0.075 mm		
Sand (SA)	0.075 to 4.75 mm	fine	0.075 to 0.425 mm
		medium	0.425 to 2.0 mm
		coarse	2.0 to 4.75 mm
Gravel (GR)	4.75 to 75 mm	fine	4.75 to 19 mm
		coarse	19 to 75 mm
Cobbles	75 to 300 mm		
Boulders	> 300 mm		

Terminology	
"trace"	1-10%
"some"	10-20%
"with"	20-30%
adjective (silty, sandy)	30-40%
"and"	40-50%

Relative Density of Granular Soils	Standard Penetration Index "N" Value (Blows/ft - 300 mm)
Very Loose	0-4
Loose	5-9
Compact	10-29
Dense	30-50
Very Dense	>50

Consistency of Cohesive Soils	Undrained Shear Strength (C_u) (kPa)	(P.S.F)	"N " Value
Very Soft	<12	<250	0-2
Soft	12-25	250-500	3-4
Firm	25-50	500-1000	5-8
Stiff	50-100	1000-2000	9-15
Very Stiff	100-200	2000-4000	16-30
Hard	>200	>4000	>30

Field Moisture Conditions	
Dry	No signs of moisture, dusty and dry to touch
Moist	Damp but no visible water
Wet	Visible free water, soil may be below the groundwater table

Rock Quality Designation	
"RQD" (%) Value	Quality
<25	Very Poor
25-50	Poor
50-75	Fair
75-90	Good
>90	Excellent

Samples and Symbols:

The type of sample recovered is shown on the log by the abbreviation listed below.

SS: Split Spoon
TW: Shelby Tube
AS: Auger Sample
GS: Grab Sample
WR: Weight of Rod
RC: Rock Core
NQ: Core Size
WH: Weight of Hammer
SSE, GSE, AGE: Environmental Sampling

Recovery (REC)

The recovery, shown as a percentage, is the ratio of length of the sample obtained to the distance the sampler was driven/pushed into the soil

RQD

The "Rock Quality Designation" or "RQD" value, expressed as percentage, is the ratio of the total length of all core fragments of 4 inches (10 cm) or more to the total length of the run.

IN-SITU Tests:

N: Standard Penetration Index
N_c: Dynamic Cone Penetration Index
k: Permeability
VS: In-Situ Vane Shear Test
C_u: Undrained Shear Strength
ABS: Absorption (Packer Test)

Laboratory Tests:

I_p: Plasticity Index
HA: Hydrometer Analysis
AL: Atterberg Limits
PC: Plasticity Chart
W_L: Liquid Limit
GSA: Grain Size Analysis
w: Water Content
CT: Consolidation Test
W_p: Plastic Limit
SA: Sieve Analysis
γ: Unit Weight
UCS: Unconfined Compressive Strength

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-1-01

METRIC

G.W.P. NO. 5031-18-00 | LOCATION Highway 17, Between White River and Wawa | ORIGINATED BY Joe Lin

DIST Algoma HWY 17 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY Anne Wang

DATUM Geodetic | DATE 2019.09.05 | LATITUDE 48.29472605 | LONGITUDE -84.91956688 | CHECKED BY Lul Yimam

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
435.3 / 0.0	PEAT, Amorphous, Dark Brown, Wet, Very Loose	1	SS	1	435	490.9				
434.6 / 0.7	Silty Sand, Some Gravel, Trace Organic Matter, Cobbles, Dark Brown, Wet, Dense	2	SS	34	434					
433.9 / 1.4	Sand With Gravel, Trace Silt, Trace Clay, Cobbles, Grey, Wet, Compact	3	SS	23	433		30	57	11	2
		4	SS	21			23	68	(9)	
		5	SS	28	432					
431.6 / 3.7	End of Borehole									

Auger refusal on probable bedrock

▼ Water Level measured in well / piezometre on 2019.10.02

Column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; STRAT PLOT; GROUND WATER CONDITIONS; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-1-02

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Joe Lin

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.11 - 2019.09.12 LATITUDE 48.29462313 LONGITUDE -84.91894737 CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
436.2 / 0.0	Peat, Sandy to Amorphous, Dark Brown, Wet, Very Loose	1	SS	1	▽	436	176	
435.6 / 0.6	Sand and Gravel, Cobbles, Trace to Some Silt, Brown to Grey, Wet, Very Dense to Compact					435		
	Cobbles, Very Dense	2	SS	100+				
		3	SS	46		434	○	
		4	SS	17		433	○	
		5	SS	15		432	○	
431.8 / 4.4	Sand With Silt, Trace Gravel, Trace Clay, Grey, Wet, Very Dense	6	SS	100+			○	9 66 23 2
431.2 / 5.0	BEDROCK, Greenstone	1	NQ	REC 100%		431		RQD 62%
		2	NQ	REC 88%				RQD 51%
		3	NQ	REC 100%		430 / 429		RQD 85%
		4	NQ	REC 100%		428		RQD 82%
427.9 / 8.3	End of Borehole							

▽ Water Level measured upon completion of drilling

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

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-1-03

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Joe Lin

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.12 LATITUDE 48.29449011 LONGITUDE -84.91834494 CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
437.2 / 0.0	PEAT, Sandy to Amorphous, Wood Pieces and Fibres, Dark Brown, Wet, Very Loose	1	SS	1	∇	617.6				
		2	SS	3		445.9				
		3	SS	1		180.6				
		4	SS	1		179.9				
		5	SS	1		723.9				
		6	SS	WH						
		7	SS	2						
431.9 / 5.3	Sand and Gravel to Gravel With Sand, Trace Fines, Cobbles, Brown to Grey, Wet, Loose to Very Dense	8	SS	9			40	52	(8)	
		9	SS	8			45	48	(7)	
		10	SS	100+			69	25	(6)	
429.2 / 7.9	End of Borehole									

End of Borehole

Auger refusal on probable bedrock

∇ Water Level measured upon completion of drilling

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; ELEVATION SCALE (437 to 430); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-1-04

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Joe Lin

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.06 - 2019.09.11 LATITUDE 48.29435854 LONGITUDE -84.91776953 CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
437.7 / 0.0	PEAT, Amorphous, Wood Fibers, Dark Brown, Wet, Very Loose	1	SS	1	∇		200	
		2	SS	2		437	287.1	
		3	SS	3		436	170.3	
435.4 / 2.3	Gravel to Sandy Gravel, Trace Fines, Cobbles, Brown to Grey, Wet, Compact	4	SS	20		435	o	88 12 (0)
		5	SS	15			o	59 33 (8)
434.0 / 3.7	BEDROCK, Greenstone	1	NQ	REC 100%		434		RQD 92%
		2	NQ	REC 100%		433		RQD 89%
		3	NQ	REC 100%		432		RQD 88%
		4	NQ	REC 100%		431		RQD 96%
430.9 / 6.8	End of Borehole							

∇ Water Level measured upon completion of drilling

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m^3

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-1-05

METRIC

G.W.P. NO. 5031-18-00	LOCATION Highway 17, Between White River and Wawa		ORIGINATED BY Jun Wang
DIST Algoma HWY 17	BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring		COMPILED BY Anne Wang
DATUM Geodetic	DATE 2019.09.24	LATITUDE 48.29465726 LONGITUDE -84.91856528	CHECKED BY Lul Yimam

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
441.3 / 0.0	Asphalt (200 mm)								
441.1 / 0.2	Fill - Sand With Gravel, Trace Silt, Trace Clay, Cobbles, Brown, Dry, Dense	1	SS	37		441	o		28 61 10 1
440.5 / 0.8	Rock Fill					440, 439			
438.3 / 2.9	BEDROCK, Greenstone	1	NQ	REC 100%		438, 437			RQD 73%
		2	NQ	REC 99%		436			RQD 72%
435.3 / 5.9	End of Borehole Difficult drilling / casing condition & slow progress through the rockfill								

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-1-06

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger, and NW Casing COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.24 LATITUDE 48.2945342 LONGITUDE -84.91791957 CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
443.5											
0.0	Asphalt (230 mm)										
443.3 / 0.2	Fill - Sand and Gravel, Trace Silt, Trace Clay, Rock Fragments, Brown to Grey, Dry to Moist, Dense to Compact		1	SS	34		443		○		48 48 (4)
			2	SS	18				○		36 55 7 2
442.1 / 1.4	Rock Fill						442				
							441				
440.7 / 2.8	End of Borehole Casing refusal on Probable bedrock										

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-2-01

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.29 LATITUDE 48.29309826 LONGITUDE -84.91111033 CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
447.3										
0.0	Gravelly Sand, Some Fines, Some Topsoil, Trace Organic Matter, Dark Brown, Moist, Very Dense		1	SS	100+			o (≈42)		28 61 (11)
447.0 / 0.3	BEDROCK, Greenstone						447			
			1	NQ	REC 98%		446			RQD 67%
			2	NQ	REC 100%		445			RQD 81%
			3	NQ	REC 100%		444			RQD 97%
443.7 / 3.7	End of Borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-2-02

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.28 LATITUDE 48.29294309 LONGITUDE -84.91042019 CHECKED BY Lul Yimam

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
446.3 / 0.0	Sand With Gravel to Sandy Gravel, Trace Fines, Trace Organic Matter, Dark Brown to Brown, Wet, Loose to Very Dense	1	SS	3	∇	446	92.3		26 70 (4)
		2	SS	100+					33 55 10 2
444.8 / 1.4	BEDROCK, Greenstone					445			
		1	NQ	REC 93%		444			RQD 33%
		2	NQ	REC 99%		443			RQD 90%
		3	NQ	REC 100%		442			RQD 84%
441.3 / 5.0	End of Borehole ∇ Water Level measured upon completion of drilling								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

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

ONTARIO MTO (RETAINER) 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MTO.GDT 17/1/20

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-2-03

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.29 LATITUDE 48.29275248 LONGITUDE -84.90981235 CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
446.6 / 0.0	PEAT, Trace Sand, Trace Gravel, Wood Pieces, Amorphous, Dark Brown, Wet, Very Loose	1	SS	1		
		2	SS	WH	617.2	
		3	SS	WH	136.5	
443.6 / 3.0	Silt, Trace Gravel, Trace Sand, Trace Clay, Organic Matter, Grey, Wet, Very Soft	4	SS	1	91.2	1 7 89 3
442.8 / 3.8	Sand With Silt, Trace to Some Gravel, Trace Clay, Dark Brown, Wet, Compact to Very Dense	5	SS	12		10 61 27 2
		6	SS	23		
		7	SS	100+		
441.2 / 5.3	End of Borehole					

Elevation scale: 446, 445, 444, 443, 442

Dynamic cone penetration resistance plot: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Auger refusal on probable bedrock

Sampling started at 0.75 m because of the presence pieces of undecomposed wood

Water Level measured in well / piezometre on 2019.10.02

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

ONTARIO MTO (RETAINER) 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MTO.GDT 17/1/20

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-3-01

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.25 LATITUDE 48.29182823 LONGITUDE -84.90490031 CHECKED BY Lul Yimam

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
460.2 / 0.0	PEAT, Amorphous, Wood Pieces, Rootlets, Dark Brown, Wet, Very Loose to Loose		1	SS	1	▽	460	331					
			2	SS	1		459	586.4					
			3	SS	1			580.3					
			4	SS	WH		458	588.1					
457.2 / 3.0	Silty Clay, Organic Matter, Grey, Wet, Very Soft		5	SS	WH		457	133.9					
456.5 / 3.7	Sand With Gravel, Some Silt, Trace Clay, Grey, Wet, Compact		6	SS	10		456			21	62	16	1
455.2 / 5.0	End of Borehole Auger refusal on probable bedrock ▽ Water Level measured upon completion of drilling												

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

ONTARIO MTO (RETAINER) 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MTO.GDT 17/1/20

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-3-02

METRIC

G.W.P. NO. 5031-18-00 | LOCATION Highway 17, Between White River and Wawa | ORIGINATED BY Jun Wang

DIST Algoma HWY 17 | BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring | COMPILED BY Anne Wang

DATUM Geodetic | DATE 2019.09.25 | LATITUDE 48.2917428 | LONGITUDE -84.90450538 | CHECKED BY Lul Yimam

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
459.6 / 0.0	PEAT, Amorphous, Dark Brown, Wet, Very Loose	1	SS	2	∇		147.9	
		2	SS	2		459	191.2	
458.1 / 1.4	Silty Sand With Gravel, Trace Clay, Organic Matter, Dark Grey, Wet, Compact	3	SS	18		458		23 46 30 1
457.3 / 2.3	BEDROCK, Greenstone	1	NQ	REC 100%		457		RQD 97%
		2	NQ	REC 100%		456		RQD 83%
		3	NQ	REC 100%		455		RQD 100%
453.9 / 5.7	End of Borehole					454		

∇ Water Level observed upon completion of drilling

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

ONTARIO MTO (RETAINER) 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MTO.GDT 17/1/20

RECORD OF BOREHOLE No BH-19-3-03

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.28 LATITUDE 48.29157693 LONGITUDE -84.90389915 CHECKED BY Lul Yimam

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
460.1 / 0.0	Silty Sand, With Gravel, Trace Clay, Rootlets, Trace Organic Matter, Dark Grey, Wet, Very Loose to Compact	1	SS	4		
		2	SS	15		25 49 24 2
458.6 / 1.5	Gravelly Sand, Trace Fines, Dark Brown, Wet, Compact to Very Dense	3	SS	17		
		4	SS	100+	103.4	36 57 (7)
457.7 / 2.5	End of Borehole					

Elevation scale: 460, 459, 458

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

Auger refusal on probable bedrock

▼ Water Level measured in well / piezometre on 2019.10.02

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

RECORD OF BOREHOLE No BH-19-3-04

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.26 LATITUDE 48.29180965 LONGITUDE -84.90426936 CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
464.3 / 0.0	Asphalt (180 mm)					
464.1 / 0.2	Fill - Sand and Gravel, Trace Fines, Cobbles, Brown to Grey, Dry to Moist, Dense	1	SS	46	○	
463.5 / 0.8	Rock Fill					
460.5 / 3.8	Sand and Gravel, Trace to Some Silt, Trace Clay, Dark Brown, Wet, Compact	2	SS	16	○	45 43 (12)
		3	SS	21	○	34 49 16 1
458.5 / 5.8	BEDROCK, Greenstone	1	NQ	REC 100%		RQD 90%
		2	NQ	REC 100%		RQD 78%
		3	NQ	REC 100%		RQD 68%
455.2 / 9.1	End of Borehole					

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (464, 463, 462, 461, 460, 459, 458, 457, 456); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; STRAT PLOT.

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

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No BH-19-3-05

METRIC

G.W.P. NO. 5031-18-00	LOCATION Highway 17, Between White River and Wawa		ORIGINATED BY Jun Wang
DIST Algoma HWY 17	BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring		COMPILED BY Anne Wang
DATUM Geodetic	DATE 2019.09.28	LATITUDE 48.29175957 LONGITUDE -84.90402415	CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
464.6														
0.0 / 464.4	Asphalt (180 mm)													
0.2	Fill - Gravelly Sand to Gravel With Sand, Some Silt, Trace Clay, Cobbles, Brown to Grey, Dry, Dense to Compact		1	SS	37		464		o		33	53	11	3
			2	SS	21				o		72	24	(4)	
463.4 / 1.2	Rock Fill						463, 462, 461							
460.3 / 4.3	BEDROCK, Greenstone		1	NQ	REC 96%		460, 459				RQD 42%			
458.5 / 6.1	End of Borehole Possible bedrock encountered at 4.32 m													

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100. PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60. REMARKS & GRAIN SIZE DISTRIBUTION (%).

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-4-01

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Moe Nasir

DIST Algoma HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.10.03 LATITUDE 48.29046062 LONGITUDE -84.89396816 CHECKED BY Lul Yimam

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
451.3 0.0	Top of Drilling Platform										
							451				
450.8 0.5	Water Surface					▼					
450.0 1.3	Lake Bottom Sandy Silt, Organic Matter, Dark Brown, Wet						450				
449.5 1.8	End of Borehole Auger refusal on probable bedrock at 1.8 m below the top of drilling platform / boat deck ▼ Lake Level / Water Surface										

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-4-02

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Moe Nasir

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.10.02 - 2019.10.03 LATITUDE 48.29033197 LONGITUDE -84.89356036 CHECKED BY Lul Yimam

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
451.3 / 0.0	Top of Drilling Platform						
450.8 / 0.5	Water Surface				▼		
449.1 / 2.3	Lake Bottom Peat, Sandy to Amorphous, Trace to Some Clay, Trace to Some Silt, Dark Brown, Wet, Loose	1	SS	8			
		2	SS	8		155.9	
		3	SS	2		233.9	
		4	SS	3		224.8	
		5	SS	4			
444.9 / 6.4	Slit, Some Sand, Trace Clay, Organic Matter, Grey, Wet, Loose	6	SS	7		89	0 11 84 5
444.2 / 7.2	BEDROCK, Greenstone	1	NQ	REC 100%			RQD 87%
		2	NQ	REC 100%			RQD 99%
		3	NQ	REC 100%			RQD 100%
440.5 / 10.8	End of Borehole						

▼ Lake Level / Water Surface

Plot legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; ELEVATION SCALE 451–441.

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-4-03

METRIC

G.W.P. NO. 5031-18-00	LOCATION Highway 17, Between White River and Wawa		ORIGINATED BY Jun Wang
DIST Algoma HWY 17	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY Anne Wang
DATUM Geodetic	DATE 2019.09.30	LATITUDE 48.29015524 LONGITUDE -84.89301315	CHECKED BY Lul Yimam

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
452.1 0.0	PEAT, Amorphous, Some Sand, Some Gravel, Dark Brown, Wet, Compact		1	SS	11		452	164.4					
			2	SS	24		451	107.3					
450.7 1.4	Silty Sand to Sandy Silt, With to Some Gravel, Trace Fine, Cobbles, Brown to Grey, Wet, Compact to Very Dense		3	SS	23		450			25	39	34	2
			4	SS	100+					16	39	43	2
449.6 2.5	End of Borehole Auger refusal on probable bedrock Water Level measured in well / piezometre												

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

ONTARIO MTO (RETAINER) 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MTO.GDT 17/1/20

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No BH-19-4-04

METRIC

G.W.P. NO. 5031-18-00 | LOCATION Highway 17, Between White River and Wawa | ORIGINATED BY Jun Wang

DIST Algoma | HWY 17 | BOREHOLE TYPE Hollow Stem Auger, and NW Casing + NQ Coring | COMPILED BY Anne Wang

DATUM Geodetic | DATE 2019.10.01 | LATITUDE 48.29022617 | LONGITUDE -84.89365822 | CHECKED BY Lul Yimam

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
455.6					
455.4 / 0.0	Asphalt (200 mm)				
0.2	Fill - Gravelly Sand, Some Silt, Trace Clay, Cobbles, Brown, Dry, Very Dense	1	SS	72	32 57 10 1
454.2 / 1.4	Rock Fill	2	SS	100+	
448.9 / 6.7	Sandy Gravel to Gravel, Cobbles, Trace Organic Matter, Dark Brown, Moist, Compact to Very Dense	3	SS	31	66 30 (4)
		4	SS	13	99 1 (0)
		5	SS	100+	
444.3 / 11.3	Bedrock, Greenstone	1	NQ	REC 60%	RQD 26%
442.5 / 13.1	End of Borehole				

Possible bedrock encountered at 11.3 m (El. 444.3 m)

Auger grinding from the depth of 6.7 m (El. 449 m), and difficult drilling condition because of probable cobbles and boulders (blast rock)

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (455 to 443); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No HA-19-7-01

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.16 LATITUDE 48.22430857 LONGITUDE -84.8604126 CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
424.0											
423.9 / 0.0	TOPSOIL										
0.1	Sandy Silt, Some Gravel, Trace Clay, Brown, Moist										
423.7 / 0.3	Sand and Gravel, Trace Silt, Trace Clay, Cobbles, Brown, Moist to Wet										
423.2 / 0.8	End of Augerhole Auger refusal on probable bedrock										

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

O 3% STRAIN AT FAILURE

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No HA-19-7-02

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.16 LATITUDE 48.22417282 LONGITUDE -84.86027641 CHECKED BY Lul Yimam

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L – WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
421.5 0.0	PEAT, Amorphous, Wood Fibres, Dark Brown, Wet										
421.1 0.4	Sand and Gravel, Trace Fines, Trace Organic Matter, Cobbles, Dark Brown, Moist to Wet					▽	421				
420.8 0.7	End of Augerhole Auger refusal on probable bedrock ▽ Water Level observed upon completion of augerhole										

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

Ministry of Transportation Ontario

GHD

RECORD OF BOREHOLE No HA-19-7-03

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.17 LATITUDE 48.22408525 LONGITUDE -84.8602172 CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
420.5											
0.0	PEAT, Amorphous, Wood Fibres, Dark Brown, Moist to Wet, Loose				∇	420					
419.6 / 0.9	Silt With Sand, Some Gravel, Trace Clay, Brown to Grey, Wet	1	AS				~30	17	25	54	4
419.0 / 1.5	Sand and Gravel, Trace Fines, Cobbles, Brown to Grey, Wet					419					
418.6 / 1.9	End of Augerhole										

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

Auger refusal on probable bedrock

∇ Water Level observed upon completion of augerhole

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No HA-19-7-04

METRIC

G.W.P. NO. 5031-18-00 LOCATION Highway 17, Between White River and Wawa ORIGINATED BY Jun Wang

DIST Algoma HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY Anne Wang

DATUM Geodetic DATE 2019.09.17 LATITUDE 48.22399599 LONGITUDE -84.860131 CHECKED BY Lul Yimam

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
421.6 / 0.0	PEAT, Amorphous, Wood Fibres, Dark Brown, Moist										
421.4 / 0.2	Sandy Silt, Some Gravel, Trace Clay, Cobbles, Brown to Grey, Moist to Wet					▽	421				
420.7 / 0.9	End of Augerhole Auger refusal on probable bedrock ▽ Water Level observed upon completion of augerhole										

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No HA-19-7-05

METRIC

G.W.P. NO. 5031-18-00 | LOCATION Highway 17, Between White River and Wawa | ORIGINATED BY Jun Wang

DIST Algoma | HWY 17 | BOREHOLE TYPE Hand Auger | COMPILED BY Anne Wang

DATUM Geodetic | DATE 2019.09.17 | LATITUDE 48.22388985 | LONGITUDE -84.86000375 | CHECKED BY Lul Yimam

