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GEOTECHNICAL INVESTIGATION REPORT

Bay Street Bridge Replacement

Magnetawan, ON

TULLOCH Document No. / Rev. 26-0649-2050-001 / Rev. 0
April 2026

ISSUED FOR USE

					
April 10, 2026	0	FINAL	E. Mighton	E. Giles	G. Liang
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26-0649-2050-001



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April 10, 2026
Project # 26-0649

Municipality of Magnetawan

4303 Highway 520
Magnetawan, ON
P0A 1P0

Attention: Mr. Scott Edwards | CRS-S, Public Works Superintendent

Re: Geotechnical Services for the Proposed Bay Street Bridge Replacement,
Magnetawan, Ontario

Dear Mr. Scott:

Please find enclosed our Geotechnical Report for the proposed replacement of the Bay Street Bridge, located 435 m west of Parks Street on Bay Street in the community of Magnetawan, Ontario.

This report outlines the results of the geotechnical investigation and provides geotechnical recommendations to aid in the design of the proposed structure replacement.

We trust the enclosed is adequate for your current needs. If there is anything further that we can assist with, please contact us at your convenience.

Sincerely,

Erik Giles, P.Eng.
Geotechnical Engineer
TULLOCH Engineering, Inc.

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1. INTRODUCTION AND SCOPE

TULLOCH Engineering Inc. (TULLOCH) was retained by The Municipality of Magnetawan (Client) to complete a geotechnical site investigation as part of the Bay Street Bridge Replacement project. The project site is located on Bay Street approximately 435 m west of the Sparks Street intersection, in the community of Magnetawan, Ontario. The existing Bay Bridge is a two-lane, cast-in-place concrete deck overlain on a laminated timber deck on timber pile bents with timber retaining walls on either abutment. The bridge was constructed in 1960 and is reaching the end of its serviceable lifespan. Based on the findings from the Ontario Structure Inspection Manual (OSIM) inspection titled Bridge #10 – Bay Street Bridge conducted by Kevin Hawley in 2023, the bridge was recommended to be replaced.

The purpose of this geotechnical investigation was to evaluate the subsurface conditions near the existing bridge abutments and provide foundation engineering recommendations for its replacement. This report presents the factual geotechnical data obtained during the field investigation and provides geotechnical design recommendations that are based on the subsurface conditions encountered, our understanding of the project scope, and our engineering experience. A site plan illustrating the borehole locations for the investigation can be found in Appendix A. Common terminology used in this report can be found in Appendix B, while specific terminology will be referenced in table notes or within the report body.

2. REGIONAL GEOLOGY AND SITE INFORMATION

Based on a review of public geological resources and maps of Ontario as published by the Ontario Geological Survey, the site is located within the intersection between two surficial geology areas that are comprised of subsurface soils associated with coarse-textured glaciolacustrine deposits to the west and fine-textured glaciolacustrine deposits to the east. Accordingly, materials that underlay this area consist primarily of sands and gravels with minor silt and clay constituents towards the west of the site and silt and clay with minor constituents of sand and gravel towards the east. The regional bedrock is reported to consist of migmatitic rocks and gneisses of underdetermined protolith, including quartzofeldspathic gneisses, orthogneisses and paragneisses (OGS 2010, 2011).

The site is situated in a generally low-lying area with very little elevation change between the abutments of the existing Bay Street bridge crossing. The banks of the river are well vegetated and generally well-defined within the river channel. A site photograph log documenting the site and boreholes at the time of the investigation is provided in Appendix C of this report.

3. SITE INVESTIGATION AND METHODOLOGY

The geotechnical investigation was completed between March 2 – 4, 2026, and consisted of advancing a total of four (4) geotechnical sampled boreholes. Borehole locations were targeted at each bridge abutment and along each approach, with boreholes advanced as close to the structure without conflict with the bridge and in a safe location for the drilling crew. Boreholes in this report are referred to as BH-26-01 through BH-26-04.

Borehole sampling was advanced to a depth of ranging from approximately 11.5 to 12.8 meters below ground surface (mbgs) in BH-26-01, -02 and -04, and to an approximate depth of 4.4 mbgs in BH-26-03. Bedrock was confirmed in BH-26-01 and BH-26-02 at each abutment of the bridge.

All boreholes were positioned and field fit to minimize traffic impact and to avoid underground utilities located on the site under the direction of a TULLOCH geotechnical representative. Borehole locations were recorded using a handheld GPS with an accuracy of ± 3.0 m. Borehole elevations were taken from recent topographical survey data associated with the bridge replacement design prior to the investigation. The following table summarizes the borehole investigation. Borehole locations can also be viewed in the site plan provided in Appendix A of this report.

Table 3-1: Summary of Borehole Information

Borehole No.	Easting (m) ¹	Northing (m) ¹	Borehole Surface Elevation (masl) ¹	Bedrock Elevation (masl) ¹	Drilling Depth (mbgs) ²
BH-26-01	605257	5057511	281.32	273.24	11.46
BH-26-02	605247	5057500	281.10	272.01	12.14
BH-26-03	605266	5057510	281.28	276.86	4.42
BH-26-04	605235	5057500	281.10	268.38	12.75

Note(s): ¹ Elevation data has been approximated from the topographic survey completed by TULLOCH. Borehole coordinates were taken via handheld GPS with accuracy ± 3 m. ² meters below ground surface.

Boreholes were advanced using a CME-55 truck-mounted rotatory drill rig owned and operated by Landcore Drilling from Chelmsford, Ontario. Borehole advancement was generally conducted with 200 mm OD diameter, continuous flight, hollow stem augers. The drill rig was equipped with standard split spoon soil sampling equipment, including a full weight automatic hammer.

In the overburden, soil samples were obtained using standard split spoon equipment in conjunction with Standard Penetration Tests (SPT) performed in accordance with ASTM D1586.

SPT sampling generally occurred at 0.76 m intervals in the upper 3.0 m of borehole advancement and 1.52 m intervals thereafter.

Bedrock was confirmed via coring at BH-26-01 and BH-26-02 using diamond rotary drilling with an NQ2 (75.7 mm OD) rock core barrel advanced within HW casing, in general accordance with ASTM D2113. Core runs were typically completed in continuous 1.5 m run intervals, until competent bedrock was recovered. Bedrock lithology and general characteristics were logged in the field, and Rock Quality Designation (RQD), Total Core Recovery (TCR) and Solid Core Recovery (SCR) were determined at the time of core retrieval.

In BH-26-03 and BH-26-04, the bedrock surface was inferred due to split spoon and auger refusal, at which point, the borehole was terminated. confirmatory rock coring was considered out of scope for these two (2) locations.

The soil sampling program was directed by a TULLOCH representative, who logged the drilling operations and identified the soil samples and rock cores as they were retrieved. The samples were classified using the soil classification system of USCS and the Canadian Foundation Engineering Manual, refer to Appendix B for terminology details. The recovered soil samples were transported to TULLOCH's CCIL Certified Laboratory in Sault Ste. Marie, Ontario, for detailed examination and testing. All samples will be stored at the laboratory for three (3) months and then disposed of unless directed otherwise. Selected soil samples were also taken to TestMark Laboratories in Garson, Ontario, for soil corrosivity testing. Detailed borehole logs can be found in Appendix D of this report.

4. LABORATORY TESTING PROGRAM

A geotechnical laboratory testing program was undertaken on representative samples taken from the boreholes and conducted in accordance with ASTM standards. Table 4-1 provides a list of the testing program. Detailed laboratory reports can be found in Appendix E of this report.

