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A REPORT TO
TOWNSHIP OF MUSKOKA LAKES

A GEOTECHNICAL INVESTIGATION FOR BRIDGE REPLACEMENT

MILFORD BAY ROAD

TOWNSHIP OF MUSKOKA LAKES

REFERENCE NO. 1808-S134

OCTOBER 2018

DISTRIBUTION

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1.0 **INTRODUCTION**

In accordance with a Subconsultant Services Agreement issued by C.C. Tatham & Associates Ltd. dated August 17, 2018, a geotechnical investigation was carried out in the vicinity of the existing bridge crossing on Milford Bay Road, approximately 240 m south of Highway 118 West, in the Township of Muskoka Lakes.

The purpose of the investigation was to reveal the subsurface conditions and to determine the engineering properties of the disclosed soils for the design and construction of the proposed bridge replacement.

The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The Township of Muskoka Lakes is situated on Precambrian Shield, where hard, erratic and plastic metasedimentary rocks occur as outcrops, or at variable depths as bedrock. Numerous swamps and lakes occupy the bedrock basin. Sand, silts, glacial outwash and drift fill generally occur above the bedrock valley.

The investigation was carried out on Milford Bay Road, near the northeast and southwest corners of an existing concrete bridge crossing, where the watercourse under the bridge is discharging into Lake Muskoka. The road is a paved 2-lane roadway, approximately 6 m in width and the existing bridge crossing is a concrete deck supported on wing walls at both ends. The surrounding area consists of treed lots and houses. The existing site gradient in the vicinity is relatively flat.

The detailed design of the proposed bridge replacement has not been finalized; however, it is our understanding that the structure will be replaced by a crossing consisting of a series of culverts, 1.8 m in diameter, having the inverts at El. 224.25 to 224.50 m.

3.0 FIELD WORK

The field work, consisting of 2 sampled boreholes extending to depths of 9.8 m and 6.7 m, was performed on September 25, 2018, at the locations shown on the Borehole Location Plan, Drawing No. 1. Each borehole was advanced to the sampling depths by a track-mounted continuous-flight hollow-stem auger machine equipped for soil sampling. Standard Penetration Tests, using the procedures described on the enclosed “List of Abbreviations and Terms”, were performed at the sampling depths. The test results are recorded as the Standard Penetration Resistance (or ‘N’ values) of the subsoil. The relative density of the granular strata and the consistency of the cohesive strata are inferred from the ‘N’ values. Split-spoon samples were recovered for soil classification and laboratory testing.

Due to the saturation of subsoil and possible disturbance by hydrostatic pressure, dynamic cone penetration tests (DCPT) were performed in the probe holes located beside the boreholes, in the soil stratum below 1.5 m from grade. The DCPT extended to depths of 10.7 m and 13.1 m where relatively high blow count was recorded. The DCPT readings are plotted on the Borehole Logs, comprising Figures 1 and 2, and they were compared with the recorded Standard Penetration ‘N’ values in the boreholes.

The ground elevation at each borehole location was determined using the bench mark (Top of Spike in East Face of Hydropole in front of 1022 Milford Bay Road), as shown on Drawing No. 1. It has a geodetic elevation of El. 227.43 m, as provided in the Site Plan prepared by C.C. Tatham & Associates Ltd.

4.0 SUBSURFACE CONDITIONS

The investigation has disclosed that beneath the road pavement and a layer of earth fill, consisting of sand and silty sand, the area is underlain by an alluvium deposit, overlying a silt stratum at a depth of 4.6 m from grade. Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 and 2. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

4.1 Pavement Structure

The existing road pavement at Borehole 1 location consists of 100 mm of asphaltic concrete overlying a layer of granular fill, 200 mm in thickness. At Borehole 2 location, the road pavement consists of 65 mm of asphaltic concrete overlying a layer of crushed asphalt, 465 mm in thickness. The subgrade consists of sand fill material.

Further sampling and testing of the granular fill will be required to determine if it is suitable for reuse as granular bases for the new pavement construction.

4.2 Earth Fill

A layer of earth fill, consisting of fine silty sand, with occasional peat inclusions, wood fragments and rootlets, was contacted below the pavement subgrade and extending to a depths of 2.1± m and 2.9± m from grade, in Boreholes 1 and 2, respectively.

The obtained ‘N’ values in the sand fill range from 3 to 28 blows per 30 cm of penetration. The relatively high ‘N’ values were restricted at the subgrade level where the earth fill could have been compacted at the time of the road construction.

The water content of the earth fill samples was determined, and the results are plotted on the Borehole Logs. The values range from 3% to 40%, with a median of 17%, indicating the sand fill is in moist to wet conditions. The high water content values may also indicate the presence of organic materials. However, the sand fill is water-bearing below a depth of 2.5± m from the prevailing ground surface.

One must be aware that the samples retrieved from boreholes 10 cm in diameter may not be truly representative of the geotechnical and environmental quality of the fill, and do not indicate whether the topsoil beneath the earth fill was completely stripped. This should be further assessed by laboratory testing and/or test pits.