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
420.6														
0.0	PEAT, Amorphous, Dark Brown, Moist													
420.4 / 0.2	Silt, Some Gravel, Some Sand, Trace Clay, Cobbles, Brown to Grey, Moist to Wet		1	AS		▽	420		w ≈ 36		21	11	64	4
419.9 / 0.7	End of Augerhole													

Auger refusal on probable bedrock

▽ Water Level observed upon completion of augerhole

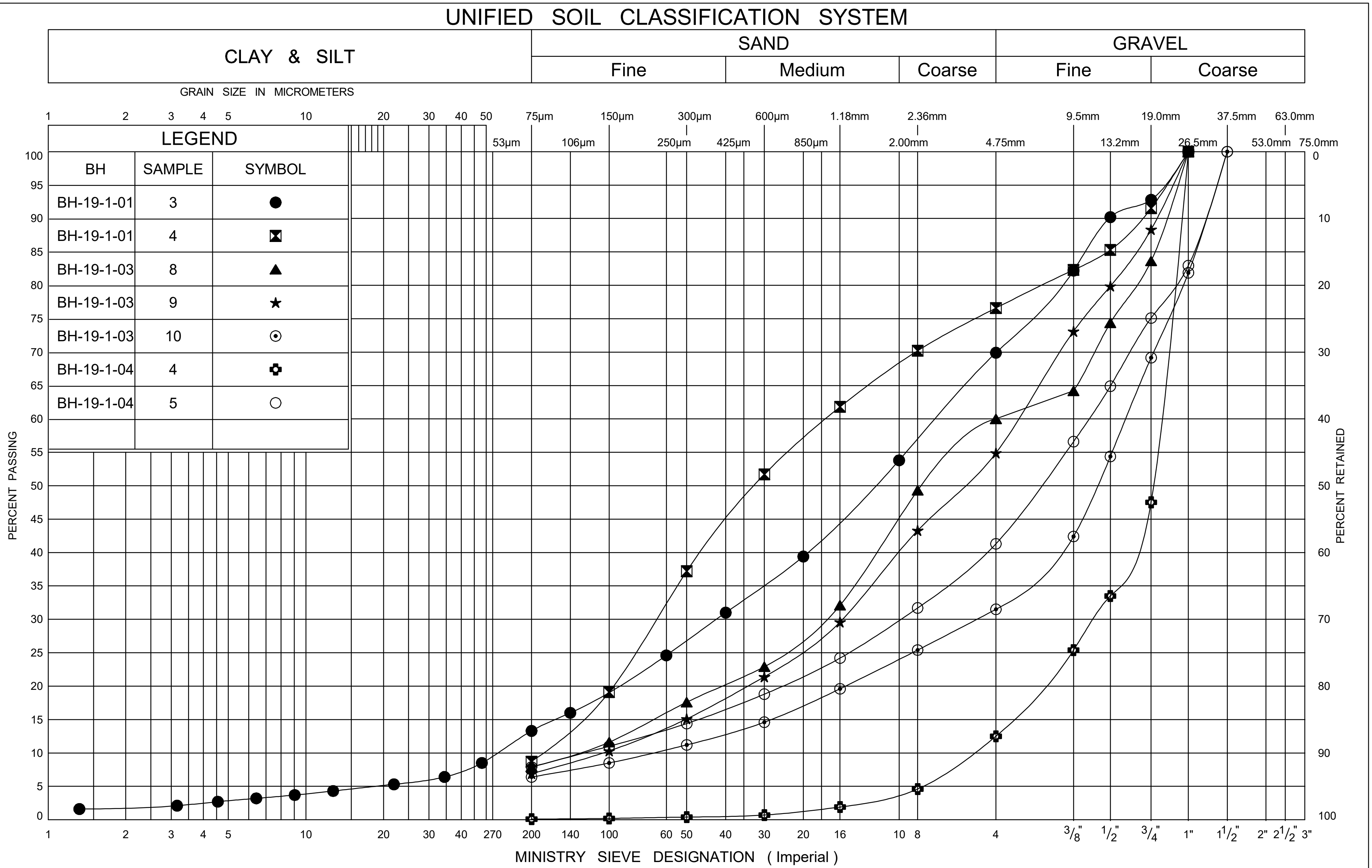
Legend: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE. Plastic limit w_P, natural moisture content w, liquid limit w_L.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

ONTARIO MTO (RETAINER) 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MTO.GDT 17/1/20

Appendix C
Laboratory Test Results of Soil Samples



BH	SAMPLE	SYMBOL
BH-19-1-01	3	●
BH-19-1-01	4	⧗
BH-19-1-03	8	▲
BH-19-1-03	9	★
BH-19-1-03	10	⊙
BH-19-1-04	4	✿
BH-19-1-04	5	○

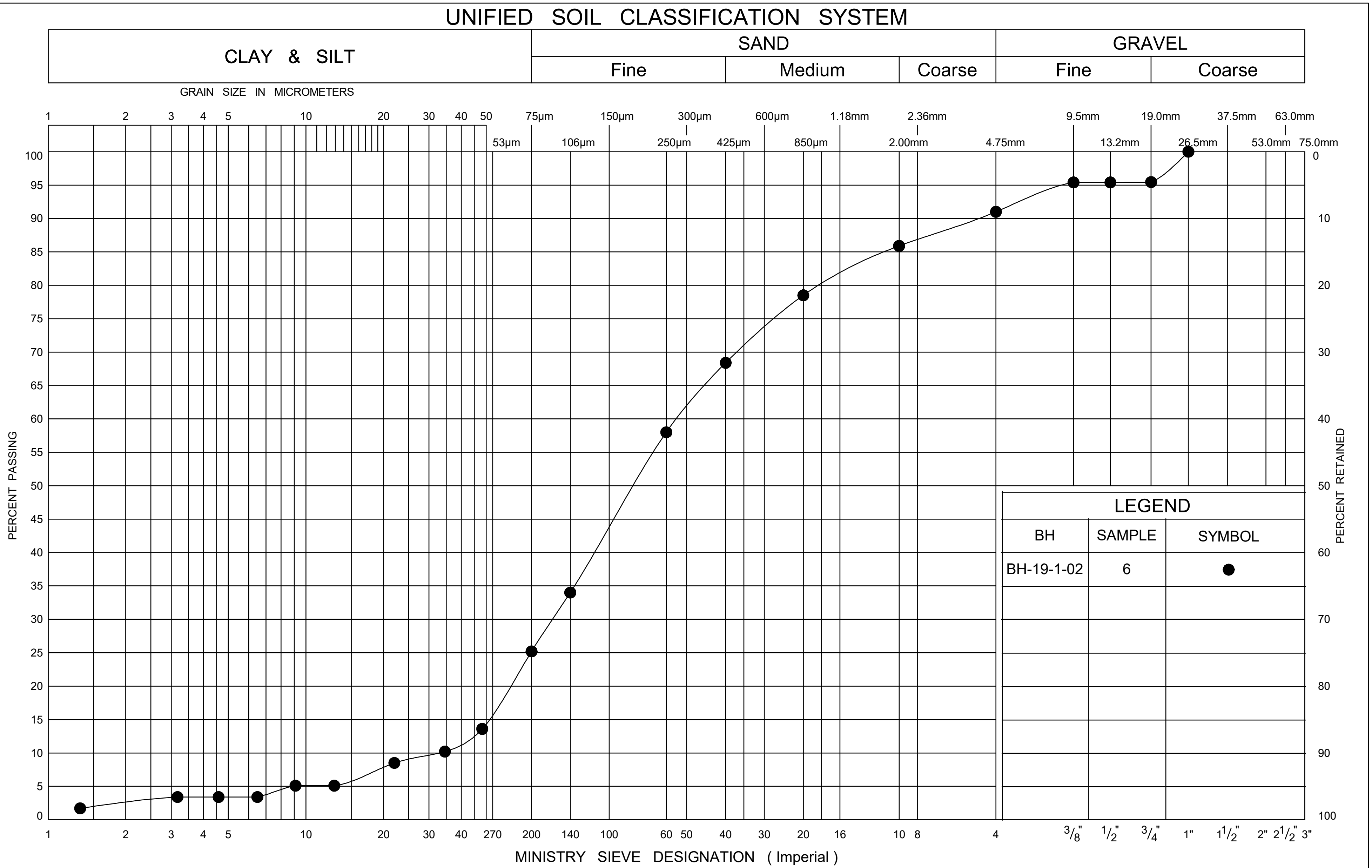
AW ONTARIO GRAIN SIZE LEFT LEGEND 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MOT.GDT 17/1/20

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Gravelly with Sand / Sand and Gravel / Sandy Gravel, Trace Silt, Trace Clay

FIG No.:	GSA-1-1
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00

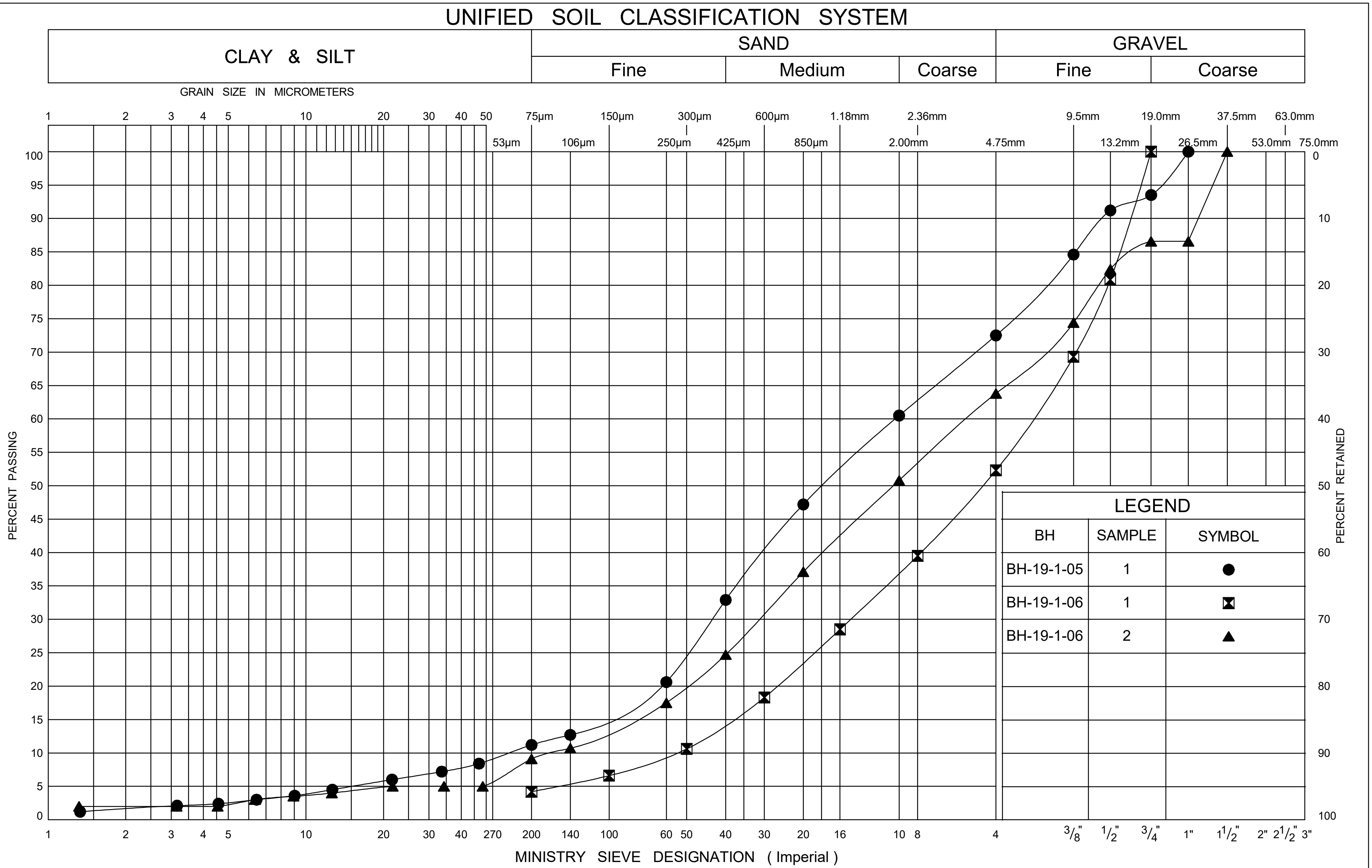


Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Sand with Silt, Trace Gravel, Trace Clay

FIG No.:	GSA-1-2
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00



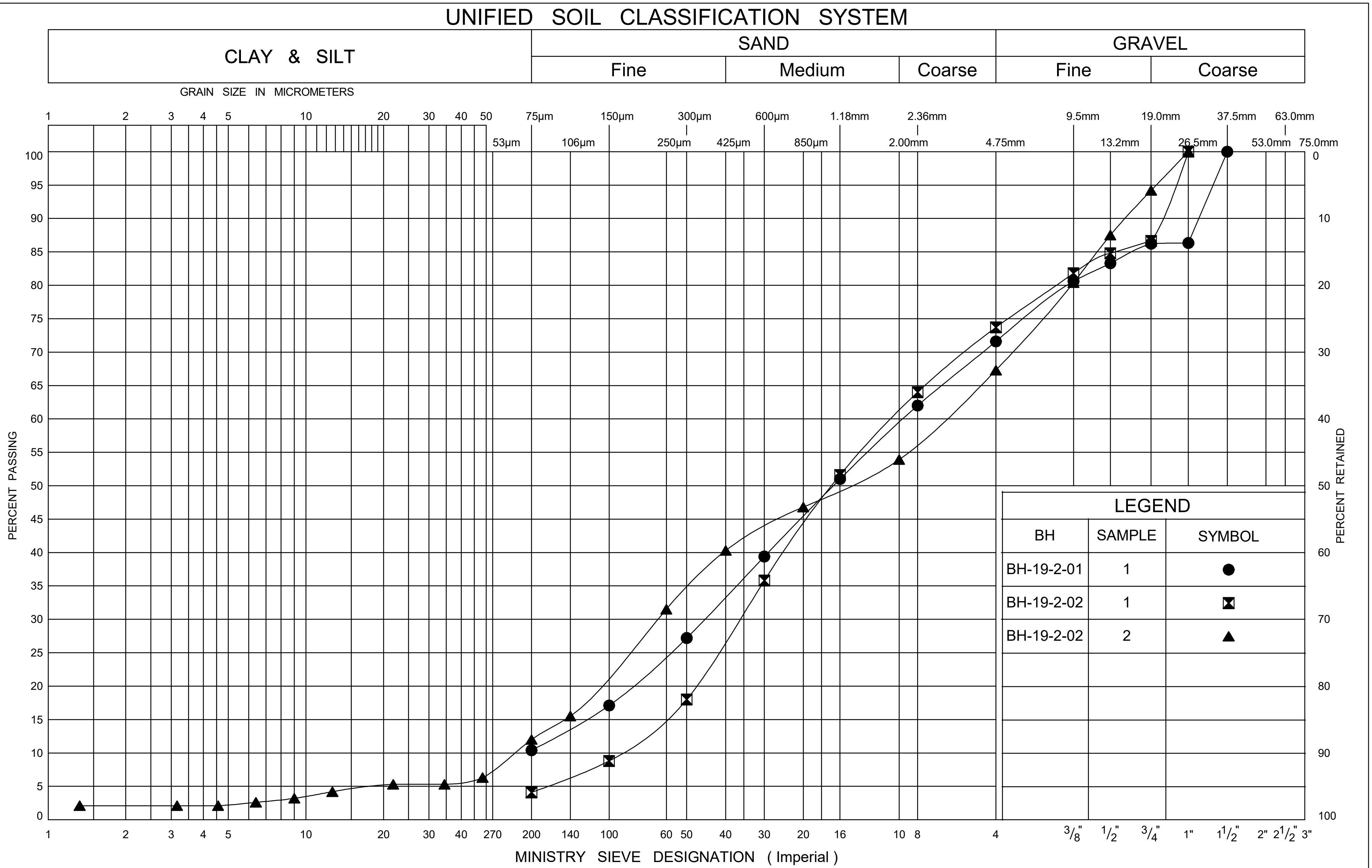
Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Fill - Sand with Gravel to Sand and Gravel, Trace Silt, Trace Clay, Trace Cobbles

FIG No.:	GSA-1-3
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00

ONTARIO MOT GRAIN SIZE 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MOT.GDT 17/1/20



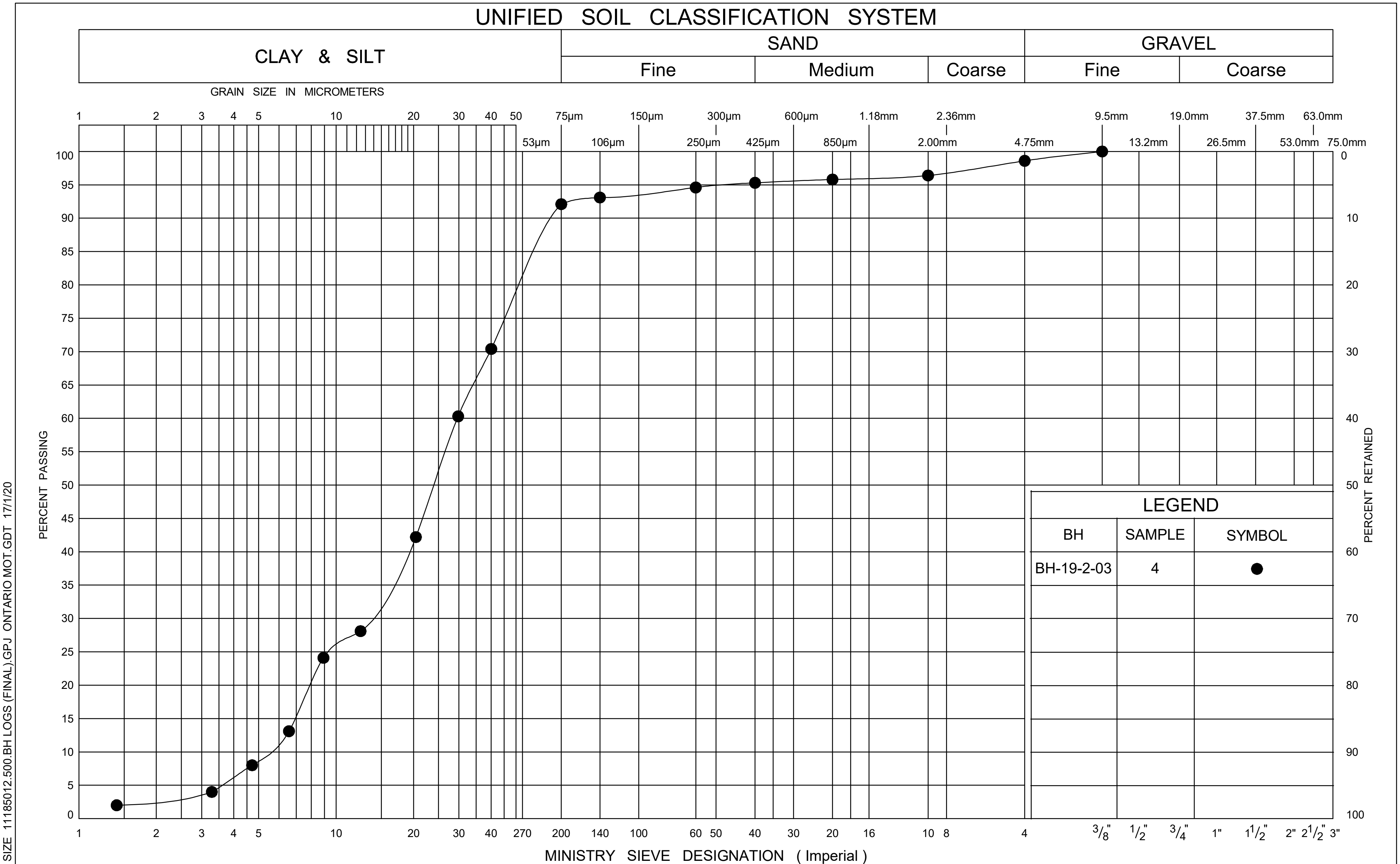
ONTARIO MOT GRAIN SIZE 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MOT.GDT 17/1/20

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Gravelly Sand / Sand with Gravel, Trace to Some Fines

FIG No.:	GSA-2-1
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00

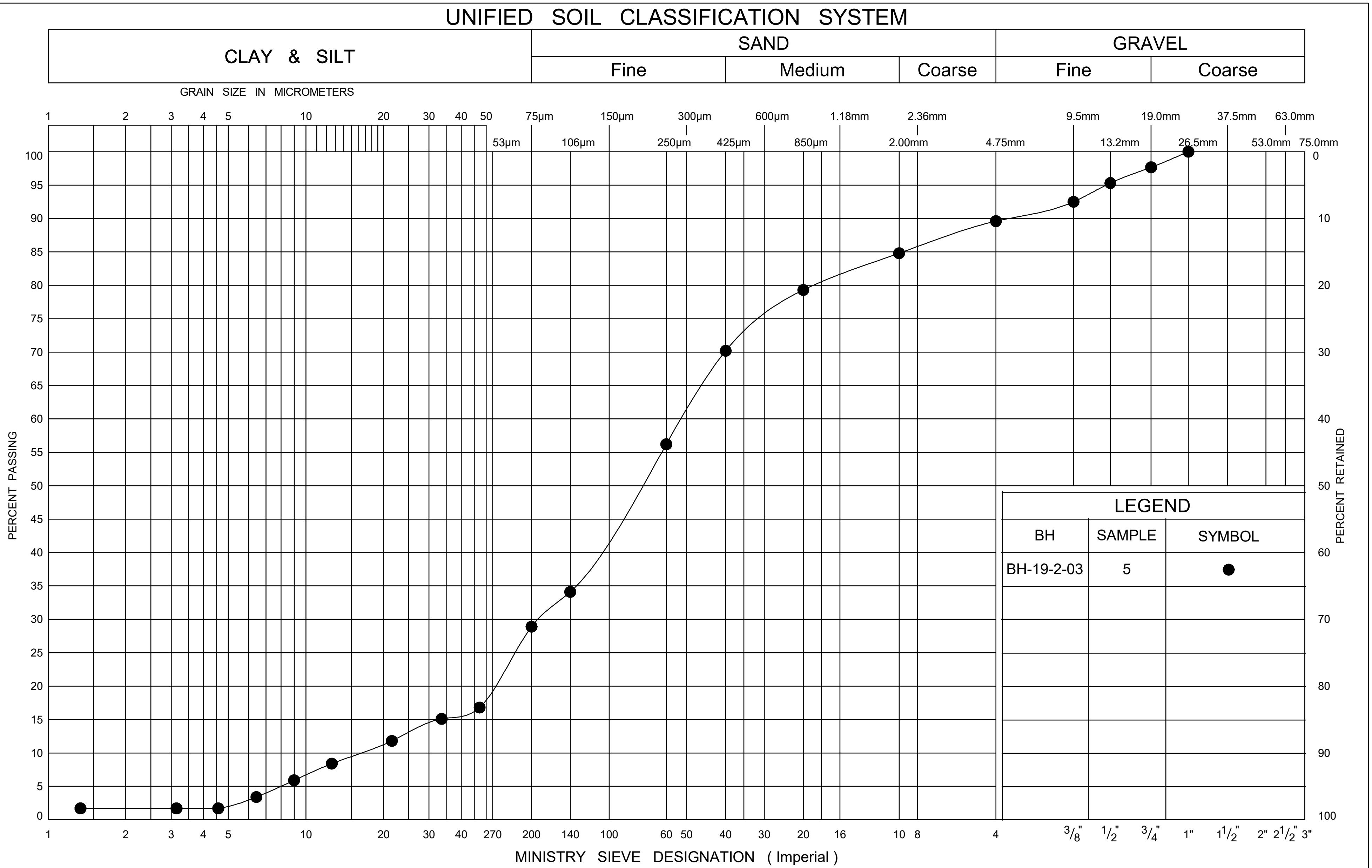


Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Silt, Trace Gravel, Trace Sand, Trace Clay