Table 4-1: Summary of Soil and Rock Laboratory Testing Program

Test	Number of Tests	Standards
Moisture Content	13	ASTM D2216
Washed Sieve Analysis	6	ASTM D6913
Atterberg Limits Test	2	ASTM D4318
Unconfined Compression Testing (rock)	3	ASTM D7012
Corrosivity Analysis ¹	2	Various

Note(s): ¹ Subcontracted Laboratory performed testing.

5. SUBSURFACE CONDITIONS

5.1. General

Subsurface conditions from the borehole stratigraphy encountered at the project site are summarized below. Detailed borehole logs and laboratory testing reports can be seen in Appendix D and E, respectively. It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. Each soil layer will be discussed as it was encountered from ground surface during the investigation.

5.1.1. Asphalt

Existing asphalt was encountered on the road surface at all borehole locations. The asphalt was measured to be approximately 25 mm thick at all borehole locations.

5.1.2. Existing Granular Road Base/Sub-Base Fills

Beneath the asphalt in boreholes BH-26-01, -02, and -04, an existing sandy fill material was encountered. The fill was observed to contain varying silt and gravel content across the boreholes. The road base/sub-base was approximately 2.3 m in thickness at BH-26-01 and BH-26-02 and approximately 2.2 m in BH-26-04. The material was well to poorly graded with fine to coarse grained sand and gravel, and some to trace non-plastic fines. The material was observed to be brown to dark brown in colour and demonstrated non-cohesive behaviour. Field moisture observations of retrieved split spoon samples indicated the material was frozen to moist at the time of the investigation. Typically, grab samples retrieved from auger cuttings were obtained during the investigation near ground surface, and SPT 'N' values in this material ranged from 19 to greater than 100 blows per 30 cm of sampler advancement, indicating a compact to very dense material density. It should be noted that frozen material could have inflated the SPT 'N' values recorded in this fill material. During borehole advancement during examination of the fill, due to its age and potential mixing/contamination over time, it was not possible to distinguish between road base and sub-base fill.

Laboratory testing conducted on representative samples of the material yielded moisture contents ranging from 7.9% to 12.8%, with an average of 10.9%.

A representative sample of the existing fill was also taken for particle size distribution testing and compared to the Granular B Type I envelope specified in OPSS 1010 and failed to conform due

to high fines content. The particle size distribution breakdown of the major soil constituents for the material is shown below in Table 5-1. A particle size distribution report is attached with the laboratory data in Appendix E.

Table 5-1: Particle Size Distribution Summary – Granular Road Base/Subbase Fill

Borehole No.	Sample No.	Size Fraction (%)		
		Gravel	Sand	Silt/Clay
BH-26-01	SS01	23.6	61.0	15.4

5.1.3. (OH) ORGANIC CLAYEY SILT to (MH) CLAYEY SILT

A deposit of organic clayey silt to clayey silt was encountered beneath the existing asphalt in BH-26-03. The deposit was encountered to a maximum depth of approximately 1.5 mbgs. The clayey silt deposit was typically found to demonstrate cohesive behavior with high plasticity. The material was observed to be dark brown to grey in colour. Grab samples retrieved from auger cuttings were obtained during the investigation within this deposit. Field moisture observations of retrieved grab samples indicated the material's water content was near or above the material's plastic limit. Granular fill was not encountered beneath the asphalt surface in BH-26-03.

5.1.4. (CL) SILTY CLAY

A deposit of silty clay was encountered beneath the silty sand in BH-26-02 and beneath the clayey silt in BH-26-03. The deposit ranged in thickness from approximately 0.7 to 2.0 m. The material was observed to be grey in colour and demonstrated cohesive behaviour. Field moisture observations of retrieved split spoon samples indicated the deposit's water content was above its plastic limit. SPT 'N' values in this material ranged from weight of hammer (BH-26-02) to 30 blows per 30 cm of sampler advancement, indicating a very soft to very stiff material consistency.

A shear vane was advanced within this deposit in BH-26-02, resulting in an in-situ undrained shear strength of approximately 32 kPa and a sensitivity of 2.7, indicating a firm material consistency.

Laboratory testing conducted on representative samples of the material yielded moisture contents ranging from 25.4% to 51.5%, with an average of 38.5%.

An Atterberg limits test conducted on a representative sample of the deposits, yielded a liquid limit of 32, a plastic limit of 22 and a plasticity index of 10, indicating a medium to low plasticity material.

5.1.5. (ML) SILT

A silt deposit was encountered beneath the clayey silt deposit in BH-26-03. The deposit was approximately 0.8 m thick. The silt deposit was generally found to contain trace fine-grained sand. The material was observed to be grey to brown in colour and demonstrated non-cohesive behaviour. Field moisture observations of retrieved split spoon samples indicated the material was wet at the time of the investigation with free standing water noted in the split spoon samples. One (1) SPT 'N' value in this deposit was recorded at 19 blows per 30 cm of sampler advancement, indicating a compact material density.

Laboratory testing conducted on a representative sample of the material yielded a moisture content of 18.9%.

5.1.6. (PT) SILTY PEAT

A silty peat deposit was encountered below the fill material in BH-26-04. The deposit was approximately 0.8 m thick. The silty peat deposit was typically found to be tactilely non-plastic and retrieved split spoon samples contained wood debris. The material was observed to be dark brown to black in colour and demonstrated non-cohesive behaviour. Field moisture observations of retrieved split spoon samples indicated the silty peat deposit was wet at the time of the investigation. One SPT 'N' value in this deposit was recorded at 3 blows per 30 cm of sampler advancement, indicating a very loose material density. This material may also represent historic topsoil as it was encountered directly below the existing road fills at the site.

5.1.7. (SM) SILTY SAND to (ML-SP) SILT and SAND

An upper silty sand deposit was encountered at approximately 3.0 mbgs in BH-26-02 below the bridge abutment, in BH-26-03 below the silt deposit, and in BH-26-04 below the peat deposit. The upper deposit ranged in thickness from approximately 1.1 to 2.7 m. A lower silt and sand deposit was encountered in BH-26-02 below the silty clay deposit, and a lower silty sand deposit was encountered in BH-26-04 below the sand deposit. The lower deposits ranged in thickness from 0.9 to 3.0 m. The silty sand to silt and sand contained predominantly fine-grained sand, with fine to coarse-grained sand observed in the lower BH-26-02 deposit. The material contained trace to no fine to coarse-grained gravel, with wood debris noted in the upper silty sand deposit at BH-26-02. The deposit was observed to be dark brown to grey in colour and demonstrated non-cohesive behaviour. Field moisture observations of retrieved split spoon samples indicated the material was wet at the time of the investigation with free standing water observed in retrieved samples.

SPT 'N' values in this deposit ranged from 3 to 32 blows per 30 cm of sampler advancement, indicating a very loose to dense material density.

Laboratory testing conducted on representative samples of the material yielded moisture contents ranging from 15.8% to 55.7%, with an average value of 29.9%.

Three (3) representative samples were also taken for particle size distribution testing. The particle size distribution breakdown of the major soil constituents for the deposit is shown below in Table 5-2. Particle size distribution reports are attached with the laboratory data in Appendix E of this report.

Table 5-2: Particle Size Distribution Summary – Silty Sand to Silt and Sand

Borehole No.	Sample No.	Size Fraction (%)		
		Gravel	Sand	Silt/Clay
BH-26-02	SS05	0.2	46.0	53.8
BH-26-04	SS03	0.0	69.9	30.1
BH-26-04	SS09A	4.2	79.8	16.0

5.1.8. (SP-SW) SAND

A deposit of sand was encountered below the silty sand in BH-26-01 and below the clayey silt in BH-26-04. The deposit ranged in thickness from approximately 1.5 to 2.3 m. The material was found to range from a poorly graded to well graded sand and contained trace to some non-plastic fines and trace fine-grained gravel. The deposit was observed to be red to brown in colour and demonstrated non-cohesive behaviour. Field moisture observation of retrieved split spoon samples indicated the material was wet at the time of the investigation. SPT 'N' values in this deposit ranged from 11 to 27 blows per 30 cm of sampler advancement, indicating a compact material density.