4.3 **Alluvium**

An alluvium, consisting of silt and sand with remnants of plant debris, was encountered below the earth fill. It extends to a depth of 4.6± m from the existing ground surface. The alluvium also contains fine fibrous decayed vegetation, which is formed by the progressive accumulation of incompletely decomposed plants in a wet environment.

The water content values of the alluvium samples range from 26% to 31%, indicating wet conditions, with organic materials. Due to the organic content, the alluvium may be subjected to long-term settlement.

4.4 **Silt**

The native silt deposit was contacted beneath the alluvium at a depth of 4.6± m from the road pavement. It contains a trace of clay and is laminated with sand seams and layers. Grain size analyses were performed on 2 representative samples and the results are plotted on Figure 3.

The natural water content of the silt samples was determined and the results are plotted on the Borehole Logs; the values range from 20% to 28%, with a median of 21%, indicating a saturated condition. The samples displayed appreciable dilatancy, indicating that the shear strength is susceptible to impact disturbance.

The obtained 'N' values range from 0 to 3 blows per 30 cm; they could be a result of disturbance by the hydrostatic pressure or suction effect during augering and sample retrieval. According to the DCPT records below a depth of 1.5 m, the relative density of the undisturbed silt deposit is in the compact range. At a depth of 10.7 m and 13.1 m from grade, at the locations of Boreholes 1 and 2, respectively, a hard stratum is apparent from the resistance in DCPT.

Based on the above findings, the deduced engineering properties pertaining to the project are given below:

- High frost susceptibility, with high soil-adfreezing potential.
- High water erodibility; it is susceptible to migration through small openings under seepage pressure.
- Semi-pervious, with an estimated coefficient of permeability of 10^{-4} cm/sec and runoff coefficients of:

Slope

0% - 2%	0.07
2% - 6%	0.12
6% +	0.18

- A frictional soil with slight cohesion, its shear strength is derived from internal friction, which is density dependent. The strength of the wet silt is susceptible to impact disturbance; i.e., the disturbance will induce a build-up of pore pressure within the soil mantle, resulting in soil dilation and a reduction of shear strength.
- In excavation, the saturated silt will slough and run slowly with seepage bleeding from the cut face. It will boil under a piezometric head of 0.3 m or less.

4.5 Compaction Characteristics of the Revealed Soils

The obtainable degree of compaction is primarily dependent on the soil moisture and, to a lesser extent, on the type of compactor used and the effort applied. As a general guide, the typical water content values of the revealed soils for Standard Proctor compaction are presented in Table 1.

Table 1 - Estimated Water Content for Compaction

Soil Type	Determined Natural Water Content (%)	Water Content (%) for Standard Proctor Compaction	
		100% (optimum)	Range for 95% or +
Sand Fill/Alluvium	3 to 40 (median 17)	10	6 to 13
Silt	20 to 28 (median 21)	12	8 to 15

Based on the above findings, the on site material below a depth of 1 m is generally too wet for compaction. The wet material will require aeration in dry and warm weather. The earth fill and the alluvium must be sorted free of topsoil, peat inclusions and other deleterious materials, if possible, prior to its reuse as structural backfill.

The backfill can be compacted by a smooth roller with or without vibration, depending on the water content of the soils being compacted. The lifts for compaction should be limited to 20 cm, or to a suitable thickness as assessed by test strips performed by the equipment which will be used at the time of construction.

If the compaction of the soils is carried out with the water content on the wet side of the optimum, the surface of the compacted soil mantle will roll under the dynamic compactive load. This is unsuitable for road construction since each component of the pavement structure is to be placed under dynamic conditions which will induce the rolling action of the subgrade surface and cause structural failure of the new pavement. It is recommended the backfill below the road subgrade should be compacted on the dry side of the optimum water content.

5.0 GROUNDWATER CONDITIONS

At the time of investigation, the water level in Lake Muskoka was at El. 225.2 m.

During the borehole drilling operation and upon the completion of drilling, the water level and cave-in depths were recorded in the boreholes. The data are plotted on the Borehole Logs and listed in Table 2.

Table 2 - Groundwater Levels

Borehole No.	Ground Elevation (m)	Seepage Encountered During Augering		Groundwater/Cave-in* Level On Completion	
		Depth (m)	Amount	Depth (m)	Elevation (m)
1	226.9	2.3	Appreciable	1.7/2.7*	225.2/224.2*
2	226.9	1.5	Appreciable	1.5/1.5*	225.4/225.4*

Free groundwater was recorded in the boreholes at 1.5 m and 1.7 m from grade, or El. 225.2 m and 225.4 m. It is very close to the water level in Lake Muskoka.