FIG No.:	GSA-2-2
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00



ONTARIO MOT GRAIN SIZE 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MOT.GDT 17/1/20

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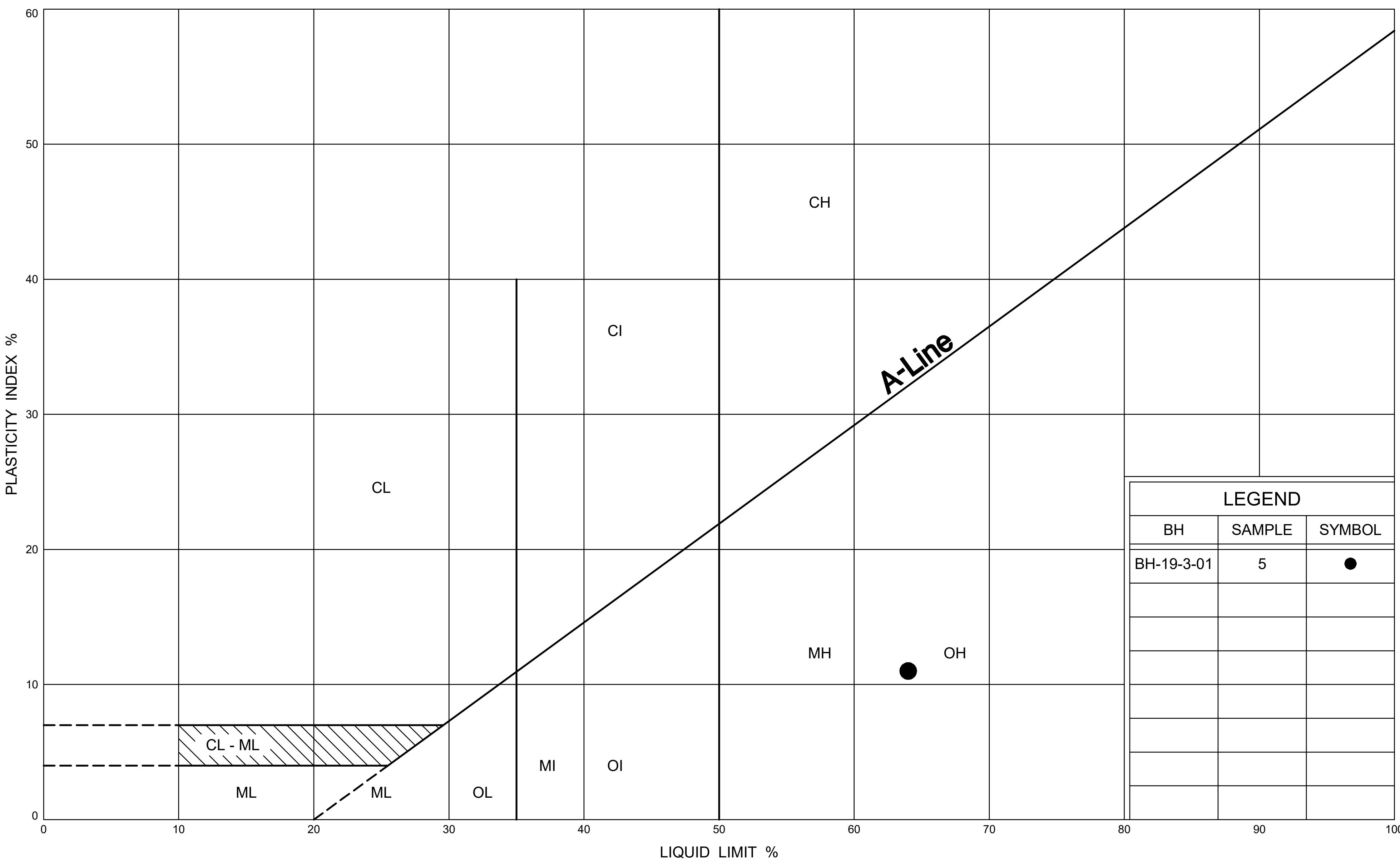
Ontario

GRAIN SIZE DISTRIBUTION

Sand with Silt, Trace to Some Gravel, Trace Clay

FIG No.:	GSA-2-3
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00



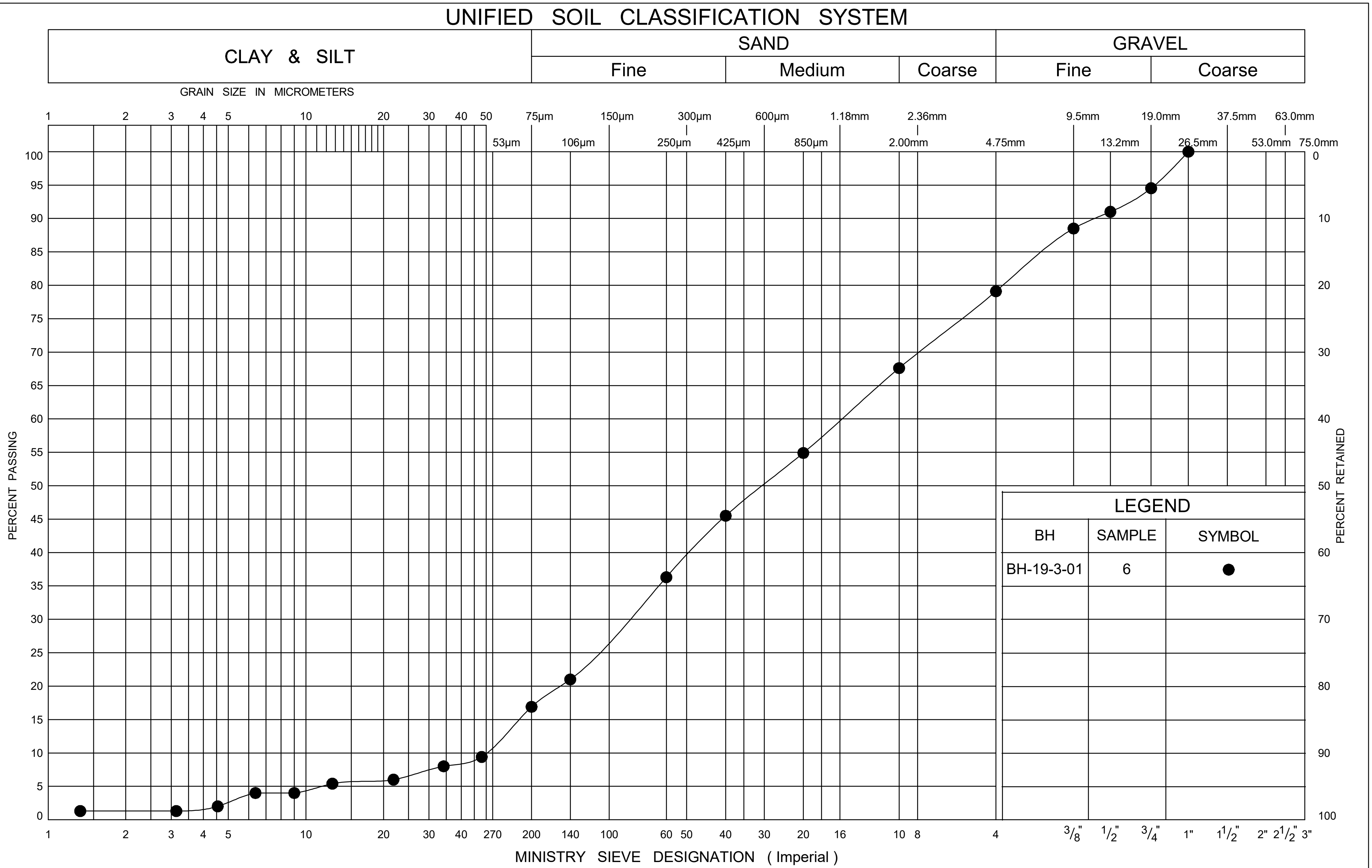
Ministry of Transportation
Ontario

PLASTICITY CHART

Silty Clay

FIG No.:	PC-3-1
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00

ONTARIO MOT PLASTICITY CHART 11185012.500.BH LOGS.GPJ ONTARIO MOT.GDT 7/1/20



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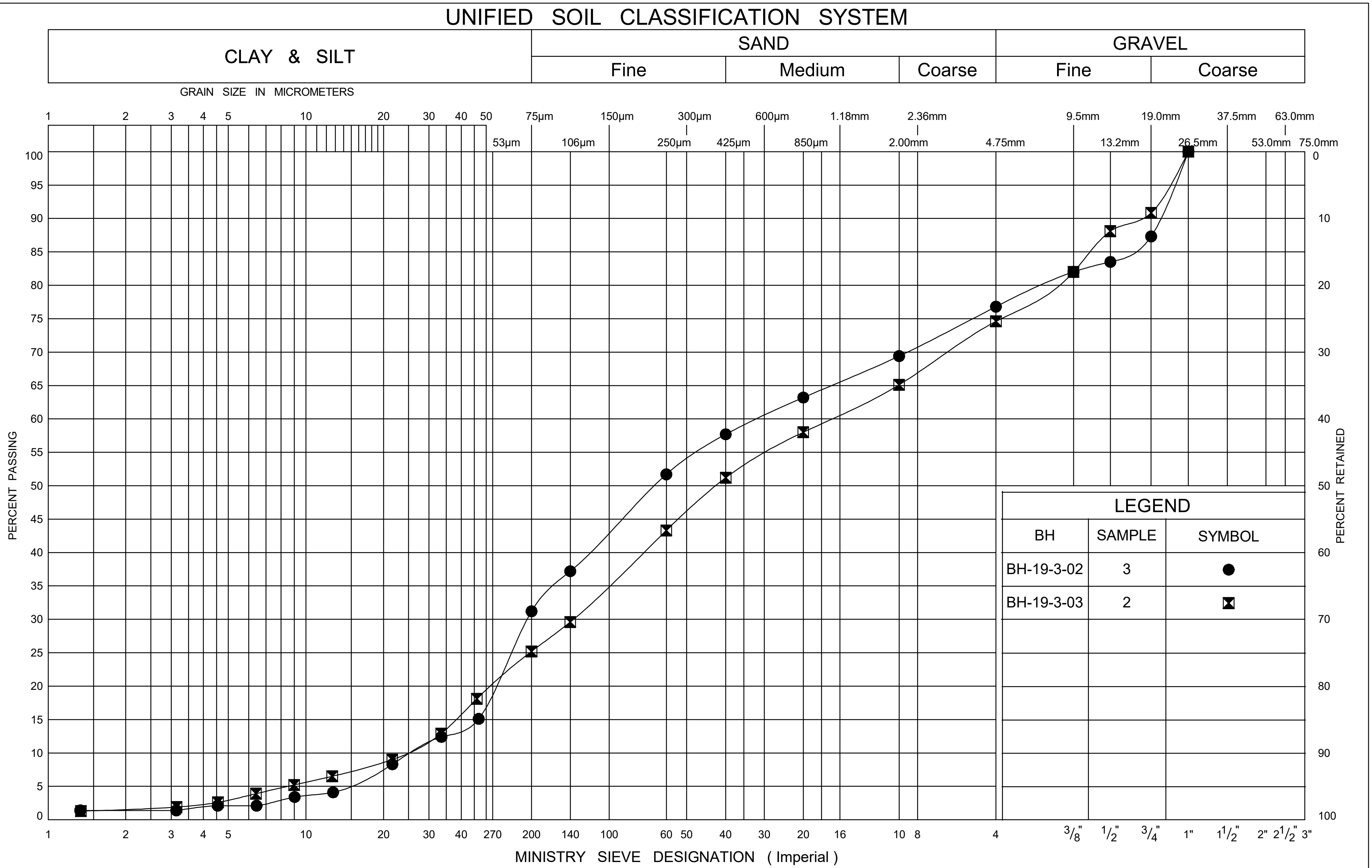
Ontario

GRAIN SIZE DISTRIBUTION

Sand with Gravel, Some Silt, Trace Clay

FIG No.:	GSA-3-1
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00

ONTARIO MOT GRAIN SIZE 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MOT.GDT 17/1/20



ONTARIO MOT GRAIN SIZE 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MOT.GDT 17/1/20

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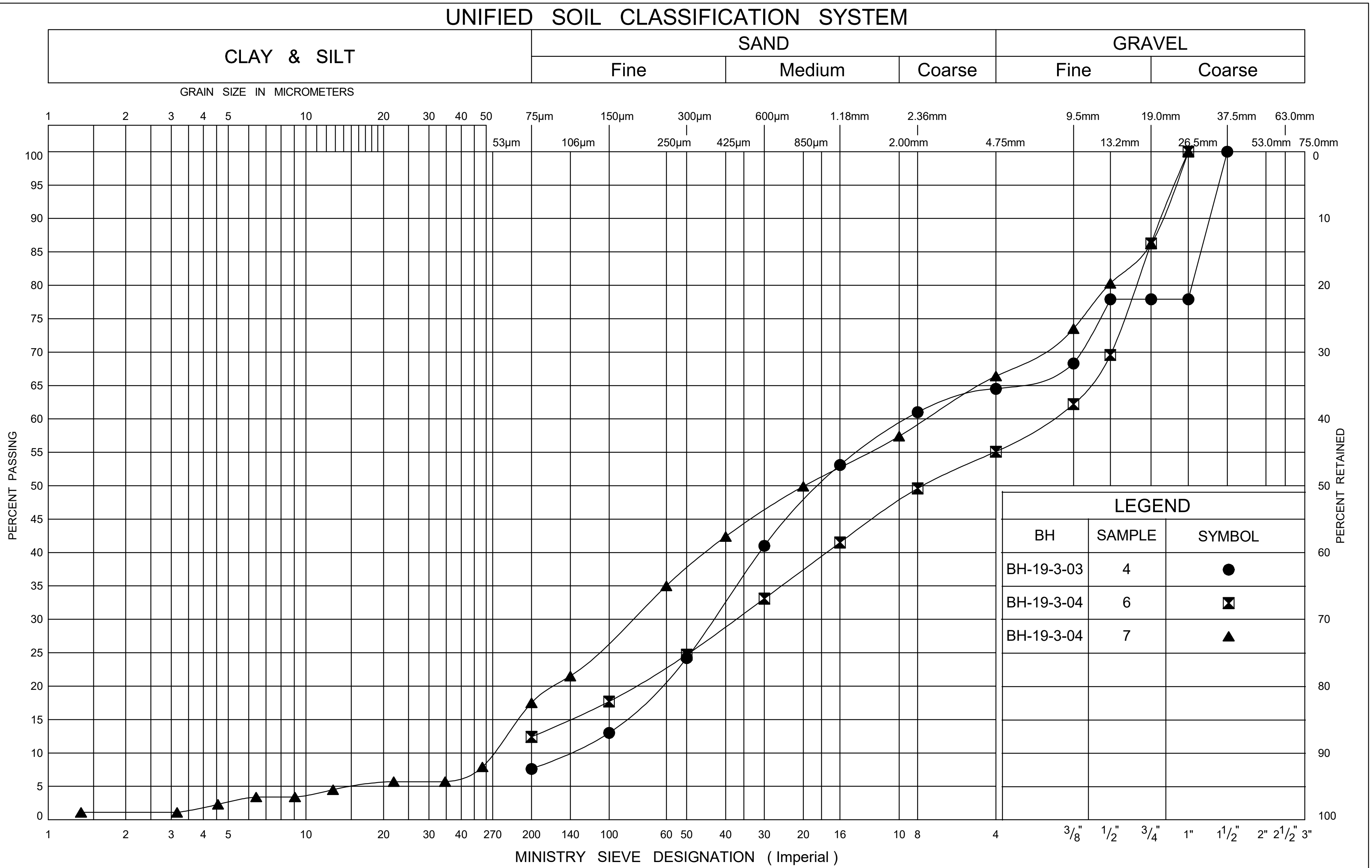
Ontario

GRAIN SIZE DISTRIBUTION

Silty Sand with Gravel, Trace Clay

FIG No.:	GSA-3-2
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00



Ministry of Transportation
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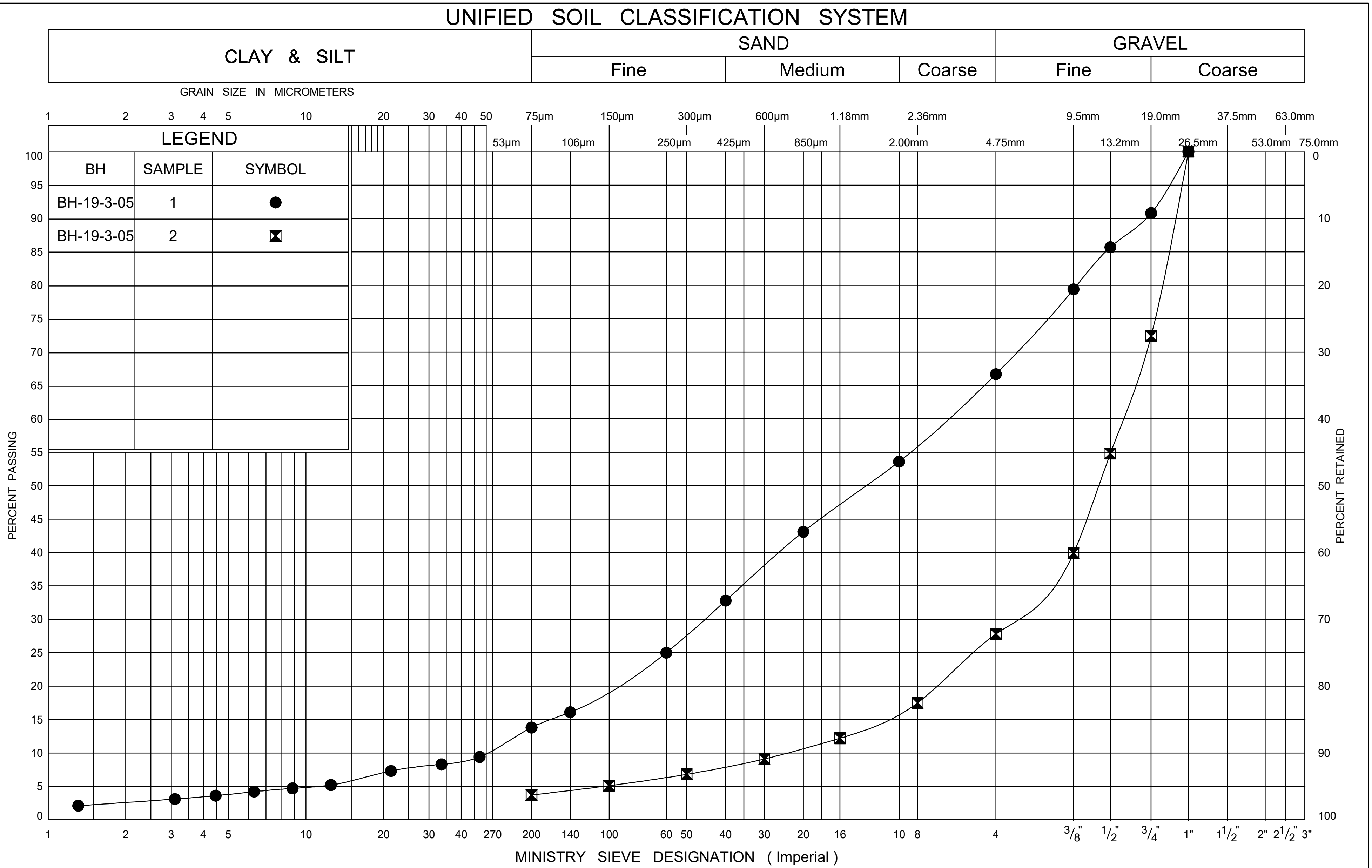
GRAIN SIZE DISTRIBUTION

Gravelly Sand, Trace to Some Silt, Trace Clay

FIG No.:	GSA-3-3
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00

ONTARIO MOT GRAIN SIZE 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MOT.GDT 17/1/20