Laboratory testing conducted on a representative sample of the material yielded a moisture content of 24.7%.

One (1) representative sample was also taken for particle size distribution testing. The particle size distribution breakdown of the major soil constituents for the deposit is shown below in Table 5-3. A particle size distribution report is attached with the laboratory data in Appendix E of this report.

Table 5-3: Particle Size Distribution Summary – Sand

Borehole No.	Sample No.	Size Fraction (%)		
		Gravel	Sand	Silt/Clay
BH-26-01	SS06	0.9	94.6	4.5

5.1.9. (ML-CL) CLAYEY SILT

A clayey silt deposit was encountered below the upper silty sand deposit in BH-26-04. The deposit was approximately 4.6 m thick. Below approximately 8.7 mbgs, the deposit was interlayered with silty sand. The clayey silt deposit was found to behave cohesively and with low plasticity, contained wood debris from approximately 5.6 to 8.7 mbgs, and contained fine-grained sand below approximately 8.7 mbgs. The material was observed to be grey in colour and demonstrated non-cohesive behaviour towards the upper section of the deposit, and cohesive behaviour throughout the remaining deposit. Field moisture observation of retrieved split spoon samples indicated the material's water content was above its plastic limit. SPT 'N' values in this deposit ranged from the weight of hammer to 1 blow per 30 cm of advancement, indicating a very soft material consistency.

A shear vane was advanced within this deposit, resulting in an undrained shear strength of 31.6 kPa and a sensitivity of 1.1, indicating a firm material consistency.

Laboratory testing conducted on representative samples of the material yielded moisture contents ranging from 37.3% to 46.6% with an average value of 42.0%.

One (1) representative sample was also taken for particle size distribution testing. The particle size distribution breakdown of the major soil constituents for the deposit is shown below in Table 5-4. A particle size distribution report is attached with the laboratory data in Appendix E of this report.

Table 5-4: Particle Size Distribution Summary – Clayey Silt

Borehole No.	Sample No.	Size Fraction (%)		
		Gravel	Sand	Silt/Clay
BH-26-04	SS06	0.0	1.2	98.8

An Atterberg limits test conducted on a representative sample of the deposits, yielded a liquid limit of 28, a plastic limit of 25, and a plasticity index of 3, indicating low plasticity.

5.1.10. (SW/GW) SAND and GRAVEL

A deposit of sand and gravel was encountered above the inferred bedrock in BH-26-04. The deposit was approximately 150 mm thick. The sand and gravel deposit was generally found to contain well graded fine to coarse-grained sand and gravel and trace non-plastic fines. The material was observed to be brown in colour and demonstrated non-cohesive behaviour. Field moisture observations of retrieved split spoon samples indicated the material was wet at the time of the investigation. One SPT 'N' value in this deposit was recorded at 32 blows per 30 cm of sampler advancement, indicating a dense material density.

5.2. Bedrock

Bedrock was cored in BH-26-01 and BH-26-02 and inferred in BH-26-03 and BH-26-04 based on auger and split spoon refusal. The bedrock conditions were consistent, with banded gneissic bedrock encountered at both borehole locations. The orthogneiss was generally red to pink with dark grey to light grey banding observed throughout the retrieved cores. The bedrock was fine to coarse-grained. A fragmented zone with trace sand filling and oxidation was observed in BH-26-02 from approximately 9.5 to 9.7 mbgs. The bedrock was fresh to faintly weathered with drill induced fractures throughout.

The calculated RQD for the retrieved rock cores ranged from 72% to 100%, indicating the observed rock mass was of fair to excellent quality. The calculated TCR ranged from 72% to 100%, and the calculated SCR ranged from 47% to 100% for the retrieved core runs.

Unconfined Compressive Strength (UCS) testing conducted on three (3) core specimens retrieved from site yielded results ranging from 64.6 to 85.2 MPa, with an average of 72.3 MPa. Based on the cumulative results of the compression testing completed on the tested core specimens, the bedrock is considered a high strength rock.

A summary of the bedrock encountered during the investigation is provided in Table 5-5 below. Detailed photographs of the retrieved rock cores can be found in Appendix F of this report.

Table 5-5: Summary of Bedrock Conditions

Borehole No.	Run #	Bedrock Lithology	Bedrock Depth		Bedrock			UCS (MPa)	End of Borehole	
			(mbgs) ¹	(masl) ²	RQD (%)	TCR (%)	SCR (%)		(mbgs) ¹	(masl) ²
BH-26-01	01	Orthogneiss	8.08	273.24	72	72	47		11.46	269.86
BH-26-01	02	Orthogneiss			100	100	93	64.6		
BH-26-01	03	Orthogneiss			100	100	100			
BH-26-02	01	Orthogneiss	9.09	272.01	77	100	72	67.0	12.14	268.96
BH-26-02	02	Orthogneiss			100	100	97	85.2		

Note(s): ¹Approximate metres below ground surface (mbgs) ² Elevation data has been approximated from the topographic survey completed by TULLOCH.

5.3. Groundwater Conditions

Groundwater level measurements were taken down open boreholes upon completion of the boreholes and after HW wash boring techniques were utilized in BH-26-01 and BH-26-02, and upon completion of the borehole in BH-26-03 and BH-26-04. The water levels measured upon borehole completion are presented below in Table 5-4, which provides a summary of the groundwater level measurements obtained. Given the sandy soils encountered on site and proximity to the creek, the water levels of the creek under the bridge are likely to govern the groundwater conditions locally for the site. Further, the organic soils encountered in BH-26-04 will likely be a conduit for groundwater infiltration and were observed to be saturated based on tactile examination.

It should be noted that the drilling investigation was conducted during winter conditions, and as such, lower water levels were likely present during the investigation than may be experienced during the wetter months in the spring or fall. Groundwater level is subject to seasonal fluctuations, with high levels occurring during wet weather conditions in the spring and fall and lower levels during dry weather conditions. As such, additional precautions should be taken for groundwater management if necessary.

Table 5-6: Groundwater Measurement Summary

Borehole Location	Water Depth (mbgs)	Soil Strata (mbgs)
BH-26-01	2.21	Fill – Gravelly Silty Sand

Borehole Location	Water Depth (mbgs)	Soil Strata (mbgs)
BH-26-02	1.81	Fill - Sand
BH-26-03	3.05	Silty Sand
BH-26-04	2.74	Silty Peat

5.4. Existing Bridge Abutments

Timber debris was recovered in a series of samples retrieved within BH-26-01 and BH-26-02 during borehole advancement. Based on the construction of the wooden abutments, it is possible that the boreholes conflicted with potential underground cribbing or timber tiebacks as part of the original bridge construction. The probable abutment conflict depth varied from approximately 2.3 to 4.9 mbgs in BH-26-01, and from approximately 2.3 to 3.0 mbgs in BH-26-02. It should be noted that these depths are estimated based upon minimal sample recovery observed from the retrieved split spoons at the time of the investigation. The material could also represent historic corduroy road building practices; however, due to the depths associated with the encountered timber material this is less likely. Further investigation during tendering is recommended via test pitting to help determine the source of this material and potential impact for bidders.