In excavation, the groundwater yield is expected to be persistent due to the proximity to Lake Muskoka. Any excavation must be enclosed by sheet piles or coffer dams and the watercourse must be rerouted from the area of excavation.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath the road pavement and a layer of earth fill, the area is underlain by an alluvium deposit, overlying a saturated silt stratum at a depth of 4.6 m from grade. The silt displayed appreciable dilatancy, indicating that the shear strength is susceptible to impact disturbance.

Free groundwater was recorded in the boreholes at 1.5 m and 1.7 m from grade, or El. 225.2 m and 225.4 m. It is very close to the water level in Lake Muskoka.

The revealed findings show that the following geotechnical considerations require special attention:

1. After removal of the asphalt pavement, organic free granular fill may be stockpiled for reuse as backfill material. For reuse as the granular bases in the new pavement construction, further sampling and testing of the granular fill will be required to determine if it meets the OPS specifications.
2. The existing earth fill should be sorted of any deleterious material before reuse in backfilling in the road subgrade.
3. All the new structures and bedding must be placed below the scouring depth or the subgrade is protected from erosion or scouring.
4. Long-term settlement in the alluvium is anticipated. Regular maintenance of the structure is necessary if the alluvium will be left in place beneath the new structure.
5. Other movement is also anticipated in the subgrade soils due to the annual freeze and thaw cycle in the subsoil.

6. Geotextile reinforcement in the subgrade is recommended beneath the culverts and crossing. The application of geotextile can reduce the risk of damage due to differential movement in the subgrade. The geotextile must be further assessed by a qualified consultant who specializes in geotextile design.
7. The backfill of the new culverts should consist of granular fill or consolidated earth fill. It must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the condition of the subgrade is compatible for road construction.
8. Any excavation below the water level will lead to base heaving. Open excavation will require appropriate dewatering, temporary channel diversion and/or confinement of sheeting.

The recommendations appropriate for the project described in Section 2.0 are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should this become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

6.1 Bridge Replacement

The detailed design of the proposed bridge replacement was not available at the time of report preparation. It is understood from C.C. Tatham & Associates Ltd. that the replacement consists of a series of culverts, 1.8 m in diameter, installed at El. 224.25 to 224.50 m, where alluvium is anticipated in the subgrade.

Due to the organic content, the alluvium may be subject to long-term settlement. Regular maintenance of the new culverts and road structure is necessary if the alluvium will be left in place beneath the new structure. In addition, seasonal movement is also anticipated in the subgrade soils due to the annual freeze and thaw cycle in the subsoil. Geotextile reinforcement is recommended beneath the culverts and crossing to reduce the risk of road damage due to differential movement in the subgrade. The geotextile must be further assessed by a qualified consultant who specializes in geotextile design.

Upon exposure of the subgrade soil, it must be inspected by a geotechnical engineer, or a senior technician under the supervision of a geotechnical engineer, to ensure that the revealed conditions are compatible with design requirements. The proposed culverts must be placed on a bedding consisting of non-frost susceptible and non-erodible rip-rap stone or gravel bedding of 50+ mm, with filter fabric between the bedding and the subgrade soil. The thickness of the bedding should be specified to prevent scouring of the subgrade soil in both the upstream and downstream sides of the culverts.

Backfill around the new culverts should consist of granular fill or consolidated earth fill, compacted simultaneous on all sides of the culverts. The lifts of each backfill layer should be limited to 20 cm, or to a suitable thickness as assessed by test strips performed by the equipment which will be used at the time of construction. The last 1 m of the road subgrade should be compacted to 98% or + of its maximum Standard Proctor dry density, with the water content close to its optimum. The compaction of backfill must be inspected by a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the condition of the subgrade is compatible for road construction.

As an alternative, the bridge crossing may be replaced by a new bridge structure on wing walls and supported by helical piers extending into the hard stratum. The design load of helical piers is directly related to the installation torque of the anchor in the underlying competent soil stratum. The optimum loads, the depths of the piles and the cost of the project should be assessed by prospective foundation contractors in these specialties. Additional boreholes will be required to explore the soils at the deeper level for the design of helical piers or other deep foundations.

The wall foundations and grade beams exposed to weathering and the freezing temperature should have at least 1.8 m of earth cover for protection against frost action.

The culvert and bridge design must meet the requirements specified in the latest Ontario Highway Bridge Design Code.

6.2 **Embankment Construction**

The embankment must consist of inorganic soils, compacted uniformly to at least 98% of its maximum Standard Proctor dry density. If the subgrade is relatively loose or soft, it should be stabilized with 50-mm Crusher-Run Limestone, or equivalent, using the stone immersion technique.

The sides of embankment must be sloped properly for stability and sodded to protect against rainwash erosion.

6.3 Pavement Restoration

The recommended pavement restoration, upon completion of the bridge replacement, is presented in Table 3. The existing asphalt can be pulverized and mixed with the existing or new granular fill for use as granular bases provided that it meets the OPS gradation requirements and the asphalt content should not exceed 30% by weight in the granular bases.