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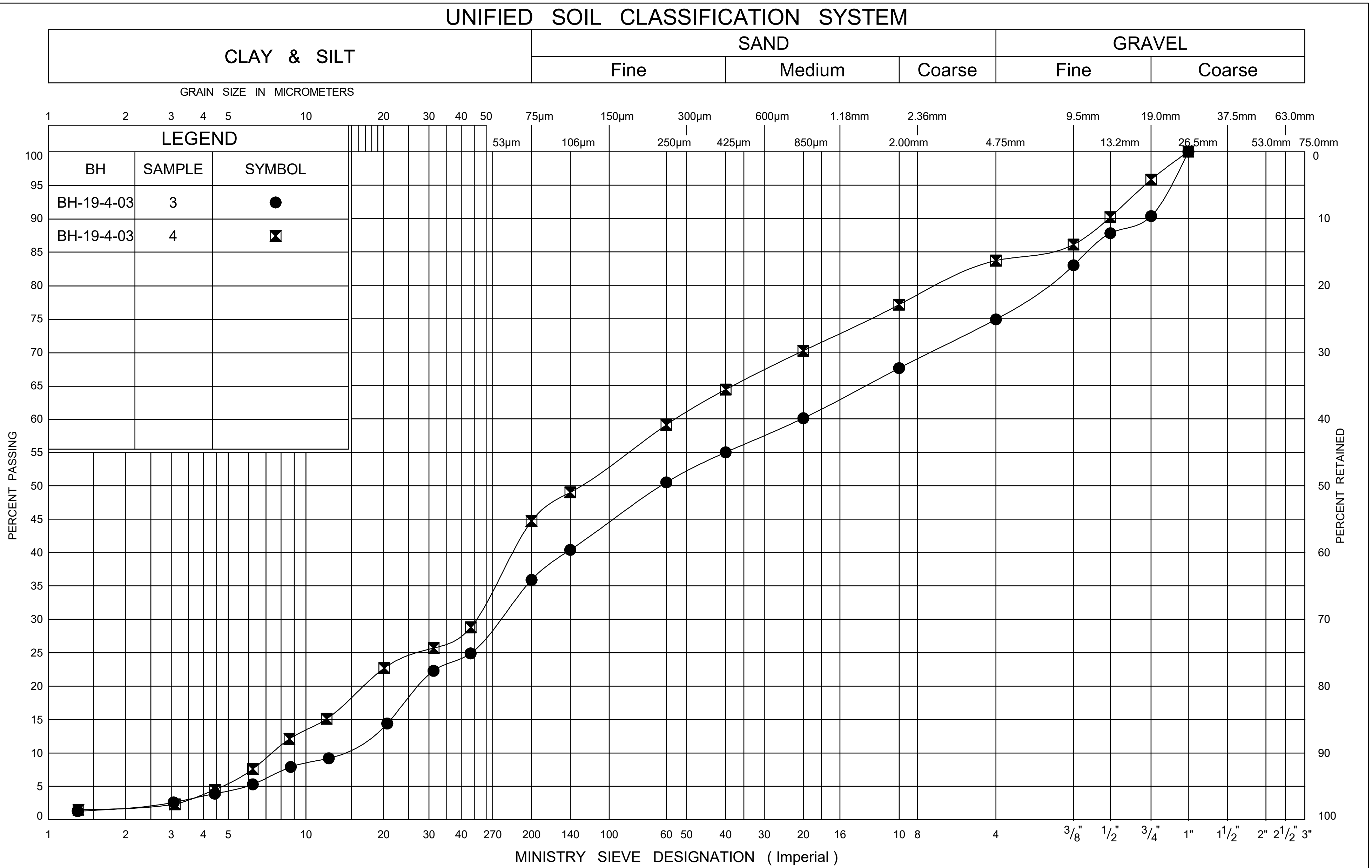
Ontario

GRAIN SIZE DISTRIBUTION

Fill - Gravelly Sand to Sandy Gravel, Some Silt, Trace Clay, Trace Cobbles

FIG No.:	GSA-3-4
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00



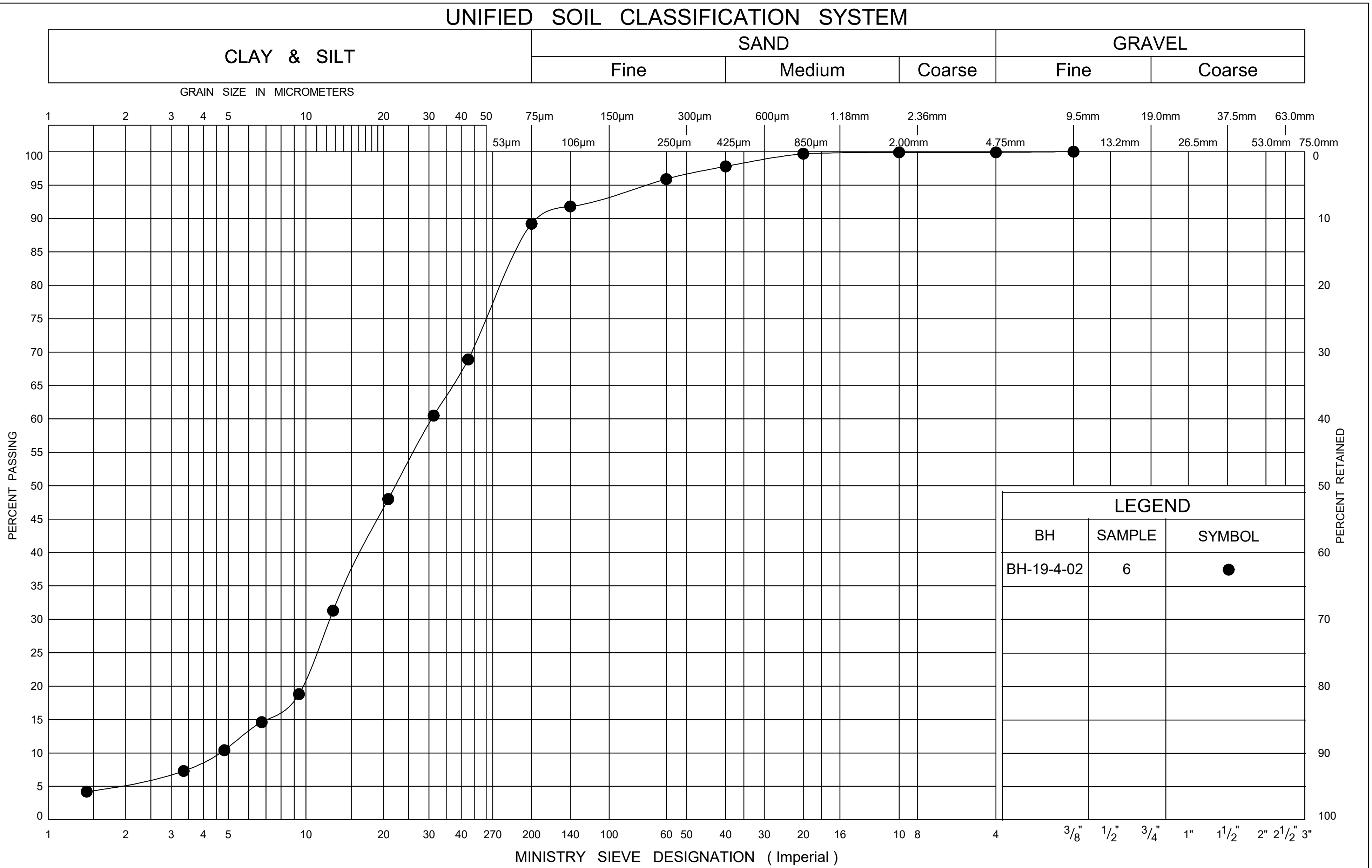
Ministry of Transportation Ontario	

GRAIN SIZE DISTRIBUTION

Silty Sand to Sandy Silt, Trace to Some Gravel, Trace Clay

FIG No.:	GSA-4-1
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00

AW ONTARIO GRAIN SIZE LEFT LEGEND 11185012.500.BH LOGS.GPJ ONTARIO MOT.GDT 7/1/20



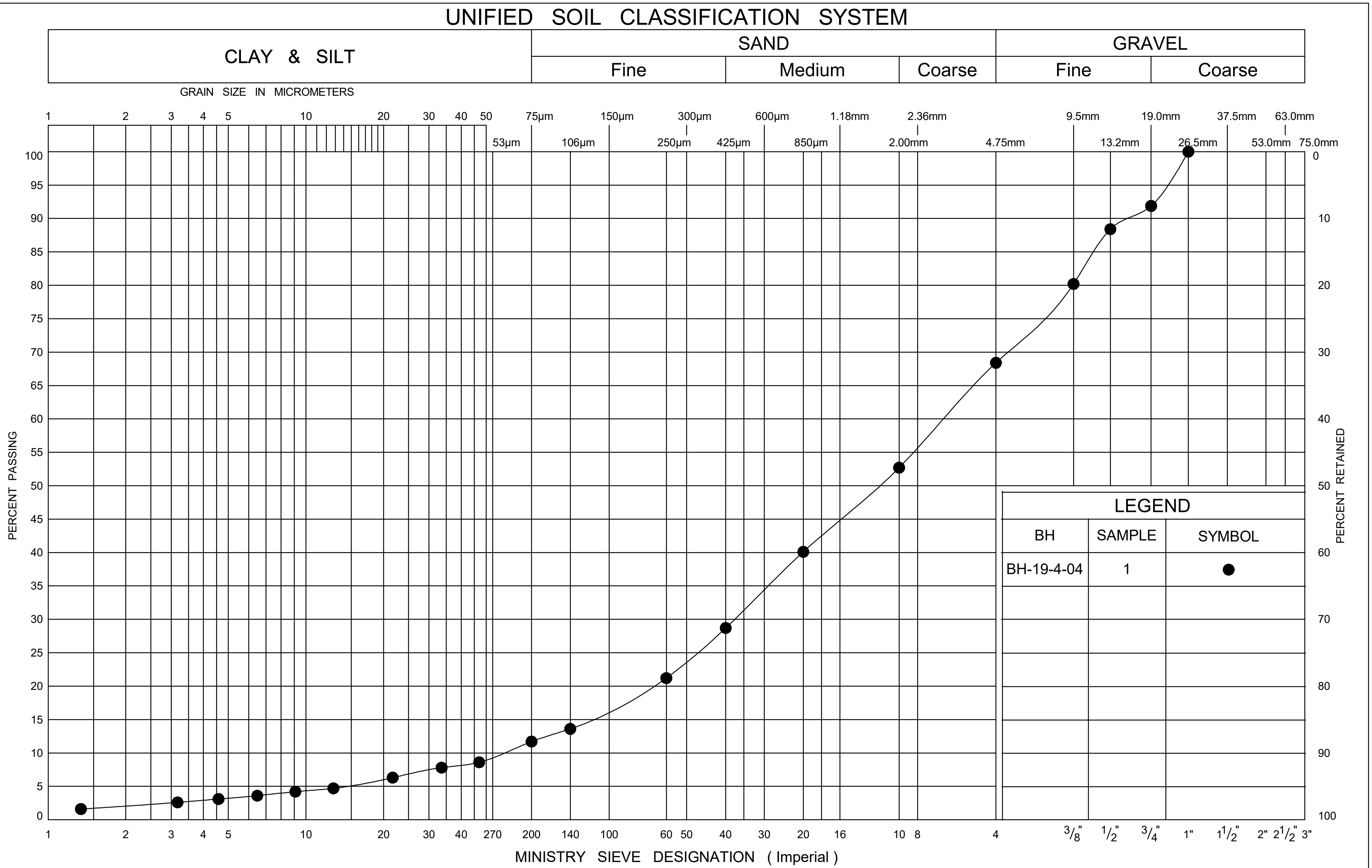
ONTARIO MOT GRAIN SIZE 11185012.500.BH LOGS.GPJ ONTARIO MOT.GDT 7/1/20

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Silt, Some Sand, Trace Clay

FIG No.:	GSA-4-2
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00



ONTARIO MOT GRAIN SIZE 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MOT.GDT 17/1/20

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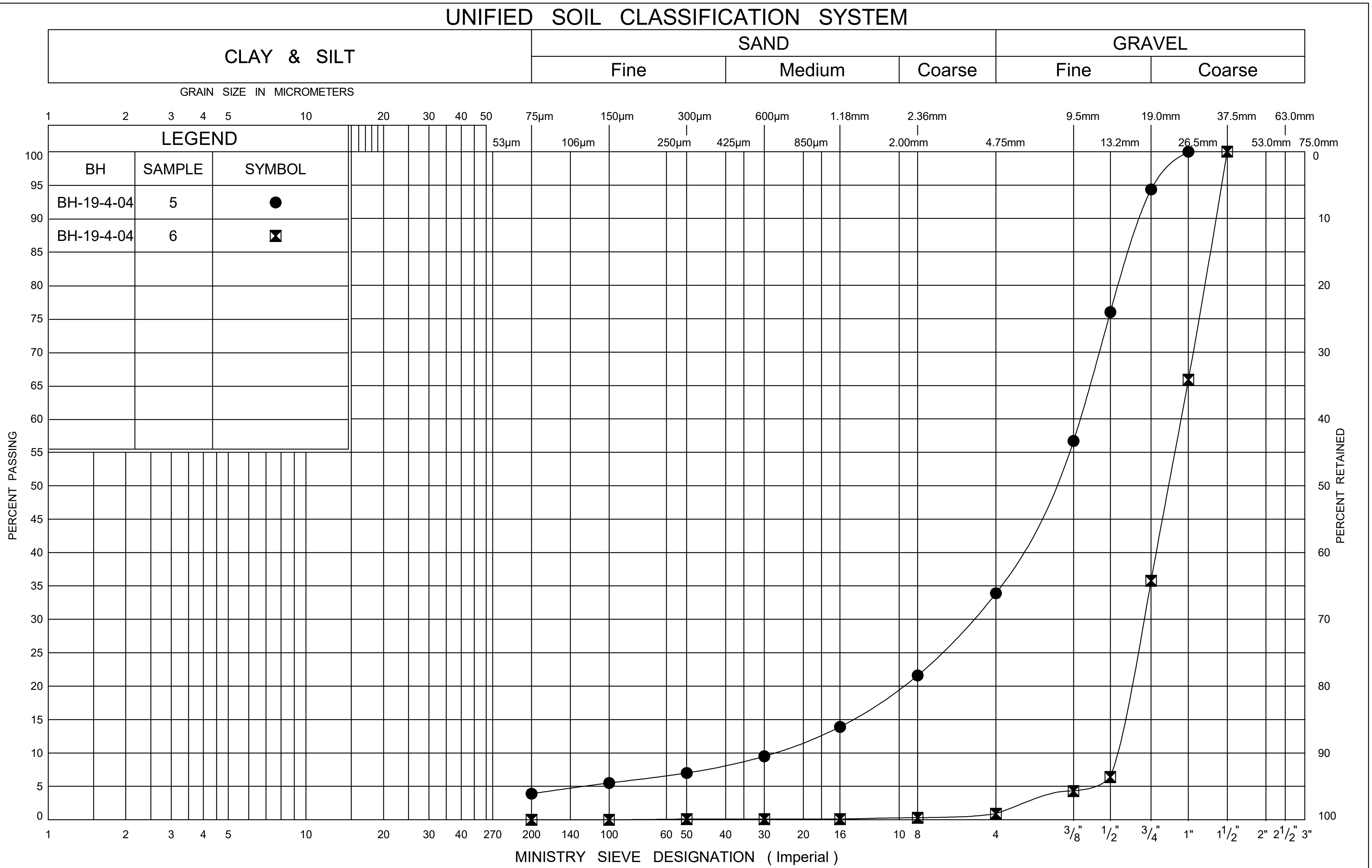
Ontario

GRAIN SIZE DISTRIBUTION

Fill - Gravelly Sand, Some Silt, Trace Clay, Trace Cobbles

FIG No.:	GSA-4-3
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00



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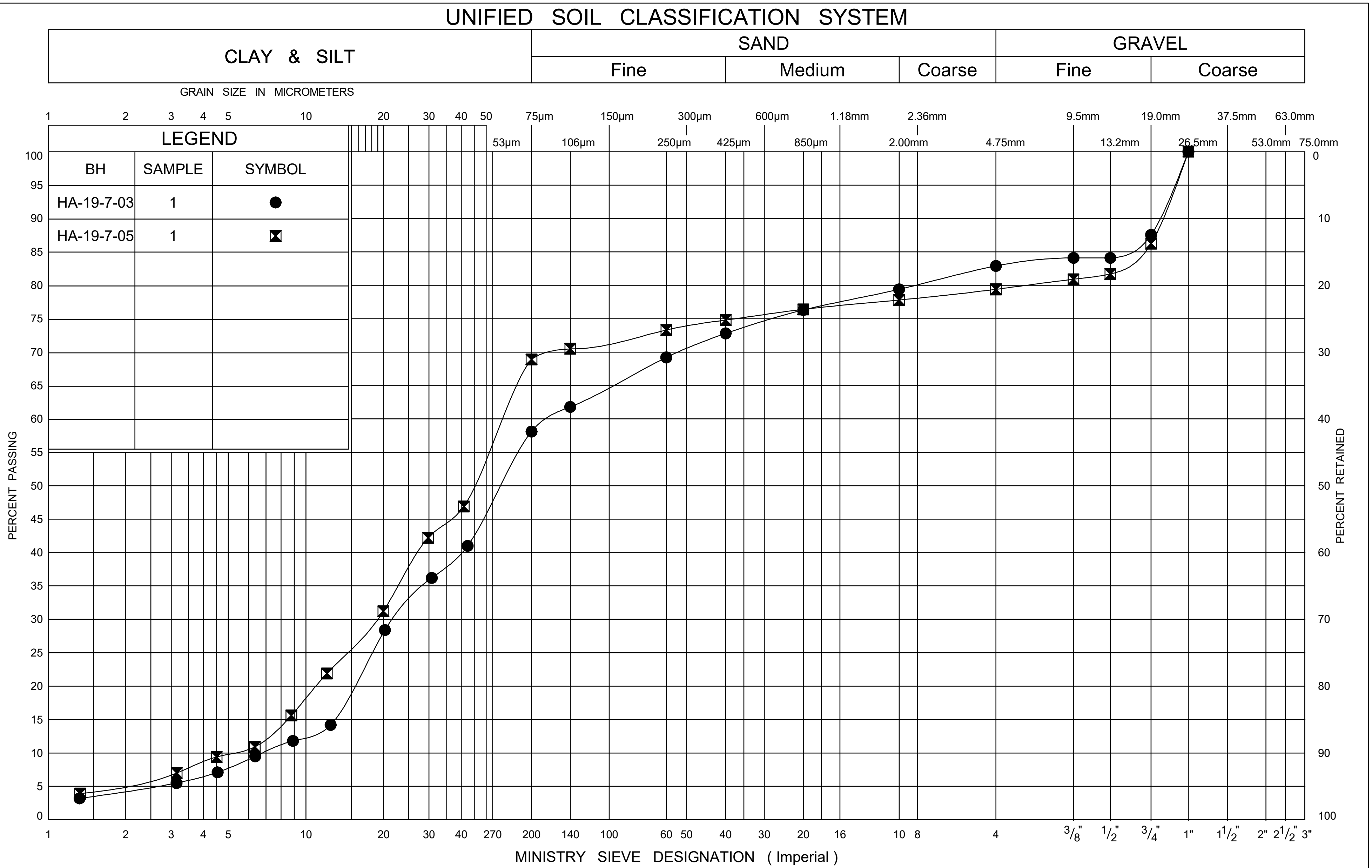
GRAIN SIZE DISTRIBUTION

Sandy Gravel to Gravel, Some Cobbles

FIG No.:	GSA-4-4
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00

AW ONTARIO GRAIN SIZE LEFT LEGEND 11185012.500.BH LOGS.GPJ ONTARIO MOT.GDT 7/1/20



AW ONTARIO GRAIN SIZE LEFT LEGEND 11185012.500.BH LOGS (FINAL).GPJ ONTARIO MOT.GDT 17/1/20

Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

Sandy Silt / Silt with Sand, Trace to Some Gravel, Trace Clay

FIG No.:	GSA-7-1
Agreement No.:	5017-E-0043
Work Item No.:	5
G.W.P. No.:	5031-18-00

Appendix D
Rock Core Photographs, Rock Core Description, and Uniaxial Compressive Strength Test Results

Photo 19 – Highway 17 - Area 1 - BH-19-1-02, R1 = 5.0 m – 5.4 m, R2 = 5.4 m – 6.2 m, R3 = 6.2 m – 7.3 m, and R4 = 7.3 m – 8.3 m

Photo 20 – Highway 17 – Area 1 - BH-19-1-04, R1 = 3.7 m – 4.4 m, R2 = 4.4 m – 4.7 m, R3 = 4.7 m – 6.2 m, and R4 = 6.2 m – 6.8 m

Photo 21 – Highway 17 – Area 1 - BH-19-1-05, R1 = 2.9 m – 4.3 m, and R2 = 4.3 m – 5.9 m

Table D1 – Area 1 Rock Core Description

Borehole No.	Core Run	Depth (m)	% REC	% RQD	Description
BH-19-1-02	1	5.0 – 5.4	100	62	**GREENSTONE.** Dark green to black in color with white quartz and plagioclase crystals, fine to medium grained, weakly to moderately foliated, slightly weathered and moderately to highly fractured with no fillings, medium to hard (based on geological hammer and knife test). Intact Rock Strength (IRS): R3 (Medium Strong Rock). Visual Petrography: Quartz, Plagioclase
	2	5.4 – 6.2	88	51	
	3	6.2 – 7.3	100	85	
	4	7.3 – 8.3	100	82	
BH-19-1-04	1	3.7 – 4.4	100	92	**GREENSTONE.** Dark green to black in color with white quartz and plagioclase crystals, fine to medium grained, weakly to moderately foliated, slightly weathered and moderately fractured with no fillings, medium to hard (based on geological hammer and knife test). Intact Rock Strength (IRS): R4 (Strong Rock). Visual Petrography: Quartz, Plagioclase
	3	4.4 – 4.7	100	89	
	3	4.7 – 6.2	100	88	
	4	6.2 – 6.8	100	96	
BH-19-1-05	1	2.9 – 4.3	100	73	**GREENSTONE.** Dark green to black in color with white quartz and plagioclase crystals, fine to medium grained, weakly to moderately foliated, slightly weathered and moderately to highly fractured with no fillings, medium to hard (based on geological hammer and knife test). Intact Rock Strength (IRS): R3 (Medium Strong Rock). Visual Petrography: Quartz, Plagioclase
	2	4.3 – 5.9	100	72	

Logged by: Joe Lin and Jun Wang

Reviewed by: Lul Yimam, PhD, P.Eng.