6. GEOTECHNICAL RECOMMENDATIONS

6.1. General

The existing Bay Street Bridge is a two-lane, single span bridge with a cast-in-place concrete deck supported on timber beams connected to a series of six (6) timber piles at each abutment. Review of the most recent available OSIM report in 2023 indicates that the timber piles are failing, and generally the bridge has reached the end of its service life. The report recommended the bridge for replacement. As such, the geotechnical recommendations in this report are tailored to full replacement of the structure.

The following sections of the report provide our interpretation of the available geotechnical data and provide geotechnical recommendations intended for the guidance of the design engineer. Where comments are made regarding construction, they are provided only to highlight any aspects that could affect the design of the project. Contractors bidding on or undertaking the construction should make their own interpretation of the provided subsurface information with respect to their planned construction methods, equipment selection, scheduling, and the like.

In addition, the following are to be noted:

- It is the Contractor's responsibility to manage the water in the existing river as well as the groundwater during construction.
- Cofferdam and temporary works are the responsibility of the contractor.
- The design and recommendations for a detour road (if required) during the replacement of the existing Bay Street Bridge are not addressed in this report.
- The final construction staging and heavy lifts on this site should be reviewed by a qualified engineer prior to executing the work to ensure that a bearing failure of the riverbank is avoided. If required, TULLOCH can provide assistance during construction to verify the geotechnical conditions and approve any design changes at the time of construction.
- Subsurface conditions discussed in this report pertain to the borehole locations; as such, due to the variable nature of soils, a geotechnical engineer from TULLOCH should be retained to review the subgrade conditions and assumptions made in this report during construction prior to foundation implementation. Upon review of subgrade conditions, if significantly different soil conditions are encountered, additional recommendations will be required and provided.

6.2. Geotechnical Design Considerations for Foundations

6.2.1. General

This report provides geotechnical foundation design recommendations in terms of Limit State Design, requiring foundation elements to withstand two (2) design conditions. The first is the factored geotechnical resistance of the foundation to withstand the imposed factored Ultimate Limit State loads – (ULS). The second is limiting the deformation/settlement of the foundation to acceptable levels under the Service loads – (SLS). At the time of preparing this report, details of the proposed structure, such as settlement tolerance and loading, were not available to TULLOCH. As such, it is assumed that 25 mm of total settlement and 19 mm differential settlement are acceptable.

The soil conditions at the Bay Street Bridge are generally poor, the presence of fine-grained saturated soils was encountered at typical foundation depths that would be expected for the bridge. Competent bearing soils were typically found either at or just above the bedrock interface encountered in the boreholes on site. As such, excavation and de-watering to the depths required for a suitable bearing layer for the installation of spread or strip style foundations is not considered

feasible for the structure. Furthermore, a suitable bearing layer within approximately 3.0m of the ground surface is not present on site, as such, shallow foundations are not considered feasible.

The presence of the bedrock, as confirmed in BH-26-01 and -02, with high strength rock and minimal weathering, makes this site to be feasible for deep foundations consisting of either driven piles to the bedrock surface, or alternatively, micropiles socketed into the competent rock, which will be discussed in further detail in the following sections of this report.

It should be noted that regardless of the option chosen, all pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection. The piling specifications should be reviewed by TULLOCH prior to tender. Contractor's submittals (design, shop drawing, methodology, load test procedures, set criteria, etc.) should be reviewed by TULLOCH prior to construction.

6.2.2. Driven Steel Piles

The abutments of the bridge can be founded on end bearing steel H-piles driven to practical refusal on the gneissic bedrock. The bedrock elevations encountered at BH-26-02 on the west side of the bridge and BH-26-01 on the east side of the bridge were El. 272.01 m and El. 269.86 m, respectively. However, bedrock in the general area of the site is likely undulating (as evidenced by the bedrock elevation difference between the boreholes on site, as well as the undulating nature of banded gneissic bedrock typical of the Magnetawan area), as such, there is a potential of deeper bedrock elevations and sloping bedrock conditions closer to the river alignment. There should be a provision made in the contract for dealing with varying pile lengths due to the potential variability in the bedrock surface elevation.

For HP310X110 piles driven into relatively flat, sound bedrock of fair to excellent quality with a minimum unconfined compressive strength of 60 MPa or greater (as indicated by the UCS breaks taken during the investigation), a factored geotechnical axial resistance at ULS of 1,500 kN may be used for design. This value represents the factored structural capacity of an undamaged pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since bedrock is considered to be an unyielding medium. As such, ULS conditions will govern the design.

Pile installation should be carried out in accordance with Ontario Provincial Standard Specification OPSS 903. As discussed in OPSS 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile. Generally, the piles should be fitted with flange reinforcement to minimize damage to the pile tip during driving (OPSD 3000.100) and suitable

toe points, such as Oslo points (OPSD 3000.201). For inclined/battered piles or piles on sloping bedrock surface, suitable driving points such as Titus Rock Injector Points or equivalent should be provided to ensure adequate seating of piles in/on the bedrock surface.

Even with suitable pile reinforcement measures, there is a risk of damage to piles or misalignment of piles driven to bedrock. Damaged or misaligned piles should be removed and new piles re-driven. Alternatively, the pile cap design needs to be flexible to allow for the installation of extra piles if necessary. A cost allowance should be carried in the contract for these items.

The required penetration resistance or the “set criteria” are dependent on hammer type and the selected pile. The set criteria can be established through a variety of methods such as empirical correlations and wave equation analyses prior to construction once the hammer and pile types are known. The criteria need to be set to also avoid overdriving and possibly damaging the pile on the bedrock surface.

Dynamic Testing (PDA Testing) should be performed near the beginning of the pile driving phase of construction to confirm the pile capacities through CAPWAP analysis. Dynamic Load Testing gives information on resistance distribution and evaluates the shape and integrity of the foundation. Piles that do not meet the design set criteria should receive additional restriking until the design set is met. All restriking should be carried out at least 24 hrs after the previous set has been achieved. TULLOCH recommends that dynamic monitoring and capacity testing be arranged (by the contractor) for about 10% of the production piles, or at least one (1) pile at each abutment, whichever is greater. Testing should be completed in conformity with ASTM D4945-00, titled "Standard Test Method for High Strain Dynamic Testing of Piles".

All piles in a group should be checked for heaving during driving of adjacent piles. Piles should be re-struck at least once in order to confirm the set and to check for upward displacement after adjacent piles have been driven. Piles that do not meet the design set criteria should receive additional restriking until the design set is met. All restriking should be carried out at least 24 hrs after the previous set has been achieved.

All piles should be inspected by TULLOCH personnel during installation to check for location, length, plumbness, initial set and restriking performance, internal damage, etc. Pile driving operations will cause vibrations that could potentially induce settlements of any nearby shallow foundations. An evaluation of any existing structures and a pre-construction condition survey should be carried out prior to pile driving operations.

6.2.3. Micropiles

An alternative to driven piles that could be considered for foundation construction on the site are micropiles socketed into the competent bedrock. Micropiles are small diameter (generally less than 300 mm) reinforced concrete drilled piles, the design of which relies on the grout-ground bond strength and neglects the contribution of end bearing. For preliminary design purposes of micropiles cased through the overburden and bonded within the strong gneissic bedrock at this site, a minimum grout-rock bond strength of 1,380 kPa (nominal/unfactored) can be used, as indicated by the lower bound limit for granitic and basaltic bedrock in the FHWA NHI-05-039 Micropile Design and Construction Reference Manual. However, the actual value will have to be confirmed by pre-production micropile load tests to be conducted at the site prior to the start of production piling. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since the bedrock is considered to be an unyielding medium. As such, ULS conditions will govern the design. Advantages of this foundation system include ease of installation in potential sloping bedrock, as well as lighter equipment for installation that does not require heavy pile drivers. However, typically, micropile foundations are more expensive to install and also require specialty design contractors.