Table 3 - Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-4
Asphalt Binder	60	HL-8
Granular Base	150	Granular ‘A’ or equivalent
Granular Sub-Base	350	Granular ‘B’ or equivalent

The pavement subgrade should be proof-rolled and the final subgrade should be inspected and assessed by a geotechnical technician or engineer. Any soft spots should be subexcavated and replaced with well compacted similar subgrade material. The final subgrade should be properly crowned and smooth-rolled to allow any percolated water to be properly drained away from the pavement structure.

All the granular bases should be compacted to their maximum Standard Proctor dry density in lifts of 200 mm or less.

The thickness of the new pavement structure, however, may not match with the existing pavement. We recommend that the thickness of each layer and the subgrade should be tapered into the existing pavement at 1 Vertical:3+ Horizontal. A step joint should be provided at the top coat of asphalt at the junction to the existing pavement.

6.4 Soil Parameters

The recommended coefficients and soil parameters are presented in Table 4.

Table 4 - Soil Parameters

Unit Weight and Bulk Factor	**Unit Weight γ (kN/m^3)**		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Granular Fill	22.0	12.5	1.20	1.00
Earth Fill and Alluvium	20.5	10.5	1.20	0.95
Native Silt	21.5	11.5	1.20	1.00
Effective Shear Strength Parameters		**Cohesion c' (kPa)**	**Angle of Internal Friction, ϕ'**	
Compacted Granular 'A'		0	35°	
Compacted Granular 'B'		0	32°	
Compacted Earth Fill (Free of Organics)		0	28°	
Alluvium/Native Silt		0	25°	
Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_o**	**Passive K_p**	
Compacted Granular Fill	0.30	0.45	3.30	
Compacted Earth Fill and Native Soils	0.40	0.55	2.50	
Coefficients of Friction				
Between Concrete and Granular Base			0.50	
Between Concrete and Subsoils			0.35	

6.5 Excavation

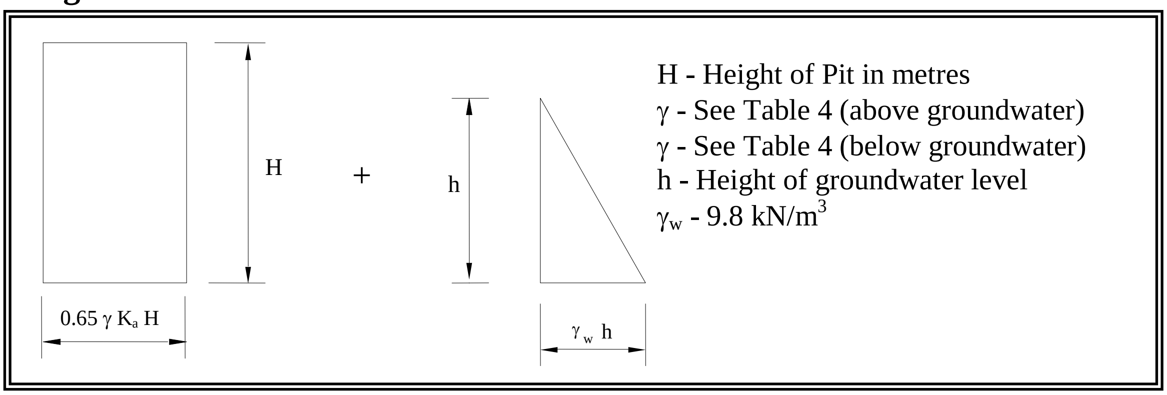
Excavation must be carried out in accordance with Ontario Regulation 213/91. For excavation purposes, the types of soils are classified in Table 5.

Table 5 - Classification of Soils for Excavation

Material	Type
Existing Granular Fill, drained Soils	3
Saturated Fill, Alluvium and Silt	4

Any excavation below the water level will lead to base heaving. Open excavation will require appropriate dewatering, temporary channel diversion and/or confinement of sheeting. In the design of the sheet piling and/or shoring structure, the recommended lateral earth pressure distribution for the revealed soils is given in Diagram 1.

Diagram 1 - Lateral Earth Pressure



In calculating the lateral earth pressure for the shoring structure, the soil parameters are provided in Section 6.4.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Township of Muskoka Lakes, and for review by the designated consultants, agencies and contractors.

Use of the report is subject to the conditions and limitations of the contractual agreement. The material in the report reflects the judgment of Kelvin Hung, B.A.Sc., and Bennett Sun, P.Eng., in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, and/or any reliance on decisions to be made based on it are the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

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PROVINCE OF ONTARIO

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS	Auger sample
CS	Chunk sample
DO	Drive open (split spoon)
DS	Denison type sample
FS	Foil sample
RC	Rock core (with size and percentage recovery)
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows for each foot of penetration of a 2-inch diameter, 90° point cone driven by a 140-pound hammer falling 30 inches.

Plotted as '—•—'

Standard Penetration Resistance or 'N' Value:

The number of blows of a 140-pound hammer falling 30 inches required to advance a 2-inch O.D. drive open sampler one foot into undisturbed soil.