Photo 22 – Highway 17 - Area 2 - BH-19-2-01, R1 = 0.3 m – 1.4 m, R2 = 1.4 m – 3.0 m, R3 = 3.0 m – 3.7 m

Photo 23 – Highway 17 – Area 2 - BH-19-2-02, R1 = 1.4 m – 2.9 m, R2 = 2.9 m – 4.4 m, R3 = 4.4 m – 5.0 m

Table D2 – Area 2 Rock Core Description

Borehole No.	Core Run	Depth (m)	% REC	% RQD	Description
BH-19-2-01	1	0.3 – 1.4	98	67	**GREENSTONE.** Dark green to black in color with white quartz and plagioclase crystals, fine to medium grained, weakly to moderately foliated, slightly weathered and moderately fractured with no fillings, medium to hard (based on geological hammer and knife test). Intact Rock Strength (IRS): R4 (Strong Rock). Visual Petrography: Quartz, Plagioclase
	2	1.4 – 3.0	100	81	
	3	3.0 – 3.7	100	97	
BH-19-2-02	1	1.4 – 2.9	93	33	**GREENSTONE.** Dark green to black in color with white quartz and plagioclase crystals, fine to medium grained, weakly to moderately foliated, slightly weathered and moderately to highly fractured with no fillings, medium to hard (based on geological hammer and knife test). Intact Rock Strength (IRS): R3 (Medium Strong Rock). Visual Petrography: Quartz, Plagioclase
	2	2.9 – 4.4	99	90	
	3	4.4 – 5.0	100	84	

Logged by: Jun Wang

Reviewed by: Lul Yimam, PhD, P.Eng.

Photo 24 – Highway 17 - Area 3 - BH-19-3-02, R1 = 2.3 m – 2.8 m, R2 = 2.8 m – 4.5 m, R3 = 4.5 m – 5.7 m

Photo 25 – Highway 17 – Area 3 - BH-19-3-04, R1 = 5.8 m – 6.1 m, R2 = 6.1 m – 7.6 m, R3 = 7.6 m – 9.1 m

Table D3 – Area 3 Rock Core Description

Borehole No.	Core Run	Depth (m)	% REC	% RQD	Description
BH-19-3-02	1	2.3 – 2.8	100	97	**GREENSTONE.** Dark green to black in color with white quartz and plagioclase crystals, fine to medium grained, weakly to moderately foliated, slightly weathered and moderately fractured with no fillings, medium to hard (based on geological hammer and knife test). Intact Rock Strength (IRS): R4 (Strong Rock). Visual Petrography: Quartz, Plagioclase
	2	2.8 – 4.5	100	83	
	3	4.5 – 5.7	100	100	
BH-19-3-04	1	5.8 – 6.1	100	90	**GREENSTONE.** Dark green to black in color with white quartz and plagioclase crystals, fine to medium grained, weakly to moderately foliated, slightly weathered and moderately to highly fractured with no fillings, medium to hard (based on geological hammer and knife test). Intact Rock Strength (IRS): R3 (Medium Strong Rock). Visual Petrography: Quartz, Plagioclase
	2	6.1 – 7.6	100	78	
	3	7.6 – 9.1	100	68	

Logged by: Jun Wang

Reviewed by: Lul Yimam, PhD, P.Eng.

Photo 25 – Highway 17 – Area 4 - BH-19-4-02, R1 = 7.2 m – 7.8 m, R2 = 7.8 m – 9.8 m, R3 = 9.8 m – 10.8 m

Table D4 – Area 4 Rock Core Description

Borehole No.	Core Run	Depth (m)	% REC	% RQD	Description
BH-19-4-02	1	7.2 – 7.8	100	87	**GREENSTONE.** Dark green to black in color with white quartz and plagioclase crystals, fine to medium grained, weakly to moderately foliated, slightly weathered and moderately fractured with no fillings, medium to hard (based on geological hammer and knife test). Intact Rock Strength (IRS): R4 (Strong Rock). Visual Petrography: Quartz, Plagioclase
	2	7.8 – 9.8	100	99	
	3	9.8 – 10.8	100	100	

Logged by: Moe Nasir

Reviewed by: Lul Yimam, PhD, P.Eng.

Uniaxial Compressive Strength of Intact Rock Core Specimens
(ASTM D7012 - Method C)

CLIENT:	Ministry of Transportation	**LAB No.:**	WLT 276-1
PROJECT/ SITE:	MTO Northeast Region Pavement	**PROJECT No.:**	11185012-500
Borehole No.:	BH-19-2-02	**Location:**	Hwy 17, Between White River and Wawa
Depth:	4.42-4.60 m (14'6"-15'1")	**Date Sampled:**	2019.09.28
Lithological Description:	Greenstone		

Initial Specimen Parameters	
Diameter, cm	4.7
Height, cm	10.0
Height-to-Diameter Ratio	2.1
Volume, cm^3	174.3
Mass, g	526.8
Bulk Density, kg/m^3	3023
Moisture Condition	As Received
Moisture Content, %	-----

Maximum Applied Load, kN	346.3
Compressive Strength, MPa	199.3

REMARKS:

PERFORMED BY:	M.Mitchell	**DATE:**	October 17, 2019
VERIFIED BY:	Michael Braverman	**DATE:**	October 21, 2019

Uniaxial Compressive Strength of Intact Rock Core Specimens
(ASTM D7012 - Method C)

CLIENT: Ministry of Transportation **LAB No.:** WLT 276-2

PROJECT/ SITE: MTO Northeast Region Pavement **PROJECT No.:** 11185012-500

Borehole No.: BH-19-3-04 **Location:** Hwy 17, Between White River and Wawa

Depth: 5.92-6.17 m (19'5"-20'3") **Date Sampled:** 2019.09.26

Lithological Description: Greenstone

Initial Specimen Parameters	
Diameter, cm	4.7
Height, cm	9.8
Height-to-Diameter Ratio	2.1
Volume, cm^3	170.6
Mass, g	487.1
Bulk Density, kg/m^3	2855
Moisture Condition	As Received
Moisture Content, %	-----

Maximum Applied Load, kN	136.4
Compressive Strength, MPa	78.6

REMARKS:

PERFORMED BY: M.Mitchell **DATE:** October 17, 2019

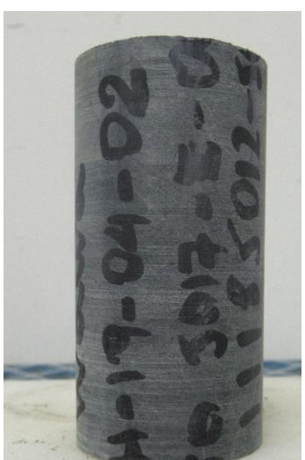
VERIFIED BY: Michael Braverman **DATE:** October 21, 2019

Uniaxial Compressive Strength of Intact Rock Core Specimens
(ASTM D7012 - Method C)

CLIENT:	Ministry of Transportation	**LAB No.:**	WLT 276-3
PROJECT/ SITE:	MTO Northeast Region Pavement	**PROJECT No.:**	11185012-500
Borehole No.:	BH-19-4-02	**Location:**	Hwy 17, Between White River and Wawa
Depth:	7.62-7.80 m (25'-25'7")	**Date Sampled:**	2019.10.02
Lithological Description:	Greenstone		

Initial Specimen Parameters	
Diameter, cm	4.7
Height, cm	10.0
Height-to-Diameter Ratio	2.1
Volume, cm^3	176.9
Mass, g	534.1
Bulk Density, kg/m^3	3019
Moisture Condition	As Received
Moisture Content, %	-----

Maximum Applied Load, kN	210.9
Compressive Strength, MPa	119.5


BH19 04-02


BH19 04-02

REMARKS:

PERFORMED BY:	M.Mitchell	**DATE:**	October 17, 2019
VERIFIED BY:	Michael Braverman	**DATE:**	October 21, 2019

Part B - Foundation Design Report

HIGHWAY 17 WIDENING
AREA 1 (STA. 10+400 – 10+600), AREA 2 (STA. 11+050 – 11+160)
AREA 3 (STA. 11+525 – 11+600), AREA 4 (STA. 12+350 – 12+480)
TOWNSHIP OF DAMBROSSIO
AREA 7 (STA. 16+700 – 16+825)
TOWNSHIP OF LALIBERT
DISTRICT OF ALGOMA
BETWEEN WHITE RIVER AND WAWA, ONTARIO

AGREEMENT NO. 5017-E-0043
WORK ITEM: #5
GWP 5031-18-00

Distribution:

- One (1) Digital Copy (PDF) and One (1) Hard Copy: Project Manager, Ministry of Transportation, Ontario
- One (1) Digital Copy (PDF) and One (1) Hard Copy: Foundations, Ministry of Transportation, Ontario
- One Hard Copy: GHD Limited, Whitby, Ontario

GEOCRES No.: 42C-047

GHD Ref.: 11185012-500

February 3, 2020

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Part B – Foundation Design Report

PART B - FOUNDATION DESIGN REPORT

Highway 17 Widening
Area 1 (Sta. 10+400 – 10+600), Area 2 (Sta. 11+050 – 11+160)
Area 3 (Sta. 11+525 – 11+600), Area 4 (Sta. 12+350 – 12+480), Township of Dambrossio
Area 7 (Sta. 16+700 – 16+825), Township of Lalibert,
District of Algoma, Between White River and Wawa, Ontario
Agreement No 5017-E-0043, Work Item: #5
G.W.P 5031-18-00

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7. Introduction

This Foundation Design Report (FDR) provides foundation design recommendations for temporarily and permanent widening of road embankments located in five different areas (Area 1, Area 2, Area 3, Area 4 and Area 7) along Highway 17, between the Towns of White River and Wawa, Ontario. The geotechnical inputs required for design of the embankments and preparation of construction documents are provided based on the findings given in Part A - Foundation Investigation Report (FIR). The inputs are based on the subsurface and groundwater conditions encountered at borehole locations and assumes that the materials are representative of the overall site conditions.

This report is intended for the use of the Ministry of Transportation of Ontario Northeastern Region (Ministry or MTO) for the purpose of designing road embankments in the above mentioned areas. It shall not be used or relied upon for any other purposes or locations, or by any other parties including construction or design-build contractors. Where comments are made in this report on construction, they are provided only to highlight aspects, which could affect the design of the project. Contractors bidding on or undertaking any work in the specified areas should examine the factual results or findings of the project, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data given in Part A, as it affects their proposed construction techniques, equipment capabilities, costs, sequencing, and other related issues. Comments or recommendations pertaining to construction should not be construed as instructions to contractors.

This project didn't include an environmental assessment, and the information contained herein must in no way be construed as an opinion of the environmental conditions of the project sites.

8. Project Description

8.1 Existing Embankments

The year of construction of Highway 17 in the region or any effort of rehabilitation was unknown to GHD. However, it is assumed that the highway and the road embankments at the specified five (5) areas have been in service for a significant period of time. Based on the borehole information and field observation, the embankments in all areas consisted of blasted rock fill overlain by a fill of granular materials. All embankments were constructed either as creek or lake crossings.

In Area 1, the existing road embankment consists of a granular fill underlain by rock fill. The presence of a bedrock beneath the rock fill was confirmed through rock coring at a depth of 2.9 m (El. 438.3 m), indicating that part or the entire road embankment was constructed on a layer of bedrock. Based on the survey information obtained from Ministry, the existing slopes on both the north and south sides vary from 1.25H:1V to 1.5H:1V or flatter. However, the existing rock fill embankments are thought to be constructed at 1.25H:1V. The slopes are protected by large boulders of rocks and surplus excavated material, and showed no signs of sloughing or failure. Because of this, the effect of rill erosion was limited to exposed slope surfaces. In addition, the performance of the pavement structure was found to be good with no indications of settlement or cracking on riding surfaces.

In Area 2, no borehole was drilled through the embankment. However, the assumption is that the embankment consists of rock fill like any other embankments in the region. The existing slopes ranged from 1.25H:1V and 1.5H:1V. However, the main road embankment should be at 1.25H:1V. No signs of distress, cracking or failure were observed on these slopes. The performance of the pavement structure was also found to be relatively good with no indications of settlement or cracking.

In Area 3, the existing road embankment consists of a fill of sand and gravel or gravelly sand underlain by rock fill. The presence of a bedrock beneath the rock fill was confirmed through rock coring at a depth of 5.8 m (El. 458.5 m), indicating that part or the entire section of the embankment was constructed on a layer of bedrock. Based on the survey information obtained from the Ministry, the slopes vary from 1.25H:1V and 1.5H:1V, although the embankment was constructed at 1.25H:1V. No signs of sloughing or failure were observed on these slopes, and the effect of rill erosion was limited to those parts of the slopes not protected by boulders. In addition, the performance of the pavement structure was found to be relatively good in that no indications of settlement or cracking were observed on the riding surfaces, except minor wheel track ruts and local erosion on shoulders.

In Area 4, the existing road embankment consists of a granular fill underlain by rock fill. Underneath the rock fill, a layer consisting of sandy gravel with some cobbles was encountered. The presence of a possible bedrock beneath the sandy gravel material was confirmed through a rock coring at a depth of 11.3 m (El. 444.3 m). Large boulders of rocks were placed on existing slopes. The existing slopes have gradient of 1.25H:1V to 1.5H:1V, and shows no signs of distress, sloughing, or failure. In addition, no indications of settlement or cracking were seen on the asphalt surface.

Visual observations in Area 7 indicated that the embankment slopes as well as the pavement surfaces are in good conditions. However, some rock boulders placed on slopes have moved downslope towards the end of the culvert. The survey information obtained from MTO suggested that the existing slopes on both sides of the highway are between 1.25H:1V and 1.5H:1V.

8.2 Proposed Road Widening

GHD has received draft design cross sections from the Ministry for Area 1, Area 2, Area 3 and Area 7, in an email dated December 18, 2019. Based on these cross sections and the revised work order provided to GHD on July 18, 2019, the works in all areas will involve permanent and temporary widening of road embankments. It is assumed that the existing roadway will not be raised in any of the areas, and the embankment widening will be part of the future widening of Highway 17 in the region. In Area 1, Area 2 and Area 3, the embankment widening will be on the south side of the highway. In Area 4, a permanent road widening is considered on the north side. Area 7 will also involve widening on the north side. Generally, all rock fill embankments will be constructed at 1.25H:1V.

The proposed widening in all areas would involve the excavation of unsuitable materials within the limits of widening, subgrade preparation, and the placement of fills to the required grade in accordance with applicable standards. The work is also expected to involve sub-excavating the existing road shoulder, put appropriate road base materials, and tie the existing and new pavement structures. In addition, the embankment widening may require drainage works and erosion control measures.

In Area 1, the review of the draft cross sections indicate a maximum road widening of 3 m. The new embankment will be constructed at a slope of 1.25H:1V. Based on the draft cross sections, utility conflicts would occur at Sta. 10+400, Sta. 10+480, and Sta. 10+540 where Bell poles have been installed. In addition, the proposed widening would extend the Ministry's Right of Way (ROW) from about Sta. 10+500 to the end of the embankment on the east side (Sta. 10+600).

In Area 2, the draft cross sections show a maximum widening of about 2.5 m at a slope of 1.25H:1V, with a utility conflict at Sta. 11+160 and ROW encroachment between Sta. 11+150 and Sta+200.

In Area 3, the draft cross sections indicate a maximum widening of 2.5 m at a slope of 1.25H:1V. The proposed widening will have a utility conflict at Sta. 11+620.

In Area 4, GHD was not provided with the draft cross sections. However, it is understood that the Ministry is currently reviewing different alternatives, from widening the road to 7 m to the option of using staged construction with temporary protection system to replace the existing culvert.

In Area 7, the maximum width of widening is approximately 4 m with 2 m wide mid-height berms. The draft cross sections show no utility conflicts and widening beyond the Ministry delineated ROW.

8.3 Foundation Conditions

The subsurface and groundwater conditions in all areas has been discussed in the Foundation Investigation Report (Part A) of this report. In summary, the depth to the bedrock in all areas were found to be shallow and the existing road embankments were placed directly on rocks.

In Area 1, the subsurface at the proposed widening on the south side of Highway 17, consisted of a 5.3 m thick peat layer, underlain by silty sand, sand with silt, sand with gravel, and sandy gravel materials. The thickness of the granular materials beneath the peat was reported to be 2.8 m. A greenstone bedrock was encountered at depth of 3.7 m (El. 434.0 m) to 5 m (431.2 m). In the middle part of the widening, where a thick peat was encountered, the bedrock depth was found to be 7.9 m (El. 429.2 m). The groundwater level measured in a well installed in the area was 1.5 m (El. 433.8 m). Higher groundwater levels were observed during and upon completion of drilling.

In Area 2, the subsurface in the western part of the proposed widening area on the south side of Highway 17, consisted of up to 1.4 m thick sand with gravel, sandy gravel, and gravelly sand materials with topsoil. In the eastern part, the overburden above the bedrock consisted of a 3 m thick peat and a 2.3 m thick layers of silt and silty sand materials. The bedrock was encountered at a depth of 0.3 m to 1.4 m in the western portion of the proposed widening, but it was as deep as 5.3 m (El. 441.2 m) in the eastern part. Groundwater level measured in a well was at a depth of 0.9 m (El. 445.7 m). Higher groundwater levels were observed during and upon completion of drilling.

In Area 3, a peat layer was encountered in the western part of the proposed widening on the south side of Highway 17. The peat was 1.4 m to 3.0 m thick and was underlain by 0.7 m thick silty clay and

1.3 m thick sand. In the eastern part, a 1.5 m silty sand was encountered beneath the ground surface followed by gravelly sand materials to the depth of 2.5 m (El. 457.7 m). Bedrock was encountered at depth in the range of 2.3 m (El. 457.3 m) to 5 m (El. 455.2 m). The groundwater level, measured in a well installed in a borehole completed in the western part, was at a depth of 1.3 m (El. 458.8 m). Higher groundwater levels were observed during and upon completion of drilling.

In Area 4, the subsurface in the lake on the north side of Highway 17, consisted of a 4.1 m thick peat underlain by 0.8 m thick organic silt. Beneath the silt, sandy silt and gravelly sand deposits were encountered. The (fen) peat consists of some alluvial sands, and is characterized by a higher degree of humification. Normally, the strength and permeability of a peat decreases as humificatioin increases. Fen peats are often underlain by soft organic mud. In the eastern part, a 1.4 m thick peat was encountered followed by silty sand and sandy silt deposits. The depth of the bedrock in the middle portion of the lake was 6.7 m (El. 444.2 m) from the lake level or the water surface. Further east and west, rock outcrops were observed at the lake sides. The groundwater level measured in a well installed in a borehole completed in the eastern part, was at a depth of 0.8 m (El. 450.7 m). This is similar to the lake level (El. 450.8 m), recorded at the time of the field investigation.

In Area 7, hand augers indicated that the depth to the bedrock is in the range of 0.7 m (El. 419.9 m) to 1.9 m (El. 418.6 m). Groundwater was observed to be close to the ground surface.

The bedrock encountered in all areas was described to be greenstone. It was dark green to black in color, fine to medium grained, weakly to moderately foliated, and consisted of quartz and plagioclase crystals. The rock cores, in most cases, were slightly weathered, and moderately to highly fractured.

No artesian groundwater conditions or signs of confined and pressurized aquifer (phreatic surface) were encountered within the depth of investigation in all areas.

9. Design Analyses and Recommendations

9.1 Stability of Existing Slopes

Slope stability analyses were first performed on existing slopes. Since the performance of the existing road embankments has been relatively good, the stability analyses of existing slopes would allow comparing the factor of safety (FS) values with respect to the stabilities of the new embankments.

For all stability analyses, the Spencer method of slope stability analysis was used to estimate the factor of safety against a rotational shear failure. The Spencer method of analysis analyzes potential circular shear surfaces by separating the materials above the failure plane into multiple segments and then using force and moment equilibrium to balance the forces in each segment. All stability analyses were carried out with a computer program called Slope-W. The geometries of the existing embankment profiles in all areas were obtained from the base plans supplied by the Ministry.

In Area 1, embankment stability with respect to rotational shear failure extending through the existing roadway embankment and out into the proposed widening, was analyzed using the information from boreholes. The stability analysis was performed at a “typical section” near the middle of the existing embankment or at Sta. 10+490. An amorphous peat of about 5 m thick was encountered in completed boreholes. The strength parameters of this peat were estimated based on experience. Generally, amorphous peat has lower shear strength parameters than fibrous peat. The peat outside of the existing embankment was assumed to be in an undrained condition. The drained, or long-term, condition reflects no excess pore water pressure present within the peat, thus its' strength is normally represented by the angle of internal friction. However, the undisturbed peat strength was assumed to be controlled by cohesion or undrained shear strength. The strength parameters of the existing embankment fill are based on assumed values for granular soils. The assumed shear strength parameters for each of the layers are given in Table 9.1. A groundwater level corresponding to the ground surface was assumed as a "likely case" because of the presence of wet ground.

In Area 3, the stability analysis considered the section through Sta. 11+620. This section is located in the eastern portion where the thickness of the existing embankment is high. The embankment widening on the south side of the highway will involve the native silt sand and gravelly sand materials. The soil strength parameters assumed for these materials and the embankment fill are given in Table 9.1. A groundwater level of 1.3 m was measured in a well installed in a borehole completed in the western part of the area, and this was used as a "likely case" for the analysis.

Table 9.1 – Soil Strength Parameters

Location	Material	Soil Property		
		Bulk Unit Weight (kN/m^3)	Internal Friction Angle (^{o})	Undrained Shear Strength (Cu) kPa
Area 1	Fill – Sand with Gravel	20	30	-
	Rock Fill	19	40	-
	Peat	10	25	10
	Sand and Gravel	20	32	-
Area 3	Fill - Gravelly Sand	20	32	-
	Rock Fill	19	40	-
	Silty Sand with Gravel	19	25	-
	Gravelly Sand	20	32	-
Area 4	Fill - Gravelly Sand	20	32	-
	Rock Fill	19	40	-
	Sandy Gravel	20	35	-
	Peat	10	25	10
	Silt, Some Sand	19	20	-

In Area 4, a stability analysis was conducted for a section across the middle of the embankment (Sta. 12+405). A 4.1 m thick peat was encountered in a borehole drilled in the lake underlain by 0.8 m thick silt, sandy silt and gravelly sand deposits. The soil strength parameters assumed for these materials and the embankment fill are given in Table 9.1. An undrained shear strength was again assumed for the peat. The lake level was used as a "likely case" to model the groundwater level.

No boreholes were drilled through the embankment in Area 2 and no stability analysis was carried out. Similarly, no analysis was carried out in Area 7 because of the lack of borehole information.