The following should be noted for micropile design and construction:

- A specialty contractor will be required to design and build the micropiles.
- If micropiles are adopted, technical specifications should be developed to address minimum requirements for micropile load testing (pre-production verification tests and production proof tests), and submittals regarding materials, equipment, construction methodology and quality control.
- Monitoring of pile installation by a qualified engineer is recommended to confirm that the piles are installed in accordance with acceptable installation practices and design specifications.

6.2.4. Pile Settlement

The settlement of a single pile can be estimated using elastic theory to account for pile elastic compression on an unyielding bedrock. As discussed above, the geotechnical serviceability limit state of piles is not expected to govern the design of piled foundations at this site for piles seated on or socketed into the competent rock encountered during the investigation.

6.2.5. Resistance to Lateral Loads

Lateral Loading may be resisted partially or fully through the use of battered/inclined piles. In the case of battered driven piles, to minimize the risk of piles deflecting along the bedrock surface, it is recommended to restrict pile batter to as steep as possible for driven piles. For drilled micropiles, this is less of a concern and battered pile construction is anticipated to be feasible as long as good drilling practices are followed.

If vertical piles are required to provide lateral resistance, the horizontal reaction to the pile can be estimated using equations 6-1 and 6-2 shown below and ranges in subgrade reaction coefficient where:

$$k_s = n_h (z/d) \quad [6-1] \text{ for cohesionless soils}$$

$$k_s = (67 S_u)/d \quad [6-2] \text{ for cohesive soils,}$$

where:

d = pile width or diameter (m)

n_h = constant of horizontal subgrade reaction (MPa/m)

z = depth below ground surface grade (m)

S_u = undrained shear strength of the Soil (MPa)

Table 6-1 shown below contains the ranges of parameters that can be used. It is to be noted that subgrade reaction approaches are only an approximation of complex pile-ground interaction, and a range of subgrade reaction coefficients are typically used to assess the sensitivity of structural response to the applied loading.

Table 6-1: Horizontal Subgrade Reaction for Pile Lateral Loading

Soil Classification	S_u (MPa)	n_h (MPa/m)
Upper Loose Silty Sand to Sand	-	1-2
Silty Clay	0.03	-
Lower Compact to Dense Silty Sand to Gravelly Sand	-	9-12

TULLOCH can provide additional guidance if lateral subgrade reaction values are required for micropiles socketed into bedrock.

Group action for lateral loading should be considered when pile spacing in the direction of the loading is less than six (6) to eight (8) pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (k_s) in the direction of the loading by a reduction factor R, as shown below in Table 6-2.

Table 6-2: Pile Spacing vs. Reduction Factor

Pile Spacing in direction of Loading, d = Pile Diameter	Subgrade Reaction Reduction Factor R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

While calculating lateral resistance, any portion of the pile within the frost depth, unless accounted for by the pile cap, should be excluded from contributing to the lateral capacity of the pile.

7. OTHER DESIGN AND CONSTRUCTION CONSIDERATIONS

7.1. Abutment Backfill

Backfill for abutments should consist of free-draining, non-frost susceptible granular materials, such as OPSS 1010 Granular B Type I, Type II or an approved equivalent. The existing road base/Sub-base fill materials were generally found to be outside of the OPSS 1010 Granular B Type 1 gradation envelope due to excessive fines. As such, it is not recommended for re-use. Furthermore, the silty peat layer encountered in BH-26-04 indicates historic stripping and road fill placement may have been unsuitable. As such, TULLOCH would recommend full depth replacement for the abutments and approaches to the limit of the proposed construction.

The native soils are unlikely to be suitable backfill materials due to their high silt/fines content and resulting frost susceptibility. For abutment backfilling, the native soils and existing fill materials should be removed behind the abutment walls within a wedge-shaped zone extending from 1.2 m behind the base of the abutments and rising upward at an inclination of 1.0 vertical to 1.5 horizontal, according to OPSD 3101.150. A suitable frost taper is required for imported abutment backfill materials within the frost zone (as shown in OPSD 3101.150).

During excavation there is also the possibility of conflict with historic timbers/potential corduroy road and organics based on the geotechnical investigation results. Any debris, organics or otherwise deleterious material should be removed when encountered to the extent possible.

All granular backfill materials should be placed in thin lifts, not to exceed 300 mm in uncompacted thickness. Lifts should be compacted to a minimum of 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). For backfilling beneath the pavement area, the degree of compaction of the fill materials within 1.5 m of pavement subgrade should be increased to 98% SPMDD. The additional compaction effort will lead to better pavement performance over time. The Granular A base and Granular B sub-base courses below pavement (OPSS 1010) should be compacted to not less than 100% of the material's SPMDD.

The use of heavy compaction equipment should be avoided immediately adjacent to abutments. At no time should the fill levels on either side of the structure differ by more than 300 mm to avoid applying unbalanced loads on the structure, which could cause lateral displacement (dislodging) and/or damage.

Further, the poor soils on-site may be prone to disturbance; as such, compaction of abutment fills should be carefully monitored by a qualified geotechnical engineer or their representative, and heavy vibration should cease if disturbance is noticed. Backfill may then continue via static rolling, and or lifts could be thickened at the request of the geotechnical engineer until a sufficient pad has been built so that vibratory compaction may continue.

7.2. Lateral Earth Pressure

The lateral earth pressures acting against abutments and wing walls should be calculated according to the Canadian Highway Bridge Design Code (CHBDC 2014 CAN/CSA-S6-14). The general equation for calculating lateral earth pressure is:

$$P = k (\gamma h + q)$$

where

P = lateral earth pressure in kPa acting at depth;

k = earth pressure coefficient;

γ = bulk unit weight in kN/m³;

h = depth to point of interest in m;

q = surcharge near wall in kPa.

Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Unless drainage provisions are made or remain functional over the design life of the structure, hydrostatic pressure will have to be added to the lateral pressure. In accordance with

Clause 6.12.3 of the CHBDC (S6-14), a minimum earth pressure of 12 kPa should be assumed to act on abutments, culvert walls and wing walls at the surface to account for compaction induced earth pressures. The properties listed in Table 7-1 can be used during the design for the backfill materials.

Table 7-1: Recommended Lateral Earth Pressure Parameters

Material	Unit Weight (kN/m ³)	Effective Friction Angle (°)	Coefficient of Lateral Earth Pressure ¹	
			K' _a	K' _o
Granular A	22	38	0.24	0.38
Granular B, Type I	21	34	0.28	0.44
Granular B, Type II	21	36	0.26	0.41

Note(s): ¹ K'_a = coefficient of active earth pressure on a laterally unrestrained (non-rigid) structure; K'_o = coefficient of earth pressure at rest for soil loading on a laterally restrained structure.

The above earth pressure coefficients assume a horizontal backfill condition, vertical back-face of the retaining structure and smooth soil-wall interface only. If the design includes a sloping ground surface in front of the retaining structure, a backfill inclination, or an inclined back-face of the retaining wall, the earth-pressure coefficients will require modification. TULLOCH should be contacted to provide appropriate coefficients for these conditions.

7.3. Frost Protection

The estimated frost penetration depth at the Site is 1.8 m as per OPSD 3090.101. If the minimum soil cover cannot be achieved, equivalent insulation should be used, as the on-site native soils are considered highly frost-susceptible due to high fines content and saturated/wet conditions. Frost tapers should be implemented for backfilling behind the bridge abutment or any trenches underlying areas (e.g., pavements) where differential frost heaving will not be acceptable, as recommended in the applicable OPSD (e.g., OPSD 3101.150).