Plotted as '○'

WH	Sampler advanced by static weight
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
NP	No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/ft)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

Cohesive Soils:

Undrained Shear Strength (ksf)	'N' (blows/ft)	Consistency
less than 0.25	0 to 2	very soft
0.25 to 0.50	2 to 4	soft
0.50 to 1.0	4 to 8	firm
1.0 to 2.0	8 to 16	stiff
2.0 to 4.0	16 to 32	very stiff
over 4.0	over 32	hard

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0 Field vane test in borehole; the number denotes the sensitivity to remoulding

△ Laboratory vane test

□ Compression test in laboratory

For a saturated cohesive soil, the undrained shear strength is taken as one half of the undrained compressive strength

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres	1 inch = 25.4 mm
1lb = 0.454 kg	1ksf = 47.88 kPa

JOB NO.: 1808-S134

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

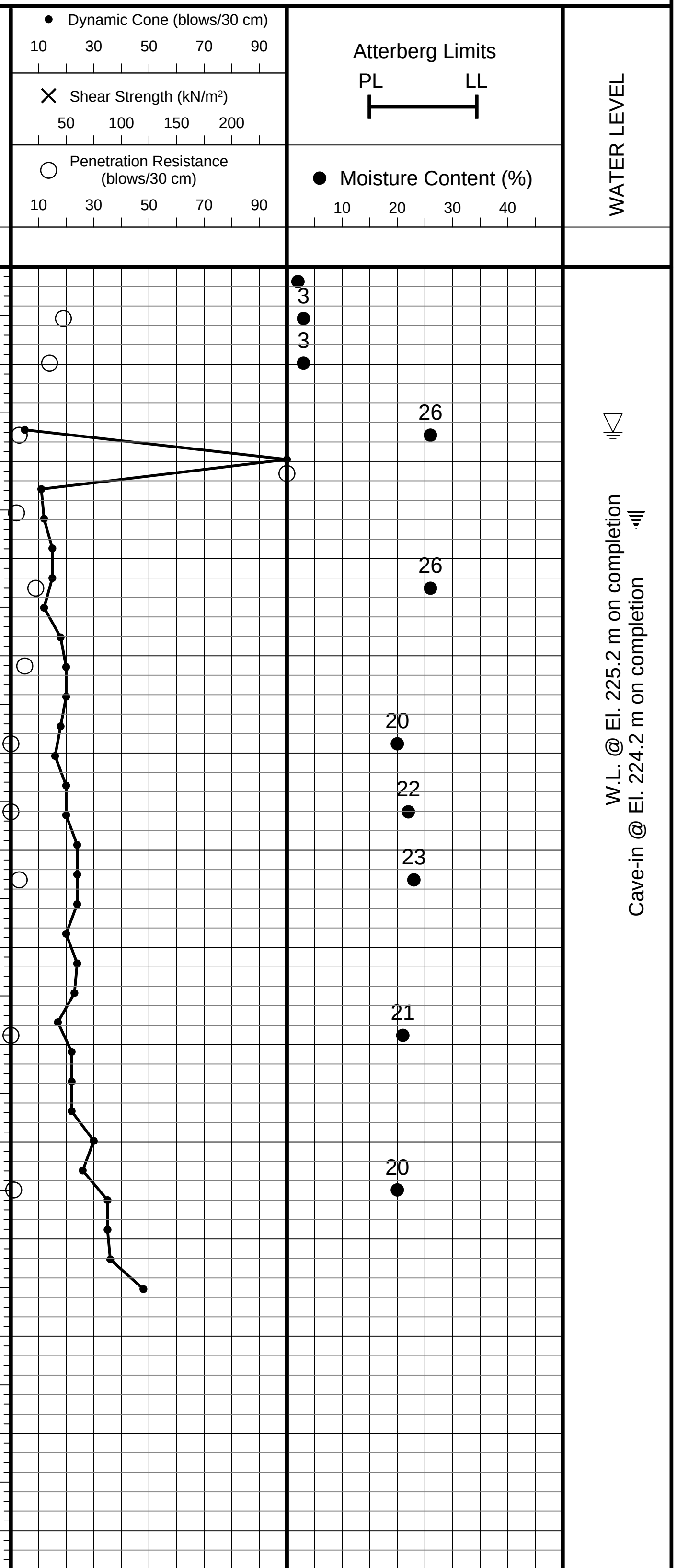
PROJECT DESCRIPTION: Proposed Bridge Replacement