The results of the stability analyses are given on Drawings 7, 8 and 9, in Appendix E. The FS value for the existing embankment in Area 1 was 1.7. Any slip plane in this area would be limited to a shallow depth because of the presence of bedrock beneath the embankment. The FS value for the embankment in Area 3 was 1.5 and this value also indicates that the consequence of failure is low. For Area 4, the FS value obtained for the existing embankment was 1.3, and this corresponds to a value that is often used as a threshold to design highway embankments with no or little risk of instability. From these results, it is possible to deduce that the rock fills used to construct the existing embankments have been well compacted and densified and the slopes are stable at the present conditions. In general, rock fills are characterized by high shear strength, particularly when they have been heavily compacted, and often have the ability to remain stable at steep slopes.

9.2 Design Recommendations

Table 9.2 provides embankment construction alternatives discussed in terms of advantages, disadvantages, relative costs and risks/consequences. The engineering evaluation of embankment design and construction considers the presence of soft soils and organic deposits, and their impact on slope instability, settlement, and bearing capacity. In addition to these, factors that can affect the choice of embankment construction are the availability of construction materials in the surroundings of the project, environmental issues, constructability, cost, and future maintenance.

The most common methods of embankment construction include excavation (displacement) and replacement, stage construction or preloading, the application of surcharge loads, physical and chemical stabilization, the use of lightweight fills, reinforcement and pile foundation. The excavation (displacement) and replacement method involves the complete or partial removal of soft soils by excavation or displacement. Stage construction is used when complete removal of soft soils or organic deposits is not possible. In this method, construction of the embankment is conducted in stages to allow the strength of the soft soil to increase due to the weight of the new fill. The basis for the use of preloading is the placement of additional fill over the top of the new embankment. On the other hand, load reduction primarily refers to the use of light weight fills to reduce the weight of the new embankment and minimize permanent stress on the underlying soft soils.

Chemical stabilization refers to the use of materials such as lime and cement. In this method, the strength gain in the soft soil is linked with the hydration products formed by cementitious reactions. Physical stabilization involves the use of dynamic or vibro-compaction, stone columns, and vertical, wick or sand drains. Dynamic compaction is used to achieve deep ground densification and vertical drains help to shorten the drainage paths thereby to accelerate the rate of pore water dissipation.

In embankment construction, reinforcement typical implies the use of geotextile or geogrid placed between the soft soil and the new fill. It is normally used to reduce differential settlement and increase the stability of the embankment. Further, the use of pile foundation refers to the installation of concrete, timber or steel piles to transfer the embankment load to a material of higher geotechnical resistance or the bedrock. Pile foundation needs high mobilization, setting up and installation costs.

Table 9.2 – Ground Improvement and Embankment Construction Alternatives

Construction Method	Methodology	Advantage	Disadvantage	Risk / Consequence	Relative Cost	Project Feasibility
Excavation / Replacement	Involves the full or partial excavation of soft materials and organic deposits.	• Minimize or eliminate the risk of post construction settlement • Ease of handling and construction • Low risk option	• May require significant amount of fill within acceptable haul distance • Potential for disturbance of existing embankment • May require extensive dewatering efforts	• Damage to the pavement structure and slope because of disturbance to the embankment; low risk.	Low to Medium	Feasible and recommended
Displacement	Involves the use of surcharge to displace soft deposits, peat and swamp	• Ease of handling and construction • Low risk option	• May result in the intrusion of fill into the area outside the limits	• Pockets of soils may remain resulting in differential settlements, low risk • May affect the surrounding areas, low risk	Low to Medium	Feasible and recommended
Stage Construction	Involves planned waiting periods between stages.	• The foundation soils can dissipate excess pore pressure due to the first fill, and settle without an additional surcharge. • Minimize the risk of post construction settlement	• The strength gain of the soft soil or peat will be required to ensure the stability of the next stage and regulate waiting periods • Extended construction schedule	• The remaining soft soil or organic deposit may become “trapped” beneath the embankment and cause long term settlement, low risk.	Low to Medium	Feasible and recommended in areas where there are ROW conflicts
Surcharge Load / Preloading	Refers to the placement of additional fill above the required grade.	• Accelerate or Force possible detrimental settlements that would otherwise occur after construction to occur during construction	• Determining the waiting period for the application of the surcharge load might need field instrumentation and monitoring	• A surcharge may create potential stability problems, low risk.	Low to Medium	Feasible and recommended in areas where there are ROW conflicts

Table 9.2 – Ground Improvement and Embankment Construction Alternatives

Construction Method	Methodology	Advantage	Disadvantage	Risk / Consequence	Relative Cost	Project Feasibility
Load Reduction	Involves the use of lightweight fills such as cellular concrete, slag, ash, expanded shale or polystyrene, and shredded waste tires or tyre chips.	• Reduces the load applied on the foundation soils and the resulting settlement.	• Special skills and techniques may be required for construction.	• Environmental concerns may exist with some of these products, low risk. • Effect on future maintenance of the roadway; low risk.	Medium to High	Not feasible
Chemical Stabilization	Involves the use of lime and cement, jet and injection grouting, electro-osmosis, deep soil mixing, and thermal techniques.	• Long term increase in strength, as the reaction between the additives and the soil will continue for months after construction.	• The use of chemical stabilization is still not common for embankment construction.	• The application of some of these methods may affect the environment; medium risk.	High	Not feasible
Physical Stabilization	Involves the use of dynamic and vibro- compaction, stone columns, vertical, wick or sand drains in conjunction with preloading.	• Are effective in accelerating strength gain in soft soils and organic deposits.	Proper monitoring may be required to check the state of stress in the soil during the loading period or	• Associated with potential construction difficulties and delays, low to medium risk.	Medium to High	Feasible but not recommended
Reinforcement	Involves the use of geotextiles, geogrids, or the use of reinforced soil slope (RSS) walls.	• Prevent soil migration, provide a stable base or reduce potential slope failure.	• May not be durable for the case of rock fill embankments.	• May be damaged during the placement of the rock fill, medium to high risk.	Medium to High	Not feasible
Pile Foundation	Involves the placement of embankments on piles or deep foundation.	• The embankment footing will be placed beyond the limits of soft soils, peat or organic deposits.	• Construction may need an extended period of time. • The use of piles may be expensive.	• May lead to differential settlement when used for embankment widening, low to medium risk	High	Not feasible

Based on the results of the slope stability analyses of existing slopes and due to the absence of signs of settlement on any of the existing embankments, the recommended option for embankment widening in all areas, is the removal of any peat, soft and loose materials and the replacement of these deposits with rock fill and/or granular materials to maintain the homogeneity and integrity of the entire road embankment. The availability of outcrops that can be used as source of rock fill, and the presence of shallow bedrock in all areas, make “excavation and replacement” as the most feasible option to construct the embankments. In areas where there is a ROW conflict and the complete removal of peat and soft soils is difficult, the excavation and replacement procedure can be combined with “stage construction” or “preloading”. One risk associated with this procedure is the possibility that a thin layer of peat or soft material become “trapped” beneath the fill and cause long term settlements.

Generally, in all areas, it is recommended to use “excavation and replacement” for embankment construction. This option bears a higher degree of reliability and a lower amount of risk with respect to long-term performance of the embankments than other methods presented in Table 9.2. The use of “excavation and replacement” require proper level of visual inspection at all times. If failure or settlement is expected based on visual observation during construction, then instrument monitoring using inclinometers, piezometers and settlement gauges should be implemented.

Given the condition of existing embankments and the subsurface conditions, soil improvement, lightweight fills, the use of geotextile, and pile foundation are not feasible. The use of physical stabilization (wick or vertical) drains may be feasible but not recommended for this project.

9.2.1 Area 1

In Area 1, the maximum thickness of the peat layer along the alignment of the proposed widening is about 5 m encountered at around Sta. 10+500. The thickness of the peat decreases to approximately 1 m in the western portion and 3 m in the east end of the embankment. The geotechnical investigation revealed the presence of a bedrock at a depth of 3.7 m to 5 m. Hence, the complete removal of the peat and any other soft material, and the replacement of these materials with rock and granular fill is a feasible option. Bedrock was also encountered beneath the existing road embankment and this would help maintaining the stability of the existing embankment slope during excavation.

For fill placement, the removal of the peat beyond the footprints of the new embankment is recommended where the ROW permits. In locations where ROW conflicts occur, the extent of disturbance into the adjacent areas and wetlands could be significant as a result of peat removal. In these areas, it is advisable to limit excavation a minimum of 1 m from the ROW.

As an alternative to excavation, the removal of the peat can also be carried out with the use of a “displacement method”. In this method, attempts are done to displace the peat assisted by surcharge, with or without the use of partial excavation to start the displacement process. Full displacement method may be employed with the surcharge load placed by end dumping. The effect of the surcharge load is to cause shear failure within the peat, and to aid the displacement of the peat to the side of the embankment. Once the displacement is completed, the surcharge load is normally left in place for a sufficient period to further force the consolidation of any remaining peat. A disadvantage of the displacement method is that waves of displaced peat may be formed at the sides and these waves of peat may act passively to prevent the displacement continuing. Therefore, it is always necessary to remove the displaced peat off the alignment of the embankment widening.

A slope stability analysis was conducted using the same section for which a stability analysis of the existing embankment was carried out. For the analysis, 1.25H:1V slope gradient was assumed. Based on the draft cross sections, the proposed widening was considered to be 3 m. In addition, blasted rock fill was assumed for the construction of the new embankment. This blasted rock fill will be overlain by a granular fill as a roadbed material. Generally, the slope stability analyses considered the placement of about 8 m rock fill and 1 m of granular fill with no peat underneath the embankment. The strength parameters for layers of the existing embankment were taken from Table 9.1. For the new rock fill and granular fill, a friction angle of 40º and 30º was assumed, respectively.

The result of the slope stability analysis is presented on Drawing 10 in Appendix E. The FS value for a rotational slip plane that passes through the new fill was found to be 1.5. This is higher than the 1.3 required for highway embankments. Embankment instability typically occurs either during embankment construction or shortly after, but can also occur over the long-term. Embankment instability occurs in the form of either a planar slide extending out into the adjacent area, or a deep-seated, rotational failure plane extending through the embankment and through the underlying soft soils. Deep seated failure plane development can occur either quickly with the formation of a large head scarp and heaving of the adjacent ground near the toe of the slope, or very slowly in a creeping type failure, where the failure plane may move only a small amount over a long period of time.

The result of the stability analysis in Area 1 indicates that deep-seated failures are unlikely if the remaining peat and soft materials are removed, and the embankments are constructed of blasted rock fill. In addition, because of the presence of a shallow bedrock beneath the existing embankment, any deep-seated rotational shear failure, if occurs, will be limited to the new embankment.

A settlement analysis was also considered to evaluate settlement on the new embankment during and after construction. Settlement in rock fill embankments is associated with the crushing of points of contact and rearrangement of rock fragments under the load or a change in moisture content. If a groundwater rises, poorly compacted, unsaturated rock fill is likely to be susceptible to settlement. The magnitude of settlement manifested in this form depends on the type and nature of the rock fill, the thickness of the rock fill, gradation, the method of construction, and subgrade conditions.

Generally, for rock fill embankments constructed on non-compressible soils, the MTO guideline for rock fill settlement and rock fill quantity estimates (MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates, September 14, 2010) should be followed. Table 9.2.1 presents the short and long term settlements provided in this guideline in terms of the embankment height for compacted and dumped rock fills. In Area 1, the rock fill is assumed to be placed on a very dense sand and gravel layer after the peat is removed. The sand and gravel layer together with the underlying bedrock are considered as non-compressible materials, and are capable of resisting the surcharge load from the rock fill. Hence, the occurrence of settlement as a result of compression of rock particles in the short or long term will be negligible, or would not exceed the amounts provided in Table 9.2.1.

Further in Area 1, the surcharge load from the weight of the new fill, is about 200 kN/m^2. The ultimate bearing capacity of the sand and gravel layer and the bedrock is higher than the estimated surcharge load, and no failure against baring capacity is anticipated because of embankment widening.

Table 9.2.1 – Short and Long Term Rock Fill Settlement

Height of Rock Fill, H (m)	Short-Term Settlement (m)		Long-Term Settlement (m)	
	Compacted Rock Fill	Dumped Rock Fill	Compacted Rock Fill	Dumped Rock Fill
Up to 5	0.5%H	1%H	-	-
>5 - 10	0.75%H	1.5%H	-	-
>10 – 15	1%H	2%H	-	-
Up to 15	-	-	0.1%H	0.2%H

In locations where the draft cross sections provided by the Ministry show ROW conflicts (from Sta. 10+500 to the end of the embankment on the east side, Sta. 10+600), the use of "excavation and replacement" method may not be feasible to remove the entire peat. In this case, the use of preloading may be considered. Generally, peat is well suitable for preloading as it has a high permeability and compress in a relatively short time when loaded. The amorphous peat typical has very low shear stiffness and tend to deform initially in an undrained manner when subjected to preloading. Preloading requires an excess load, usually about 25% higher than the weight of the new embankment load, placed evenly on the peat. The time required for preloading varies according to the extent of the soft soil and the nature of the underlying material. In Area 1, the maximum thickness of the peat at the location of ROW conflicts is 3 m, and the peat is underlain by non-compressible materials. Hence, a period of 4 to 6 months should be enough to bring about a sufficient degree of consolidation and limit the post construction settlement to 50 mm. Differential settlement would also be minimal.

In addition to preloading, stage construction may also be considered to allow the strength of the peat to increase under the weight of an applied fill. In this method, the first lift is laid thick enough to withstand construction traffic as well as prevent local shear failure of the peat. Construction of subsequent stages commences when the strength of the underlying peat is sufficient to maintain stability. Fibrous peat is suitable for stage construction, but the method can also be used in amorphous peat with longer timescales for stages. Generally, in Area 1 where the peat is less than 3 m, a two or more stage fill placement may be considered, with a first lift of about 1.5 m placed as a working platform. Subsequent layers will be placed about 30 days later, when 50% - 70% of the primary consolidation is reached. This would reduce or eliminate post construction settlements. During the placement of fill at each stage, care should be exercised to satisfy stability requirements.

A slope stability analysis was conducted for the new embankment with a 3 m thick preloaded peat underneath the rock fill. A friction angle of 25º was assumed for the peat underneath the rock fill to reflect the drained or long-term condition. Based on the draft cross sections, a 3 m embankment widening, a 1.25H:1V slope, and the placement of 5 m rock fill was considered. The strength parameters for layers of the existing embankment were taken from Table 9.1.

The result of the slope stability analysis is presented on Drawing 11 in Appendix E. The FS value for a rotational slip plane that passes through the new fill and the underlying peat was found to be 1.3. The result of the stability analysis indicates that deep-seated failures are unlikely if the remaining peat and soft materials are preloaded for a significant period of time or a stage construction that allows the strength of the peat to increase under the weight of an applied fill is implemented.

In Area 1, the draft cross sections provided by the Ministry show Bell utility conflicts at three locations (Sta. 10+400, Sta. 10+480, and Sta. 10+540). GHD was not provided with the installation drawings of these poles. Generally, it is recommended that these utility poles be relocated outside the influence of the new embankment prior to construction, as the increased vertical and lateral stresses may cause distress to the poles. Foundations for wood pole structures either use direct embedment with native or granular backfill or are designed as special foundations for the specific site. Direct embedment is simpler to relocate. If relocation is not possible, the foundation of these poles should be strengthened with guy wires and ground anchors before construction commences. The guy wires and anchors should be placed outside the zone of construction and should not affect the construction activity. They should also stay in place well beyond the embankment was completed if the rock fill was placed on a peat, to minimize the effect of any post construction settlement on pole foundations.

9.2.2 Area 2

In Area 2, a peat and a "very soft" silt was encountered in the eastern portion of the proposed embankment widening area. The thickness of these peat and silt layers was 3.8 m The removal and replacement will, therefore, be limited to the depth of about 4 m. Underneath the soft layers, the borehole logs indicated the presence of a 1.5 m thick "compact" to "very dense" sand with silt. The sand is considered to be non-compressible, and is underlain by a bedrock. In the western portion of the proposed widening, rock outcrops were observed on both sides of the existing culvert.

No stability analysis was conducted for Area 2, because of the absence of a borehole completed at the existing embankment. However, the existing slope didn't show any signs of distress or failure apart from minor rill erosions, and the new embankment can be constructed using the a slope gradient of 1.25H:1V. Based on the good performance of the existing slope, no slope instability problems are anticipated for a rock fill embankment constructed with 1.25H:1V side slopes.

In Area 2, the draft cross sections provided by the Ministry showed no ROW conflict. Hence, the new embankment can be constructed with "excavation and replacement". The thickness of the rock fill required in the eastern portion will be about 4 m and the degree of compressibility of this rock fill should be completed during construction. Hence, long-term settlement beyond the amount suggested in Table 9.2.1, is not anticipated in this area. However, most complex differential settlements occur when embankments contain several different types of fill materials in various zones or when the embankment is not placed on a leveled and similar subgrade. In this area, any possible variation in fill compression because of the difference in the depth of the bedrock from east to west should be inspected and appropriately addressed during construction. Any differential settlement or creep at the transition zone can be greatly reduced if heavy compaction of the rock fill is implemented.

Due to the presence of shallow bedrock, failure against bearing capacity is not expected in Area 2.

In Area 2, the draft cross sections show a ROW encroachment between Sta. 11+150 and Sta. 11+200, and Bell utility conflict at Sta. 11+160. As in the case of Area 1, preloading or stage construction is recommended in this locations. Since the maximum thickness of the peat is approximately 4 m, a period of 6 months to 8 months would be enough to achieve the required result as the underlying layers are non-compressible. During this time, much of the consolidation within the peat will be completed. As a result, no post construction settlement that would exceed the amount provided in the MTO guidelines for tolerable post paving movements (50 mm) is anticipated. If a stage construction is considered, the recommendations provided for Area 1 should be followed.

To avoid utility conflicts, the Bell wood pole at Sta. 11+160 should be relocated. Alternatively, guy wires and ground anchors can be used to strengthen the foundation of this pole.

9.2.3 Area 3

In Area 3, the draft cross-sections provided by the Ministry show no ROW conflicts, and excavation and replacement should be feasible to remove the peat and any soft material throughout the alignment of embankment widening. The thickness of the peat and silty clay encountered in the western portion is about 3.7 m. Removal and replacement will, therefore, be required for a depth of about 4 m. In the east, the subsurface consisted of about 1.5 m thick "very loose" to "compact" silty sand with gravel, and it is recommended to remove this material. Bedrock was encountered beneath the existing embankment at a depth similar to the ground level of the proposed widening, and this would help maintaining the stability of the existing embankment during excavation.

A slope stability analysis was carried out using the same section where a stability analysis of the existing embankment was conducted. For this analysis, a slope of 1.25:1V was adopted. As per the draft cross sections, a 2.5 m embankment widening was considered. Further, the slope stability analysis assumed the placement of about 5 m thick rock fill placed on gravelly sand. The amount of rock fill required decreases further east as the depth of excavation decreases. The soil strength parameters of the existing embankment were taken from the previous analysis. For the new rock fill, a friction angle of 40^{o} was assumed. Similarly, the friction angle for the granular fill was 30^{o}.

The result of the slope stability analysis is presented on Drawing 12 in Appendix E. The FS value for a rotational slip plane that passes through the new fill was found to be 1.4. This is higher than the 1.3 required for highway embankments, and the result indicates that deep-seated failures are unlikely if the "very loose" silty sand encountered in the east part of the embankment is removed. It is assumed that similar results would be obtained if analysis would be done for a section in the west where a recommendation is given to remove the peat and silty clay completely.

In addition to stability analyses, the potential for the occurrence of settlement of the new embankment during and after construction was also assessed. Generally, since the peat and very loose materials will be removed, and the gravelly sand layer and the underlying bedrock are considered as non-compressible materials, the occurrence of settlement in the area is anticipated to be negligible, or would not exceed the amounts provided in MTO guidelines given in Table 9.2.1.

Further, the surcharge load from the weight of the new fill is computed to be about 140 kN/m^2. The ultimate bearing capacity of the sand and gravel layer and the bedrock is higher than the estimated surcharge load, and no failure against baring capacity is anticipated in Area 3.

The draft cross sections indicate that a Bell pole installed at around at Sta. 11+620 may create a utility conflict during construction. Since the complete removal of the "very loose" silty sand materials is recommended in this area, the pole needs to be relocated before construction commences. If evidences exist that the pole was founded on bedrock, then strengthen the pole foundation using anchors may be considered instead of relocation. In this case, the contractor's procedure to excavate around the pole foundation should be reviewed and approved by a geotechnical engineer.

9.2.4 Area 4

In Area 4, a peat and "very soft" silt was encountered in a borehole drilled in the lake. The total thickness of these soft materials was about 5 m. A 1.4 m thick peat was also encountered east of the lake. This peat and any soft soil should be excavated and replaced with a rock fill.

A slope stability analysis was carried out using the same section for which a stability analysis of the existing embankment was conducted. For the stability analysis, a gradient of 1.25H:1V was utilized. Using the current plans, a 7 m embankment widening was considered. In addition, the slope stability analysis assumed complete removal of the peat and underlying soft deposits and the placement of about 5 m of rock fill. The amount of rock fill required decreases further east and west. The strength parameters for soil layers of the existing embankment were taken from the previous analysis. For the new rock fill and granular fills, friction angles of 40^{o} and 30^{o} was assumed, respectively.

The result of the slope stability analysis is presented on Drawing 13 in Appendix E. The FS value for a rotational slip plane that passes through the new fills, the existing sandy gravel material beneath the embankment, and the peat in the lake was found to be 1.6. This is higher than the 1.3 obtained for the existing slope. The result of the stability analysis indicates that deep-seated failures are unlikely if the peat and the soft silt is removed, and replaced by rock fill for embankment construction.

In addition to stability analyses, the potential for the occurrence of settlement of the new embankment was also assessed. Generally, since the peat and soft materials are removed, and the underlying bedrock on which the fill will be placed is non-compressible, the occurrence of post construction settlement will be negligible. However, unsaturated rock fill dumped in the lake is likely to be susceptible to compression when it is initially inundated with water, as an increase in moisture content may wash out the fine components. To mitigate this, it is recommended that the rock used in Area 4 should be durable, and should have coarse particles with very little fine materials.