7.3.1. Adfreeze Stress

For steel pile foundations exposed to frost action, an adfreeze uplift stress of 100 kPa (unfactored) recommended by the Canadian Foundation Engineering Manual (CFEM) (2023) should be used in the pile design for the soil-pile contact area within the design frost depth of 1.8 m. If granular soils are used as backfill materials against the pile/foundation pole below the ground surface, then adfreeze adhesion values can be as high as 150 kPa, as discussed in the CFEM.

7.4. Erosion Protection

The native soils encountered on site are highly erodible and scour prone due to their fine-grained nature. Erosion and scour protection should be provided along the slopes and sides of the bridge abutments. Consideration should be given to appropriately sized rip-rap on the road embankments near the bridge abutments within the flood plain of the river as determined by hydraulic analysis that is considered outside the scope of this report. Alternatively, if sheet piling is installed, it may be left in place to act as scour protection after the construction is complete. The erosion/scour protection should be designed by a hydraulic engineer.

7.5. Excavations and Groundwater Control

Trench excavations, if required, must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA), Regulation 213/91, as well as the specifications of OPSS 539 and OPSS 902. In accordance with OHSA, the native soils at the site can be classified as Type 4 material below the groundwater level. As such, excavations must be sloped at 3H:1V from the base of excavation unless the materials are dewatered or supported. If dewatered, the soils should behave as Type 3 material and trench excavations can be sloped at 1H:1V from the base of the excavation. Any organic soils encountered during abutment excavation should be treated as Type 4 soils.

Excavations that extend below the groundwater table will require control measures. Medium to low permeability soils exist throughout all borehole locations, with soils within the excavation predominantly consisting of wet silty sands. Sump and pump techniques are anticipated to be enough to de-water shallow excavations that do not extend significantly below the groundwater table or river level, i.e., less than approximately 0.5 m. However significantly extending into the groundwater table will likely warrant active de-watering, such as well points or other active groundwater control techniques, or a groundwater cut-off (e.g., sheet pile cofferdam) system may need to be installed to dewater the site if deeper excavations below the water table are required. The actual dewatering methods should be established at the contractor's discretion within the context of a performance specification for the project. It should be noted that, under the Ontario Water Resources Act, the Water Taking and Transfer Regulation 387/04, for the purpose of construction site dewatering, an Environmental Activity Sector Registry (EASR) registration is required if dewatering discharge is greater than 50,000 L/day but less than or equal to 400,000 L/day. If water taking is greater than 400,000 L/day a Permit to Take Water (PTTW) is required from the Ministry of the Environment, Conservation and Parks. Reference is also given to OPSS 517 and 518 which pertain to construction dewatering. At this site, an EASR registration will be

required if active de-watering is implemented, and a PTTW may need to be pursued depending on the groundwater control method selected. A hydrogeological study will likely have to be completed to assess dewatering volume, zone of influence and impact on any adjacent properties.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during normal dry weather conditions.

The excavation of the soils at the site should be achievable using conventional excavating equipment. The presence of cobbles and boulders within the native soils cannot be ruled out, given the glacial origin of the soils.

7.6. Shoring/Support Systems for Excavation

Shoring/Sheet piling may be an effective way to create hydraulic isolation and support of excavations for abutments/pile caps. For general guidance, the design of the shoring system may be completed using the apparent earth pressure diagrams for braced excavations in granular soils, while clay banding was encountered in BH-26-02, the predominant soils encountered were non-cohesive silts and sand. Pressure diagrams for granular soils are discussed in Section 20.8.1.4 of the CFEM (5th Edition, 2023) shown below in Figure 7-1. As the excavation is expected to extend through sandy fill into the predominantly silty sand native soil, a bulk unit weight of soils of 18 kN/m^3 and an active earth pressure coefficient of 0.36 may be used. A uniform surcharge pressure should be added to the earth pressure diagram to account for construction surcharge loads adjacent to the excavations. A surcharge of 12 kPa is typically used for light construction loads; heavier loads can be assessed individually using elastic stress distribution solutions.

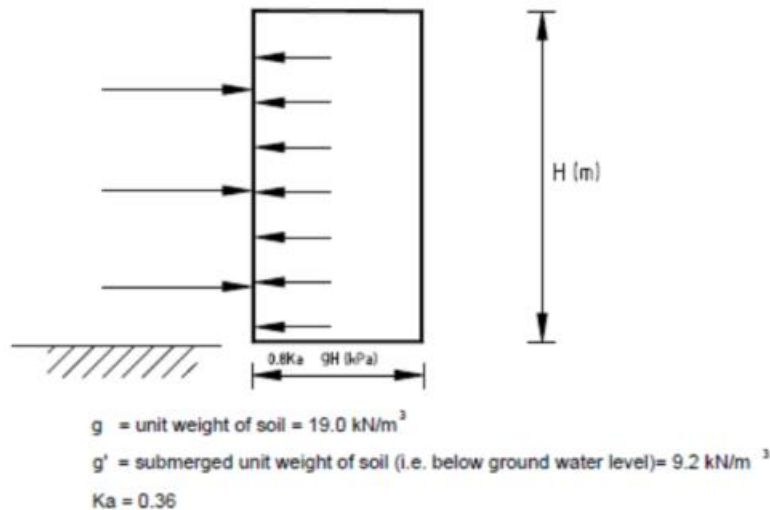


Figure 7-1: Pressure Diagram for Loose to Compact Non-Cohesive Soils (Silty Sand to Silt)

7.7. Reuse of Excavated Soil

Further to the discussion provided in Section 7.1, the existing fill materials are not recommended for re-use given their high fines content. Further, the presence of organic soils below the pavement and road fills are also not considered a suitable subgrade for road fill materials. Given the high fines content and failure of the existing fill to meet the OPSS 1010 Granular B Type 1 gradation window, the fill materials are not recommended for re-use. Furthermore, the native soils on site, due to their fine-grained nature, are also not considered feasible for use as engineered fill. It should be noted that all soil disposal should be done in accordance with all applicable regulations including O.Reg 406/19. Chemical soil testing and disposal recommendations are considered outside the scope of this document.

7.8. Pavement Design

The following represents the minimum pavement structure design recommendation, as TULLOCH understands, that there is no available traffic data or any published traffic studies for the project site. As such, pavement design has been conducted with an assumed daily traffic rate in accordance with the Routine (Empirical) Method – Experience-Based Standard Section design method as presented in the Pavement Design and Rehabilitation Manual (PDRM) (MTO 2013) for a pavement structure located in the northeastern region of Ontario with granular fill depths not less than those given for an Annual Average Daily Traffic (AADT) volume ranging from 1000 to 1500. The pavement structure design has been conducted to provide a Granular Base Equivalency (GBE) in accordance with the PDRM and previous experience for similar roadway structures. The designer should confirm the traffic assumptions.

The existing road pavement structure should be reinstated in a like for like fashion. New granular must match into the underside of existing to ensure unimpeded cross drainage.

The following minimum flexible pavement structure is recommended based on the site's geotechnical conditions and assumed traffic data; if the existing conditions are found to be greater than the minimum thicknesses presented below, the existing conditions will govern design:

- Asphalt concrete should not be less than 50 mm thick, consisting of HL-3
- Base Course: OPSS Form 1010 Granular A, 150 mm thick
- Subbase: OPSS Form 1010 Granular B Type II, Granular B Type I or approved equivalent, minimum 300 mm thick

It should be noted that the organic layers encountered in the investigation below the asphalt and road fills in BH-26-03 and BH-26-04 should be removed prior to upfilling and placement of road fills where encountered.