METHOD OF BORING: Hollow-Stem/ Dynamic Cone

PROJECT LOCATION: Milford Bay Road, Township of Muskoka Lakes

DRILLING DATE: September 25, 2018

El. (m) / Depth (m)	SOIL DESCRIPTION	Number	Type	N-Value	Depth Scale (m)
226.9	Pavement Surface				
0.0	**100 mm ASPHALTIC CONCRETE**	1	AS	-	0
	200 mm GRANULAR FILL				
	Brown	2	DO	19	
	EARTH FILL sand to silty sand a trace of gravel	3	DO	14	1
	with peat seams	4	DO	3	
224.8 / 2.1	**WOOD FRAGMENTS**	5	DO	60/5	2
	Brown/grey	6	DO	2	
	ALLUVIUM sand with rootlets	7	DO	9	3
		8	DO	5	4
222.3 / 4.6	Grey, very loose, distubed	9	DO	0	5
		10	DO	0	
		11	DO	3	6
	SILT				7
	some sand, a trace of clay occ. sand seams and layers	12	DO	0	8
		13	DO	1	9
217.1 / 9.8	**END OF BOREHOLE**				10
	CONE PENETRATION TESTS				
216.2 / 10.7	**END OF CONE PENETRATION TESTS**				11



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JOB NO.: 1808-S134

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

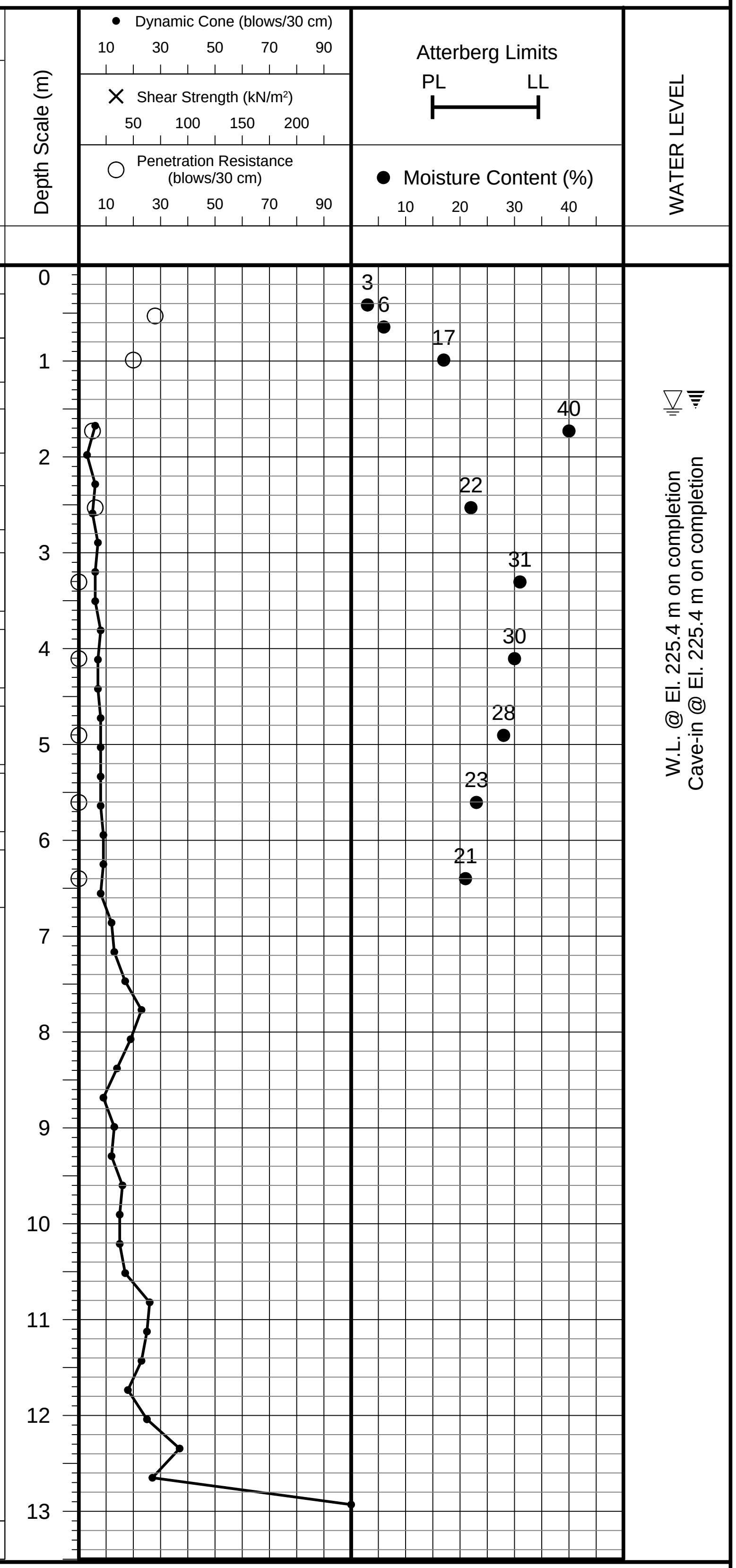
PROJECT DESCRIPTION: Proposed Bridge Replacement