Further, the surcharge load from the weight of the new fill is computed to be about 220 kN/m^2. The ultimate bearing capacity of the sand and gravel layer and the bedrock is higher than the estimated surcharge load, and no failure against baring capacity is anticipated in Area 4.

9.2.5 Area 7

In Area 7, the hand auger holes indicate the presence of a probable bedrock at depth in the range of 0.7 m to 1.9 m. The depth of excavation will, therefore, be shallow. However, the information from auger holes is very limited in essence and should be supplemented by a borehole investigation.

No stability analysis was conducted for Area 7, because of the absence of boreholes completed in the area. However, the existing slope didn't show any signs of distress or failure apart from minor rill erosions, and any embankment widening can be constructed using a rock and granular fill. Based on the good performance of the existing slope and field inspections, no slope instability problems are anticipated for a rock fill embankment widening constructed with 1.25H:1V.

Rock fill embankments with a thickness of greater than 10 m require a minimum of 2 m wide berms, constructed in accordance with Ontario Provincial Standard Drawing (OPSD) 202.010 (Slope Flattening Using Surplus Excavated Material on Earth and Rock Embankment).

9.3 Culvert Replacements

The work order received from the Ministry indicated that existing culverts in Area 1 and Area 4 might be replaced. However, no boreholes were drilled at culvert locations and details of culvert replacements were not known. Recent discussions with the Ministry's Project Manager indicated that the Ministry intends to carry out culvert replacement in Area 4 without widening. Generally, the subsurface materials encountered in Area 1 and Area 4 beneath the peat and soft soils should be able to provide the geotechnical resistance required for structural culverts. GHD can be contacted to provide recommendations specific to the areas, when details of the preferred culverts are known.

It is assumed that the replacement of culverts will require staged construction that will allow traffic to use one side of the highway with the aid of a temporary traffic signal. If a staged construction is required at any of the areas, a properly designed roadway protection system should be provided. The design and installation a roadway protection system is the responsibility of the contractor.

In Area 1 and Area 4, the presence of shallow bedrock and/or very dense materials may not create favorable conditions for driving sheet piles to construct shoring systems. Hence, the use of soldier piles and timber laggings supported by anchors may have to be considered. In both areas, a stable slope could be maintained for excavation through the rock fills, if a staging scheme is considered. Based on the field observation and performance of the existing embankments, a slope of 1.25H:1V is considered feasible for construct of temporary slopes in the existing embankment fills.

Temporary roadway protection system should be designed and installed in accordance with Ontario Provincial Standard Specification (OPSS) 539, Level 2 performance. The soil parameters given in Table 9.3 are recommended for the design of the roadway protection systems in all areas.

Table 9.3 – Recommended Soil Parameters for Shoring

Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (^{o})	Undrained Shear Strength Cu (kPa)
Peat	10	25	10
Fill – Gravelly Sand	20	32	-
Rock Fill	19	40	-
Sandy Gravel	20	35	-
Silt, Some Sand	19	20	-

9.4 Frost Depth

According to OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario), the frost penetration depth in the area is 2.2 m.

10. Construction Considerations

10.1 Excavation

Excavation for construction of the new embankments is expected to extend to a depth of approximately 5 m in Area 1 and Area 4, and 4 m in Area 2 and Area 3 below existing grade or the bottom of the lake. It is the contractor's responsibility to ensure the existing embankments and roadways in all areas are adequately protected from destabilization during excavation.

Generally, all excavations should be carried out to conform to the manner specified in Ontario Regulation 213/91 and the Occupational Health and Safety Act and Regulations for Construction Projects (OHSA). The excavations above the water table not exceeding 1.2 m in depth may be constructed with unsupported slopes. The peat encountered in Area 1, Area 2, Area 3, and Area 4 can be classified as Type 4 soil using OHSA classification, as peat is usually deposited below the groundwater table. Type 4 soils must be sloped at 3H:1V or flatter. Hence, any excavation of peat in Area 1, Area 2 and Area 3 should be conducted at a slope of 3H:1V. Alternatively, if the peat and other soft soils are removed, the remaining granular soils can be classified as Type 3 (or in some cases Type 2) soils above groundwater table. As such, unsupported walls of excavations in these soils should be sloped from the bottom of excavation with a slope of 1H:1V or flatter. In Area 4, the peat excavation may require the construction of a temporary protection system to limit disturbance in the lake. Similarly, temporary protections can also be used in Area 1, Area 2 and Area 3 where the excavation of the peat and soft materials at 3H:1V would extend beyond the ROW limit.

No vertical un-braced excavations greater than 1.2 m in depth should be performed in the soil or rock fill where workers must enter the excavation. The protection system for excavations should follow OPSS 539 (Construction Specification for Temporary Protection Systems). In addition, the Non Standard Specific Provisions (NSSP 1 and NSSP 2) on Excavation and Slope Stability, and Temporary Protection Systems provided in Appendix F, should be followed.

Excavation of the soils at the project site may be carried out using conventional excavation equipment. During excavation, no excavated material should be piled close to the top of the excavation slope. All excavated surfaces should also be kept free from frost. In addition, runoff and stream flow shall be directed into roadside ditches and outlest, away from open excavations.

Chemical testing of any excess soils generated during construction should be performed prior to shipment off-site, to assess the available handling and disposal protocols.

10.2 Fill Placement

The embankment widening should be carried out in accordance with OPSD 203.030 (Embankments Over Swamp, Existing Slopes Maintained), and simultaneous backfilling should be performed in accordance with OPSS.Prov 209, Section 209.07.03 (Embankments over Swamps and Compressible Soils). Fill materials used to construct the new embankments should be similar to the rock fill used for the construction of the existing embankments. Generally, the use of blasted rock fill is most feasible in all areas. Material for rock fill shall be obtained from approved sources. The material shall be excavated, processed, and handled as necessary to conform to gradation requirements. Rock foundation surfaces shall be cleared of all loose material not conforming to the specifications for rock fill. When there is a need to place large quantities of rock fill, it is advisable to carry out excavation and placement trials prior to commencement of the main earthworks.

In Area 1, Area 2, and Area 3, the fill placement is such that the rock fill shall be dumped and spread into position in horizontal layers not to exceed 1.5 m in thickness. The rock fill should be spread so that the coarser material does not accumulate on the surface of the layer. Experiences indicated that this is best achieved by dumping the rock fill on top of and behind the edge of the advancing fill surface and using a heavy bulldozer to push the material beyond the edge. This causes the larger particles to roll to the bottom of the layer and fines to fill the voids between them, thus creating a dense layer the surface of which consists mainly of the finer material. The fines mixed with the coarser material of the rock fill must be non-cohesive so as to be capable of being vibrated or washed through the layer. The presence of cohesive fines makes the achievement of a satisfactory state of compaction extremely difficult. The spreading of a separate layer of fine material on the surface of the rock fill should be avoided as it could substantially reduce the effective depth of compaction.

The placement of rock fill should be in accordance with OPSS.Prov 206 (Construction Specification for Grading). Compaction should be based on OPSS 501 (Construction Specification for Compacting).

If large horizontal flows of water are observed during construction, a relatively uniformly graded rock fill, completely devoid of fines can be placed at the bottom of excavation to serve as drainage layer. It can be assumed that this rock fill is in a well compacted condition as placed, in that inter-granular contact will not create any further rearrangement of the particles and compression.

In Area 4, rock fill placed by end dumping in-the-wet should be densified by deep compaction technique such as dynamic compaction. This can be carried out as soon as a platform of about 1.5 m has been established above the lake level. Above this level, the rock fill can be placed and compacted in layers. Alternatively, if excavation is done in-the-dry, construction of the embankment to the full depth can be carried out in layers. The rock fill material in Area 4 should be durable and resistant to degradation by lake level fluctuation, and should have very little fine material.

Roadbed materials above the rock fill should be placed and compacted to meet the requirements of OPSS 1010 (Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material and OPSS 501. Organics should be replaced by Granular A material.

10.3 Dewatering

The measured and observed groundwater levels in all areas are near the ground surface and all excavations may have to be carried out under wet conditions. Should any zones of intensive water-bearing soils be intersected by earth cuts or excavation, the degree of groundwater infiltration should be assessed by GHD at the time of construction, and appropriate recommendations provided. Such conditions may require intensive forms of groundwater control, including (but not necessarily limited to) sheet piling or caisson walls, depending on the depth of excavations and groundwater levels.

In Area 4, the lake level is at El. 450.8 m and excavation to remove the peat may need the construction of a coffer dam consisting of sand bags and puddle clay on the upstream side and on the sides, to divert the flow of water on one side of the culvert and work in stages. Sheet piles can then be installed to proceed with excavation. If restrictions are imposed on placing puddle clay or dewatering system in the lake, construction in-the-wet can be done by excavating without dewatering, placing rock fill by end dumping, overbuilding above the lake level, and use dynamic compaction.

For any dewatering system required in any of the areas, the contractor should be responsible for selection, performance and detailed design of the dewatering system including the cofferdam. The dewatering system should be designed to conform to the requirement of OPSS 518 (Construction Specification for Control of Water from Dewatering Operations). In addition to these standard specifications, the inclusion of the Special Provision for Dewatering Structure Excavations provided in Appendix F into contract documents should be considered.

If short-term pumping of groundwater at volumes greater than 50,000 L/day and less than 400,000 L/day is required during the construction stage, the Environmental Activity Sector Registry (EASR) must be completed. The EASR streamlines the process, and water pumping may begin once the EASR registration is completed, the fee paid and supporting document prepared. If water taking in excess of 400,000 L/day is required, a Permit to Take Water (PTTW) must be obtained in advance. PTTW applications may take up to 90 working days for the Ontario Ministry of Environment, Conservation and Parks (MECP) to review and approve. The actual rate of groundwater taking performed during construction will be a function of the final design, time of year, and the contractor's schedule, equipment and techniques. It is recommended that the contractor retain qualified and specialist dewatering and groundwater control expertise during construction.

10.4 Drainage

Drainage structures should be maintained to keep creeks flow downstream. It is anticipated new roadside ditches will be used to provide drainage during construction. The construction of new embankments should ensure that roadbed materials daylight appropriately to the ditches.

10.5 Winter Construction

Fill should be placed and compacted when air temperatures are consistently above freezing. Such a construction procedure will require "summer" construction. However, rock fill embankments can also be constructed in winter as they contain little fine materials. If the construction is carried out in winter, then care must be exercised not to use open-graded rock materials. Thaw settlement has been documented in some countries where open-graded blasted rock was used to construct embankments and that the thawing could be from free flowing water at the subgrade level.

10.6 Wells / Piezometers

Wells were installed in all areas to measure static groundwater level. Any decommissioning of these wells must be performed by a licensed well contractor, in compliance with Ontario Regulation 903.

11. Closure

This report was prepared by Mr. Lul Yimam, PhD, P.Eng., Senior Geotechnical/Foundation Engineer and MTO Key Personnel. Mr. Nyle McIlveen. P.Eng. conducted an independent review of the report.

Sincerely,

GHD

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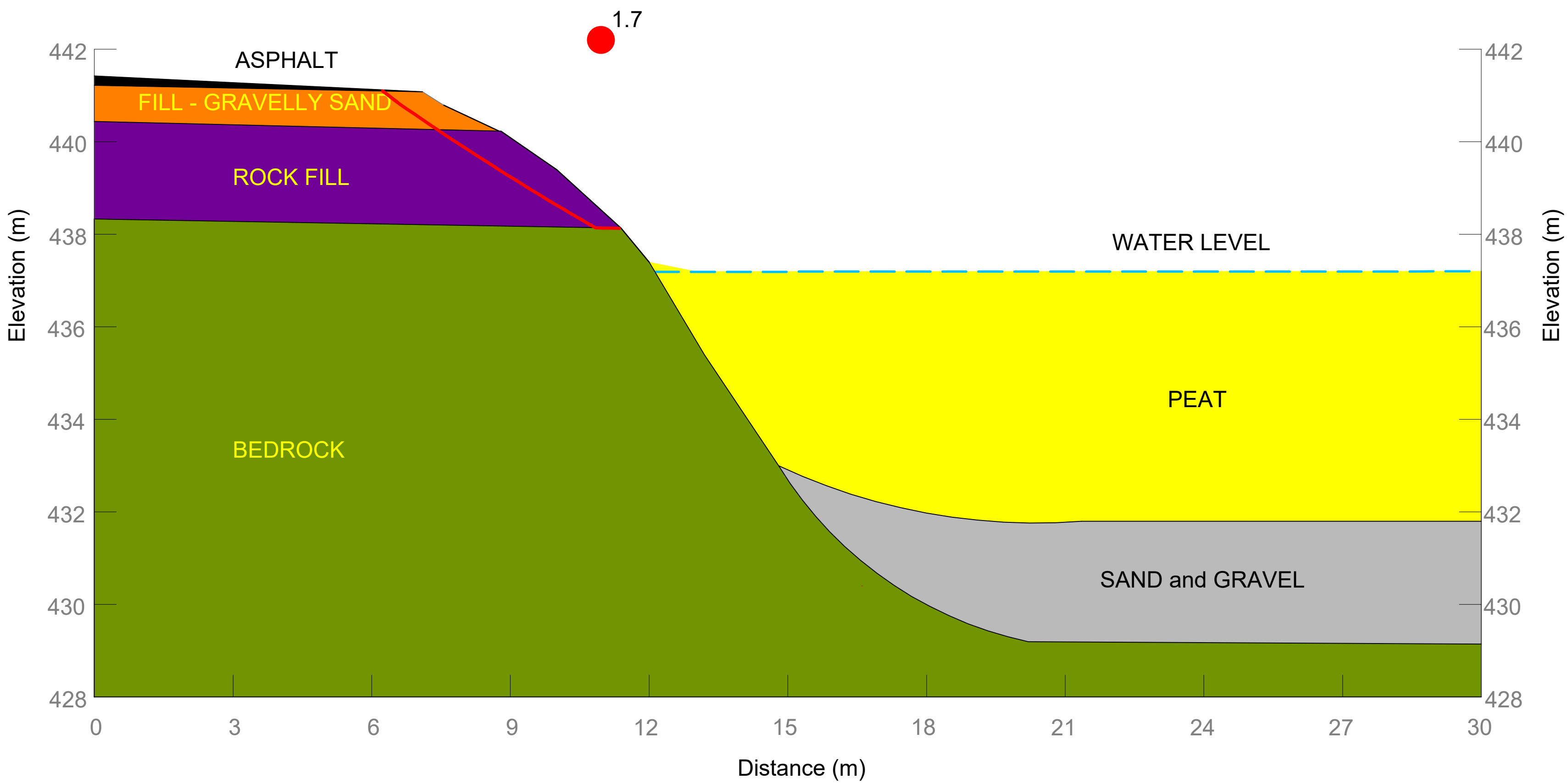
Principal

Appendix E
Results of Slope Stability Analyses

Soil Strength Parameter

Property \ Material	Asphalt	Fill - Gravelly Sand	Rock Fill	Peat	Sand and Gravel	Bedrock
Bulk Unit Weight (kN/m³)	-	21	21	10	21	-
Internal Friction Angle (°)	-	28	40	10	30	-
Undrained Shear Strength, (Cohesion) (kPa)	-	-	-	5	-	-



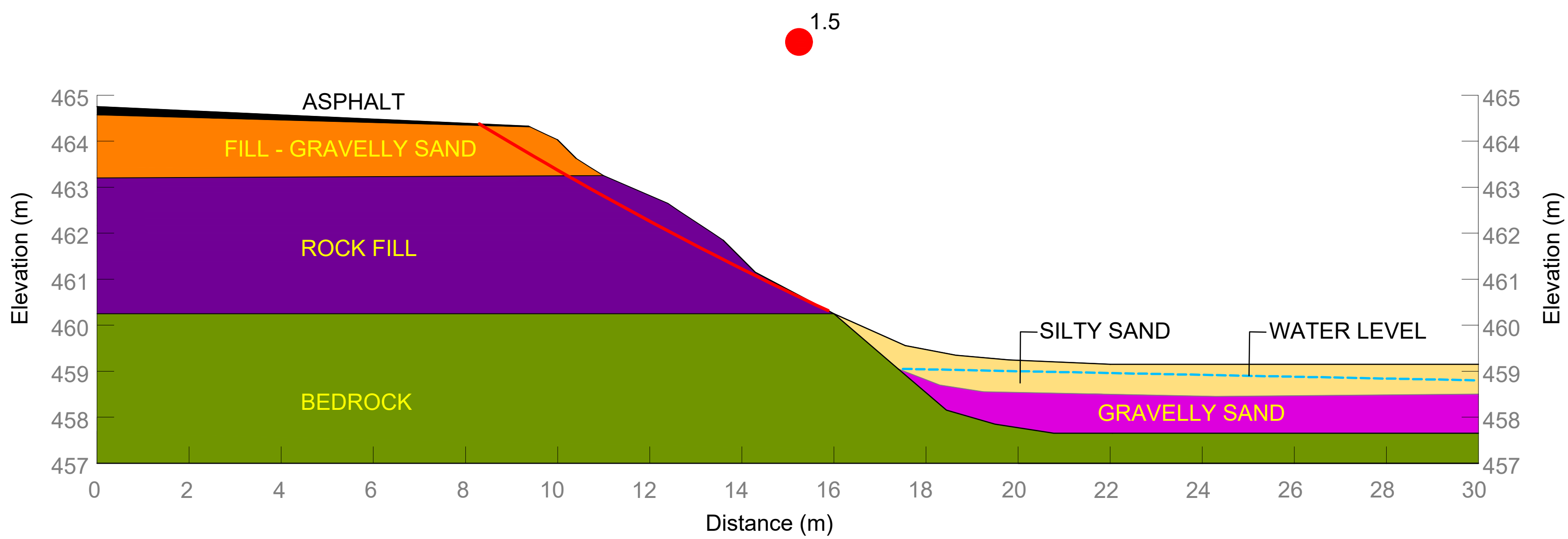
Existing Slope Typical - Slope Stability Analysis

Foundation Investigation and Design
Highway 17 - Widening, Between White River and Wawa
Area 1

Agreement: 5017-E-0043	Work Item: #5	Drawn:	AW	Project No.:	11185012 (500)
G.W.P: 5031-18-00	Hwy No.: 17	Checked:	LY	Revision:	
Township: Dambrossio		Reviewed:	LY	Date:	2020.01.17
District: Algoma		Approved:	NM	Drawing:	7

Soil Strength Parameter

Property \ Material	Asphalt	Fill - Gravelly Sand	Rock Fill	Silty Sand	Gravelly Sand	Bedrock
Bulk Unit Weight (kN/m^3)	-	21	21	19	21	-
Internal Friction Angle (°)	-	28	40	20	30	-
Undrained Shear Strength, (Cohesion) (kPa)	-	-	-	-	-	-



Existing Slope Typical - Slope Stability Analysis

Foundation Investigation and Design

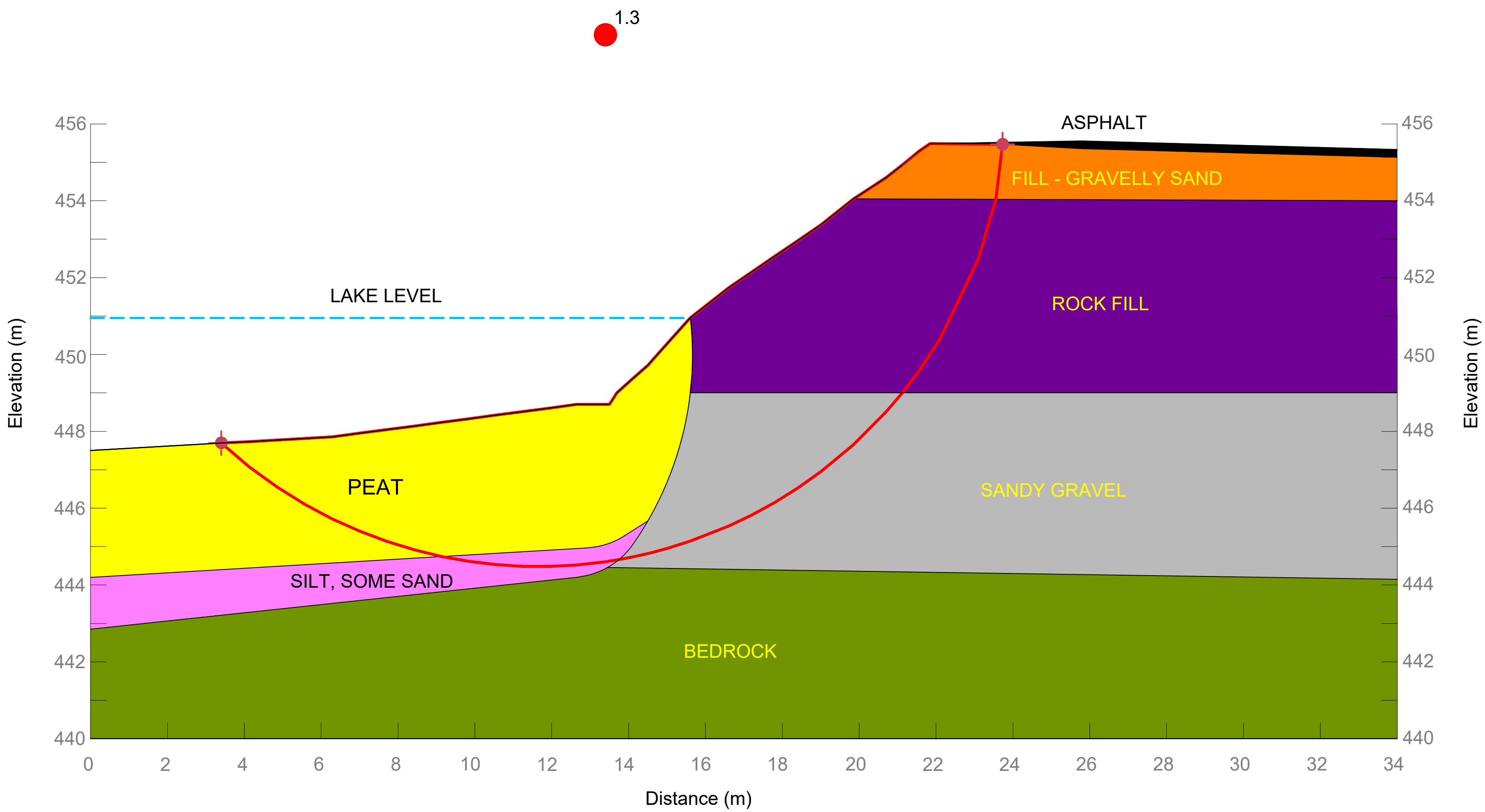
Highway 17 - Widening, Between White River and Wawa