For the road construction, remove all organics, loose granular fill, and other unsuitable material down to a suitable subgrade level, as verified by a geotechnical engineer, or their representative. Following removals, the exposed subgrade should be proof rolled, applicable to within 95% of the material SPMDD using the control strip method. If over-excavation is required, OPSS Select Subgrade Material (SSM) or selected approved native materials can be used as backfill to bring the subgrade up to the design elevation. The final exposed subgrade should be crowned at a minimum of 2% cross fall gradient towards the edge of the pavement. The pavement subgrade should be inspected and approved by a qualified geotechnical engineer prior to placement of the pavement structure.

The subbase material, Granular B Type I or Granular B Type II, and base course material, Granular A, should be placed in lifts not to exceed 300 mm and compacted to a minimum of 100% SPMDD. The surface of the pavement must be crowned at a cross fall of 2% towards the drainage facilities of the pavement.

7.9. Approach Reinstatement

Backfill to the abutments should be completed to the underside of the pavement structure, as recommended in the previous sections. The reinstated pavement structure of the bridge approaches should match the existing adjacent pavement structure at a minimum and comply

with the current engineering standards for The Municipality of Magnetawan and/or applicable provincial standards.

Additionally, periodic maintenance for the road surface may be required at the approaches to account for any differential settlement of the abutment backfill and the new bridge. Consideration should be given to applying only the base course asphalt layer or operating the crossing as a gravelled surface for approximately one (1) year after construction to allow initial settlement to occur. Ideally, the approach settlement should be monitored to confirm a suitable duration prior to placing the final asphalt layers.

7.10. Seismic Design Considerations

The 2024 Ontario Building Code (OBC) stipulates the methodology for earthquake design analysis. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration, and the site classification for seismic site response.

The parameters for the determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.B in the OBC. The site classification is based on the average shear wave velocity in the top 30 meters of the site stratigraphy. If the average shear wave velocity is not known, the site class can be estimated from energy corrected Standard Penetration Resistance (N60) and/or the average undrained shear strength of the soil in the top 30 meters.

Based on the 2024 OBC, this site has been classified as a “Class “B” – “Rock” for deep foundations placed directly on the encountered bedrock. These seismic design parameters should be reviewed in detail by the structural engineer and incorporated into the design as required by the 2024 OBC or other governing codes.

7.11. Soil Corrosivity

Representative testing was completed for soil corrosivity and sulphate concentrations for the bridge location. The results of the testing are shown below in Table 7-2. Samples were tested at TestMark laboratories based in Garson, Ontario.

Table 7-2: Soil Corrosivity Results

Borehole No./Sample No.	Depth (m)	Resistivity (Ω cm)	pH	Redox Potential (mV)	Sulphide (μ g/g)	Sulphate (μ g/g)	Chloride (μ g/g)
BH-26-01 SS02	1.5	6490	4.65	348	<0.4 ¹	33	30.1
BH-26-02 SS03	3.0	4520	6.88	367	<0.3 ¹	26	57.6

Notes:¹Test detection limit

The results of the chemical testing were assessed in reference to the AWWA C-105 Standard from ANSI/AWWA Corrosivity Rating System. A score greater than 10 indicates the requirement of corrosion protective measures for buried cast iron alloys. All the tested samples analyzed for BH-26-01 and BH-26-02 scored a ranking of 1 and 2, respectively, which is below the threshold.

In addition, chloride ions can lead to corrosion of steel. Typically, soils with chloride concentrations greater than 500 μ g/g are considered corrosive. As noted in the table, chloride concentrations are less than 500 μ g/g in the tested samples.

The concentration of sulphate indicates the degree of sulphate attack for concrete buried at the site. As shown in the table, the sulphate concentrations are less than 1000 μ g/g indicating a low degree of sulphate attack. Type GU Portland Cement should be suitable for use at this site. The test results presented in Table 7-2 may be used to aid in the selection of any coatings and corrosion protection systems for buried steel infrastructure. For detailed chemical results please see Appendix E, Laboratory Data.

8. CLOSURE

This geotechnical report has been prepared by TULLOCH for the exclusive use of The Municipality of Magnetawan and their authorized agents for the replacement of the Bay Street Bridge on Bay Street in the Municipality of Magnetawan, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering for the above-noted location. Classification and identification of soils and geologic units have been based upon commonly accepted methods

employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Please refer to Appendix G, Notice to Reader, which pertains to this report.

We trust that the information in this report will be sufficient to proceed with the project. Should further elaboration be required for any portion of this project, we would be pleased to assist.

A handwritten signature in black ink, appearing to read 'Erik Giles'.

Prepared By:
Erik Giles, P.Eng.
Geotechnical Engineer

A handwritten signature in black ink, appearing to read 'George Liang'.