METHOD OF BORING: Hollow-Stem/ Dynamic Cone

PROJECT LOCATION: Milford Bay Road, Township of Muskoka Lakes

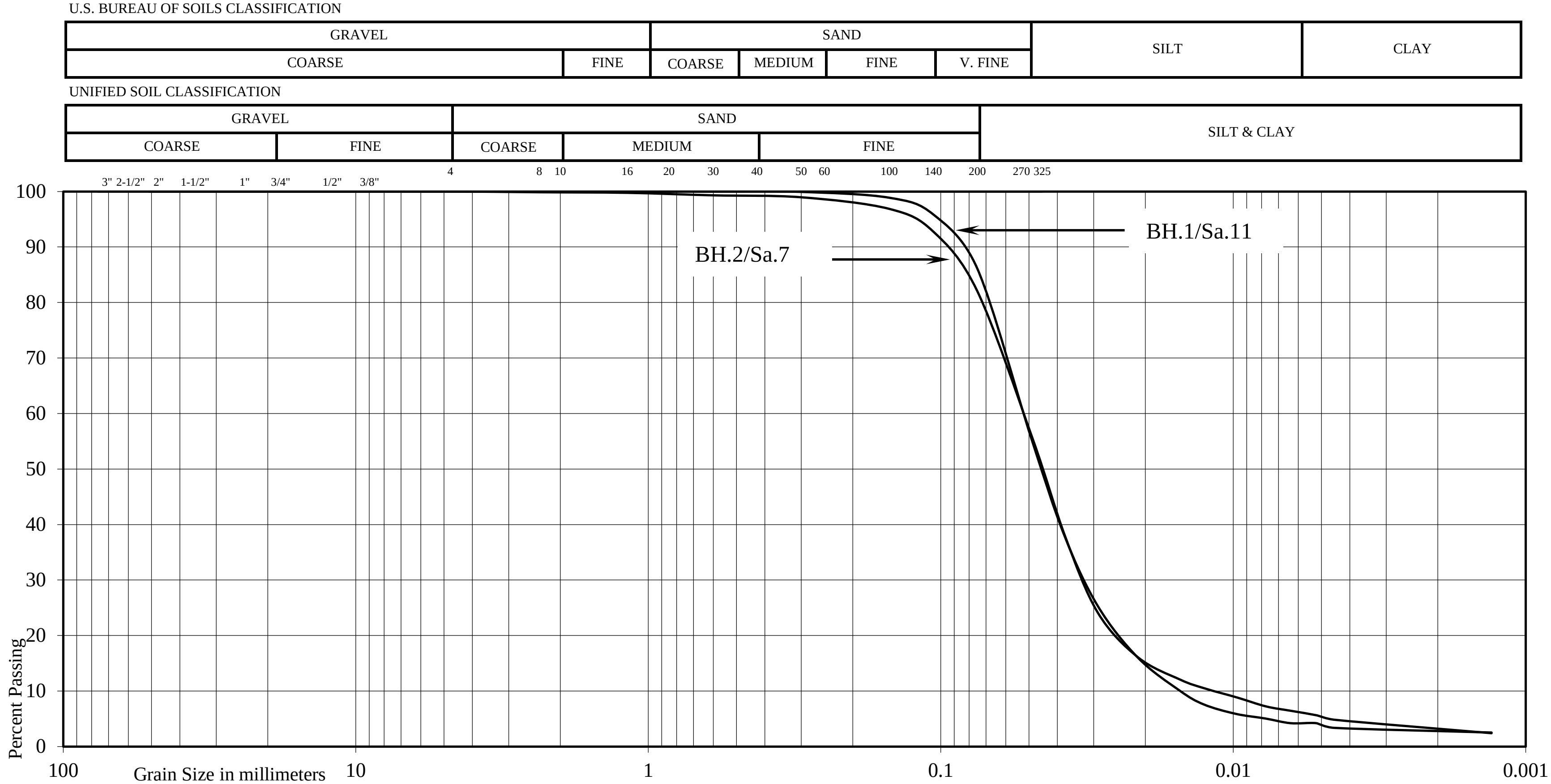
DRILLING DATE: September 25, 2018

El. (m) / Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value
226.9	Pavement Surface			
0.0	**65 mm ASPHALTIC CONCRETE**			
	465 mm PULVARIZED ASPHALT	1A / 1B	DO	28
	Brown/grey	2	DO	20
	EARTH FILL			
	sand to silty sand, occ. wood fragments and rootlets — with peat seams	3	DO	5
		4	DO	6
224.0 / 2.9	Dark brown	5	DO	0
	ALLUVIUM			
	sand with rootlets	6	DO	0
222.3 / 4.6	Grey, very loose, distubed	7	DO	0
	SILT	8	DO	0
	some sand, a trace of clay, occ. sand seams and layers	9	DO	0
220.2 / 6.7	**END OF BOREHOLE**			
	CONE PENETRATION TESTS			
213.8 / 13.1	**END OF CONE PENETRATION TESTS**			