Area 3

Agreement:	5017-E-0043	Work Item: #5	Drawn:	AW	Project No.:	11185012 (500)
G.W.P:	5031-18-00	Hwy No.: 17	Checked:	LY	Revision:	
Township:	Dambrossio		Reviewed:	LY	Date:	2020.01.17
District:	Algoma		Approved:	NM	Drawing:	8

Soil Strength Parameters

Property \ Material	Asphalt	Fill - Gravelly Sand	Rock Fill	Sandy Gravel	Peat	Silt, Some Sand	Bedrock
Bulk Unit Weight (kN/m³)	-	21	21	21	10	19	-
Internal Friction Angle (°)	-	28	40	30	10	15	-
Undrained Shear Strength, (Cohesion) (kPa)	-	-	-	-	5	-	-



GHD

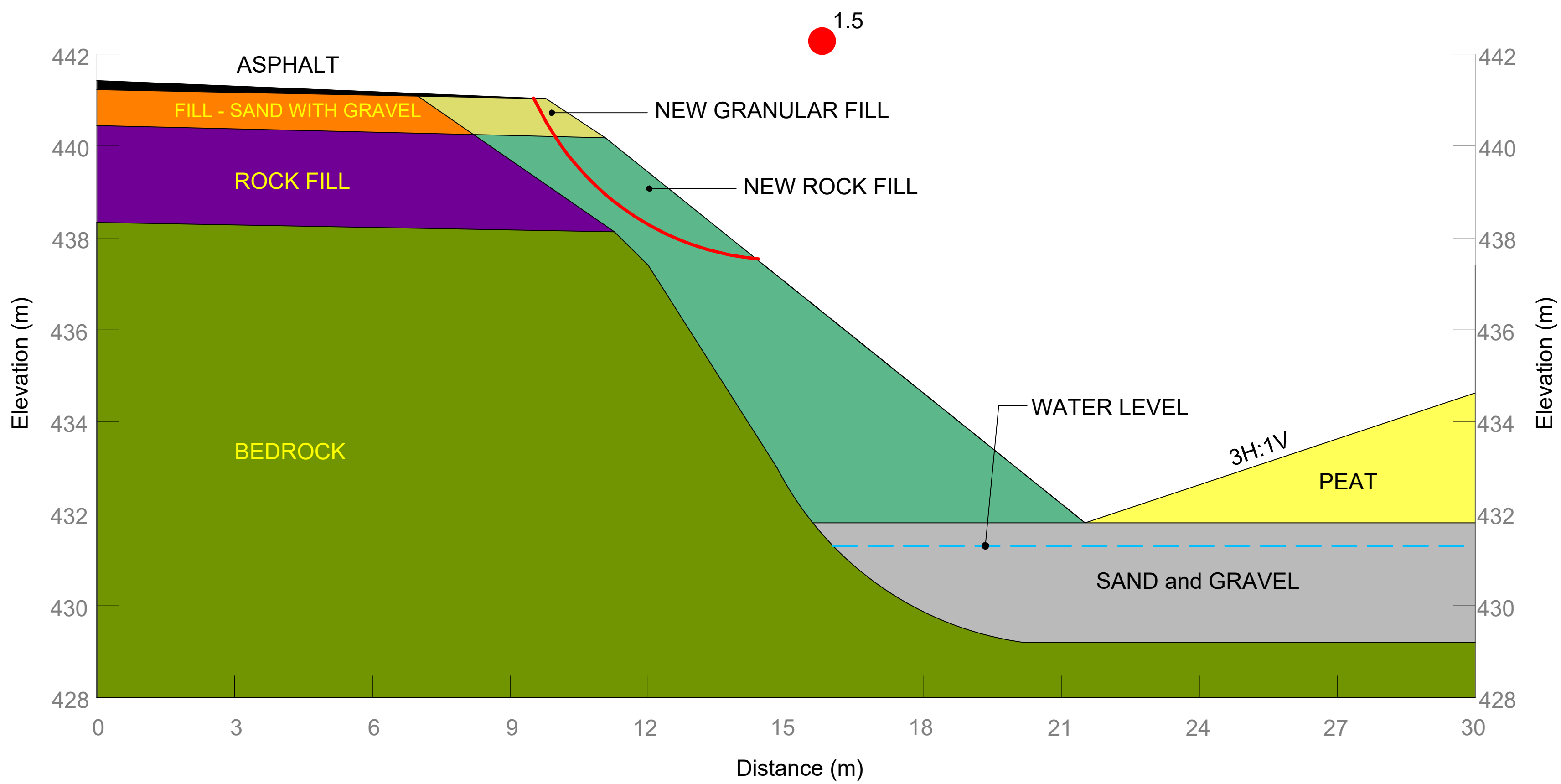
Existing Slope Typical - Slope Stability Analysis

Foundation Investigation and Design
Highway 17 - Widening, Between White River and Wawa
Area 4

Agreement:	5017-E-0043	Work Item: #5	Drawn: AW	Project No.:	11185012 (500)
G.W.P:	5031-18-00	Hwy No.: 17	Checked: LY	Revision:	
Township:	Dambrossio		Reviewed: LY	Date:	2020.01.17
District:	Algoma		Approved: NM	Drawing:	9

Soil Strength Parameters

Property \ Material	Asphalt	Fill - Sand With Gravel	Rock Fill	Peat	Sand and Gravel	Bedrock	New Granular Fill	New Rock Fill
Bulk Unit Weight (kN/m³)	-	20	19	10	20	-	20	19
Internal Friction Angle (°)	-	30	40	25	32	-	30	40
Undrained Shear Strength, (Cohesion) (kPa)	-	-	-	10	-	-	-	-



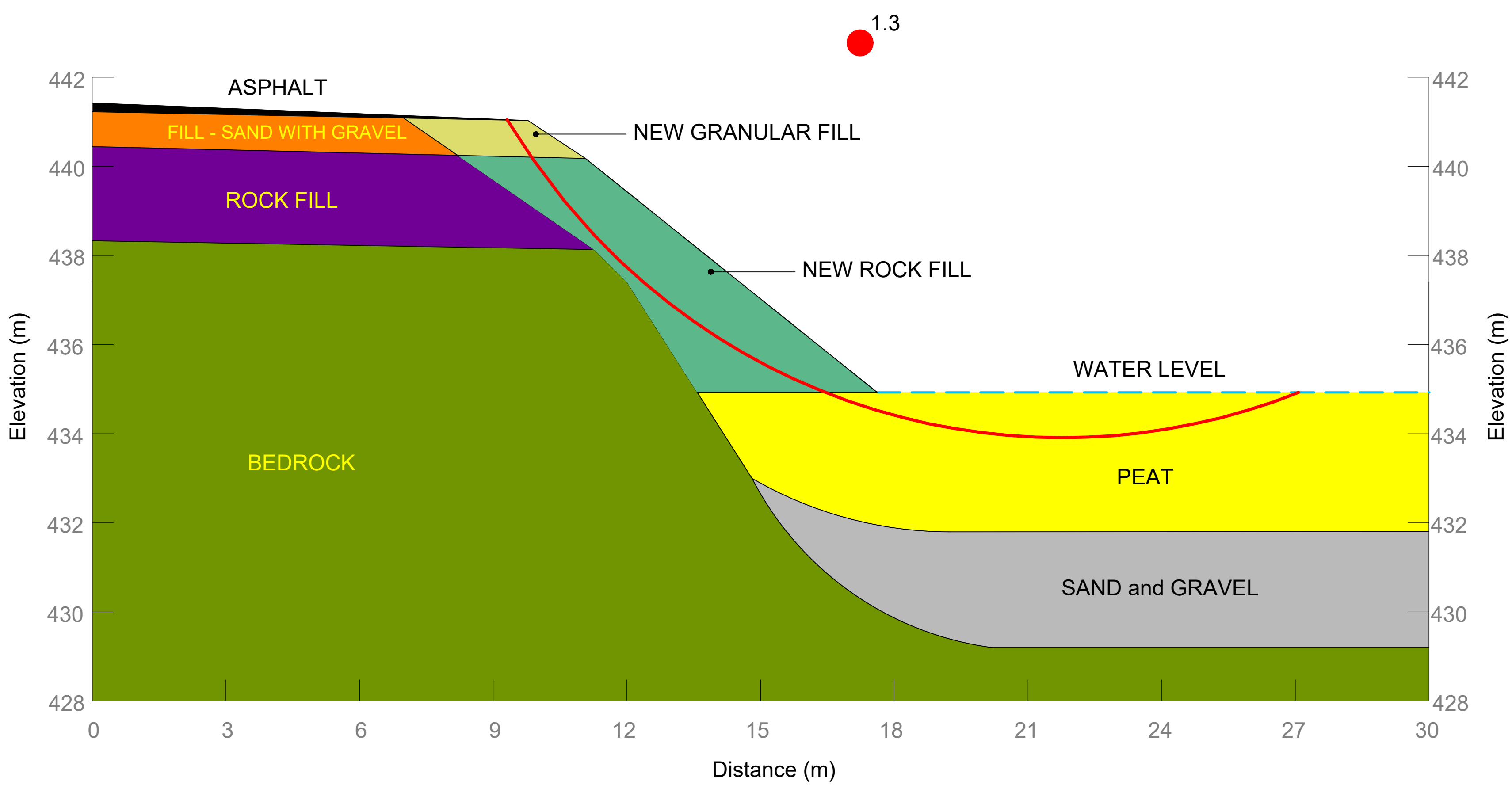
Proposed Slope (1.25H:1V) Slope Stability Analysis

Foundation Investigation and Design
Highway 17 - Widening, Between White River and Wawa
Area 1

Agreement:	5017-E-0043	Work Item: #5	Drawn:	AW	Project No.:	11185012 (500)
G.W.P:	5031-18-00	Hwy No.: 17	Checked:	LY	Revision:	
Township:	Dambrossio		Reviewed:	LY	Date:	2020.01.17
District:	Algoma		Approved:	NM	Drawing:	10

Soil Strength Parameters

Property \ Material	Asphalt	Fill - Sand With Gravel	Rock Fill	Peat	Sand and Gravel	Bedrock	New Granular Fill	New Rock Fill
Bulk Unit Weight (kN/m³)	-	20	19	10	20	-	20	19
Internal Friction Angle (°)	-	30	40	25	32	-	30	40
Undrained Shear Strength, (Cohesion) (kPa)	-	-	-	10	-	-	-	-



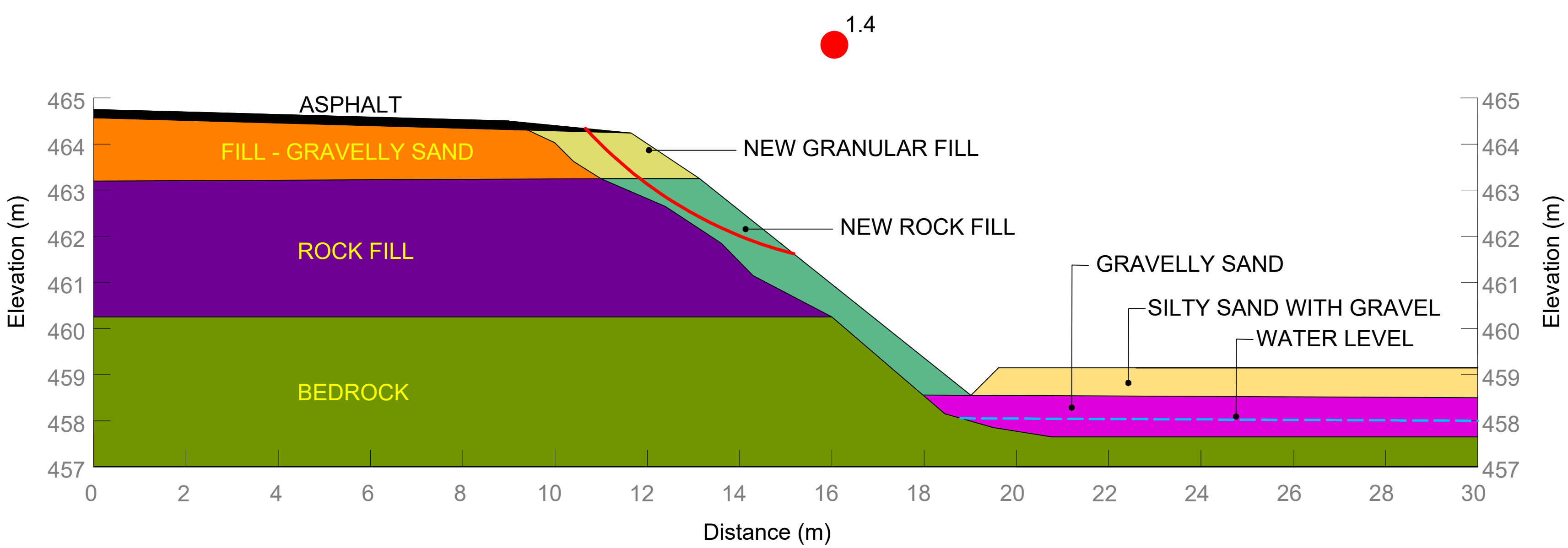
Proposed Slope (1.25H:1V) Slope Stability Analysis

Foundation Investigation and Design
Highway 17 - Widening, Between White River and Wawa
Area 1

Agreement:	5017-E-0043	Work Item: #5	Drawn:	AW	Project No.:	11185012 (500)
G.W.P:	5031-18-00	Hwy No.: 17	Checked:	LY	Revision:	
Township:	Dambrossio		Reviewed:	LY	Date:	2020.01.17
District:	Algoma		Approved:	NM	Drawing:	11

Soil Strength Parameters

Property \ Material	Asphalt	Fill - Gravelly Sand	Rock Fill	Silty Sand With Gravel	Gravelly Sand	Bedrock	New Granular Fill	New Rock Fill
Bulk Unit Weight (kN/m³)	-	20	19	19	20	-	20	19
Internal Friction Angle (°)	-	32	40	25	32	-	30	40
Undrained Shear Strength, (Cohesion) (kPa)	-	-	-	-	-	-	-	-



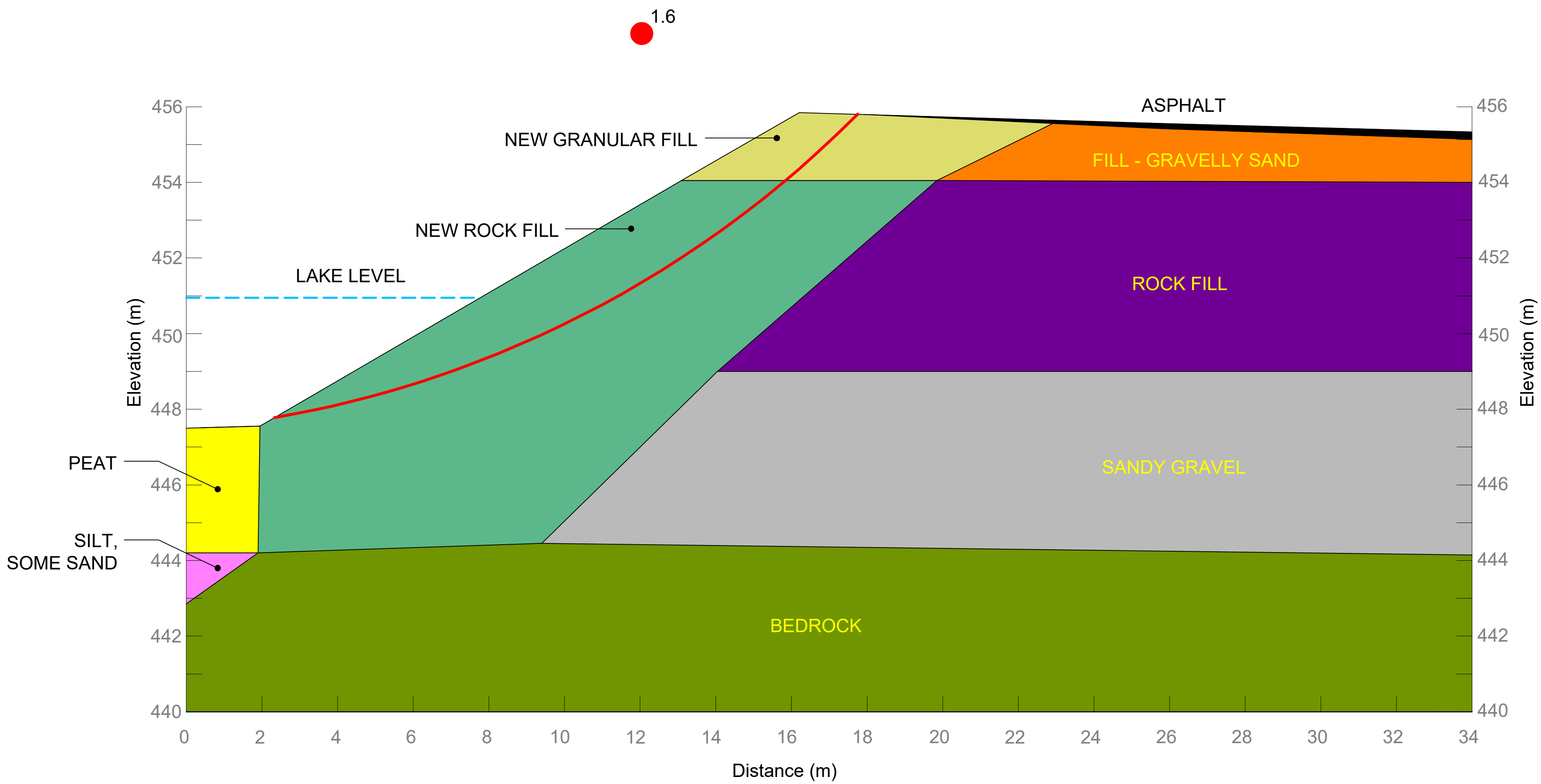
Proposed Slope (1.25H:1V) Slope Stability Analysis

Foundation Investigation and Design
Highway 17 - Widening, Between White River and Wawa
Area 3

Agreement:	5017-E-0043	Work Item: #5	Drawn:	AW	Project No.:	11185012 (500)
G.W.P:	5031-18-00	Hwy No.: 17	Checked:	LY	Revision:	
Township:	Dambrossio		Reviewed:	LY	Date:	2020.01.17
District:	Algoma		Approved:	NM	Drawing:	12

Soil Strength Parameters

Property \ Material	Asphalt	Fill - Gravelly Sand	Rock Fill	Sandy Gravel	Peat	Silt, Some Sand	Bedrock	New Granular Fill	New Rock Fill
Bulk Unit Weight (kN/m³)	-	20	19	20	10	19	-	20	19
Internal Friction Angle (°)	-	32	40	35	25	20	-	30	40
Undrained Shear Strength, (Cohesion) (kPa)	-	-	-	-	10	-	-	-	-



Agreement:	5017-E-0043	Work Item: #5	Drawn:	AW	Project No.:	11185012 (500)
G.W.P:	5031-18-00	Hwy No.: 17	Checked:	LY	Revision:	
Township:	Dambrossio		Reviewed:	LY	Date:	2020.01.17
District:	Algoma		Approved:	NM	Drawing:	13

Proposed Slope (1.25H:1V) Slope Stability Analysis

Foundation Investigation and Design
Highway 17 - Widening, Between White River and Wawa
Area 4

Appendix F
List of Ontario Provincial Standard Specifications (OPSS) and Drawings (OPSD), Non-Standard Special Provisions (NSSP), and Special Provision – Dewatering Structure Excavations

List of Ontario Provincial Standard Specifications (OPSS) and Drawings (OPSD) Referenced in the Report

Document	Title
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS.Prov 206	Construction Specification for Grading
OPSS 501	Construction Specification for Compacting
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSS 518	Construction Specification for Control of Water from Dewatering Operations
OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment
OPSD 3090.100	Foundation Frost Penetration Depth for Northern Ontario
OPSD 203.030	Embankments over Swamp – Existing Slopes Maintained
OPSS.Prov 209	Embankments over Swamps and Compressible Soils

NON-STANDARD SPECIAL PROVISIONS (NSSP)

NSSP 1 – Excavations and Slope Stability (Addition to OPSS.Prov 206 and OPSS 539)

The contractor is advised that the excavation of the peat and soft soils in all areas may require careful design of excavation and slope geometries and shoring schemes. The maximum limit of open excavation allowed adjacent to the travelled roadway in all areas will be determined by bedrock exposures and may require slope stability analyses. Staged excavation of limited extent shall be employed to maintain the stability of slopes and to protect the existing embankment during the excavation and backfilling operations for road widening. At the project site, peat excavation shall be carried out in sections no greater than 3 m wide along the embankment slope and no greater than 5 m long perpendicular to the embankment slope. Temporary protection systems may be required for excavation work within ROW limits. The contractor is responsible for the design of temporary shoring systems. The contractor is also advised to restrict the stockpiling of material and the placement of heavy equipment near slope crests to prevent slope instabilities.

NSSP 2 – Installation of Temporary Protection Systems (Addition to OPSS 539)

The contractor is advised that bedrock was encountered at shallow depth in all areas and cobbles and/or boulders may exist that can limit the depth of shoring elements. The contractor shall select and use the appropriate methods for shoring installation and excavations to account for such possible obstructions.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision

Amendment to OPSS 902, November 2010

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a [* Designer Fill-In, See Notes to Designer] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of [** Designer Fill-In, See Notes to Designer] metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website https://www.ontario.ca/environment-and-energy/map-well-records or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

The Quality Verification Engineer shall witness the following Interim Inspections of the work:

a) Dewatering of excavation for structure.

b) Completion of excavation for foundation.

c) Excavation for backfill and frost tapers.

d) Backfilling.

A copy of the written permission to proceed shall be submitted to the Contract Administrator prior to commencement of the successive operation.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

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 according to OPSS 805. Measures not specified in OPSS 805 shall be 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.
Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.
Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

<u>Designer Fill-Ins</u>

* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

** Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item **<u>only</u>** on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

about
GHD

GHD is one of the world's leading professional services companies operating in the global markets of water, energy and resources, environment, property and buildings, and transportation. We provide engineering, environmental, and construction services to private and public sector clients.

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