GRAIN SIZE DISTRIBUTION

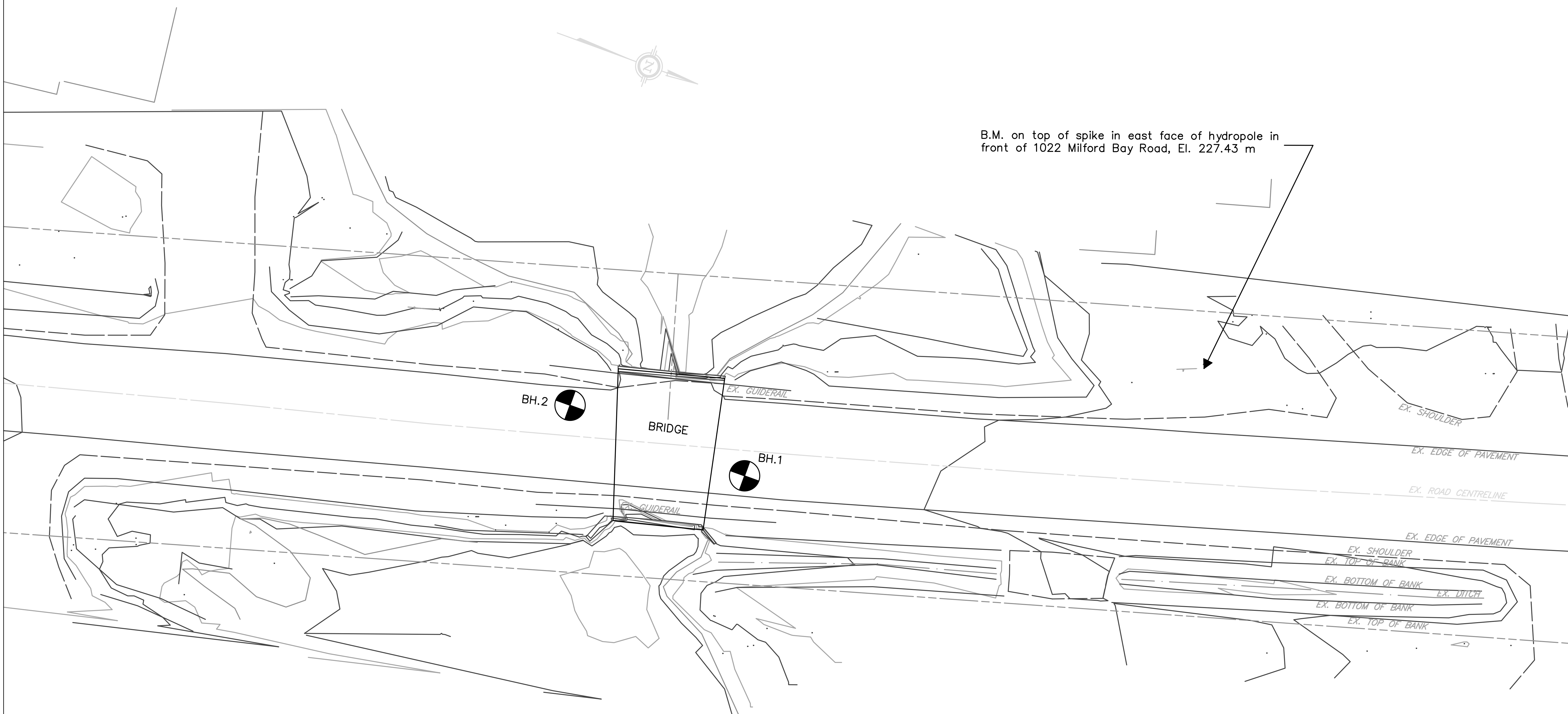
Reference No: 1808-S134



Project:	Proposed Bridge Replacement			BH./Sa.	1/11	2/7
Location:	Milford Bay Road, Township of Muskoka Lakes			Liquid Limit (%) =	-	-
				Plastic Limit (%) =	-	-
Borehole No:	1	2		Plasticity Index (%) =	-	-
Sample No:	11	7		Moisture Content (%) =	23	28
Depth (m):	6.3	4.9		Estimated Permeability		
Elevation (m):	220.6	222.0		(cm./sec.) =	10^{-4}	10^{-4}

Classification of Sample [& Group Symbol]: SILT, some fine sand, a trace of clay

Figure: 3



LEGEND

⊕ – Borehole Location

Soil Engineers Ltd.
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BOREHOLE LOCATION PLAN

SITE: Milford Bay Road, Township of Muskoka Lakes

DESIGNED BY: –	CHECKED BY: –	DWG NO.: 1	
SCALE: 1: 250	REF. NO.: 1808–S134	DATE: October 2018	REV –

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE

SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.: 1808-S134
REPORT DATE: October 2018
PROJECT DESCRIPTION: Proposed Bridge Replacement
PROJECT LOCATION: Milford Bay Road, Township of Muskoka Lakes

LEGEND

ASPHALT FILL ALLUVIUM WOOD
GRANULAR SILT

WATER LEVEL (END OF DRILLING) CAVE-IN

BH No.:	1	2
El. (m):	226.9	226.9