Reviewed By:
George Liang, P.Eng., PhD
Geotechnical Engineer

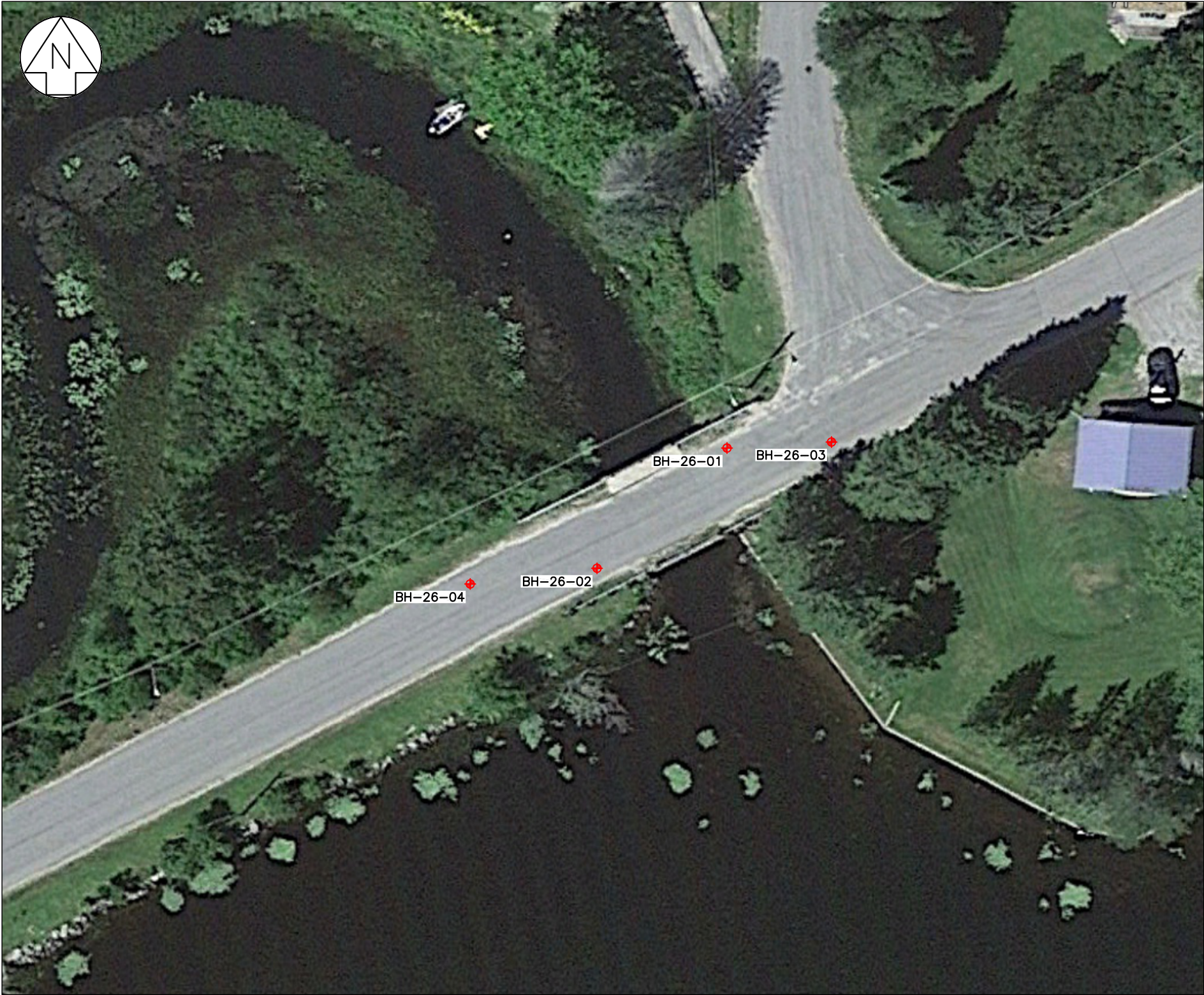
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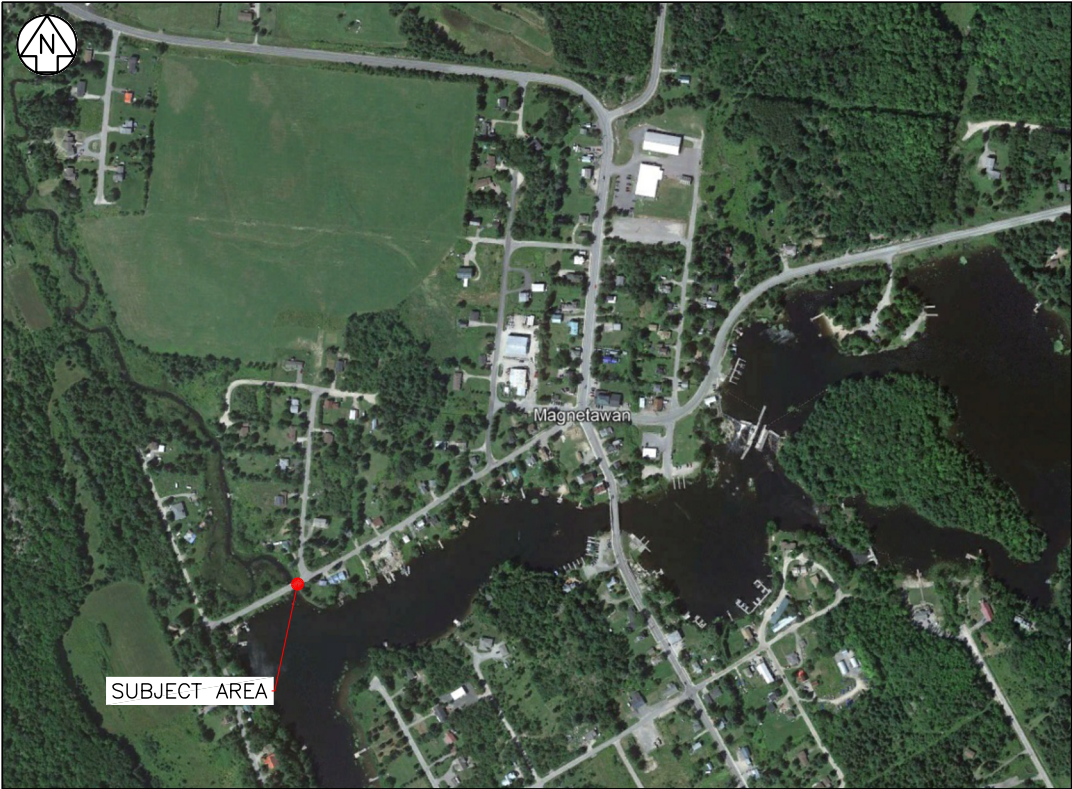
APPENDIX A

SITE LOCATION PLAN

P:\2026\260649\05 Geotechnical\03 Drawings\03.02 Current\Borehole Siteplan\26-0649-001.dwg



BOREHOLE LOCATION PLAN



PROJECT LOCATION

COORDINATES			
BOREHOLE ID	EASTING	NORTHING	ELEVATION (m)
BH-26-01	605 257	5 057 511	281.32
BH-26-02	605 247	5 057 500	281.10
BH-26-03	605 266	5 057 510	281.28
BH-26-04	605 235	5 057 500	281.10

NOTES:

1. COORDINATES ARE IN UTM ZONE 17T (NAD83 CSRS).

LEGEND:

BH-26-01  BOREHOLE LOCATION

0	2026-04-02	LM	ISSUED FOR REPORT
No.	DATE	BY	ISSUES / REVISIONS



DRAWING:

BOREHOLE
LOCATION PLAN

PROJECT:

**BAY STREET BRIDGE REPLACEMENT
GEOTECHNICAL INVESTIGATION**

DRAWN BY: L. MENEGHETTI	CHECKED BY: E. MIGHTON	PROJECT No. : 26-0649	
	APPROVED BY: E. GILES	DRAWING No. 26-0649-001	REVISION No. 0
SCALE: AS NOTED	DATE: 2026-04-02		

APPENDIX B

TERMINOLOGY

ABBREVIATIONS, TERMINOLOGY AND PRINCIPAL SYMBOLS USED IN REPORT AND BOREHOLE LOGS

BOREHOLES AND TEST PIT LOGS

Soils

AS	Auger/Grab Sample	w	Water Content
SS	Split Spoon	wP	Plastic Limit
SH	Shelby Tube	wL	Liquid Limit
PISTON	Thin-walled Piston	VANE	Field Vane
WS	Washed Sample	OR	Organic Content
SC	Soil Core	GR	Gravel
BS	Block Sample	SA	Sand
WH	Weight of Rods & Hammer	SI	Silt
WR	Weight of Rods	CL	Clay

Bedrock

TCR	Total Core Recover	VN	Vein
SCR	Solid Core Recovery	CO	Contact
FI	Fracture Frequency Index	KV	Karstic Void
HQ	Rock Core (63.5 mm dia.)	MB	Mechanical Break
NQ	Rock Core (47.6 mm dia.)	PL	Planar
BQ	Rock Core (36.5 mm dia.)	CU	Curved
JN	Joint	UN	Undulating
FLT	Fault	IR	Irregular
SH	Shear	SM	Smooth
SK	Slickensided	SR	Slightly Rough
BD	Bedding	R	Rough
FO	Foliation	VR	Very rough

IN SITU SOIL TESTING

Standard Penetration Test (SPT) "N" value is the number of blows required to drive a 51 mm OD split barrel sampler into the soil a distance of 300 mm with a 63.5kg weight free falling a distance of 760 mm after an initial penetration of 150 mm has been achieved.

Dynamic Cone Penetration Test (DCPT) is the number of blows required to drive a cone with a 60-degree apex attached to "A" size drill rods continuously into the soil for each 300 mm penetration with a 63.5 kg weight free falling a distance of 760 mm.

Cone Penetration Test (CPT) is an electronic cone point with a 10 cm base area with a 60-degree apex pushed through the soil at a penetration rate of 2cm/s.

Field Vane Test (FVT) consists of a vane blade, a set of rods and torque measuring apparatus used to determine the undrained shear strength of cohesive soils.

SOIL DESCRIPTIONS

The soil descriptions and classifications are based on an expanded Unified Soil Classification System (USCS). The USCS classifies soils on the basis of engineering properties. The system divides soils into three major categories: coarse grained, fine grained and highly organic soils. The soil is then subdivided based on either gradation or plasticity characteristics. The classification excludes particles larger than 75 mm. To aid in quantifying material amounts by weight within the respective grain size fractions, the following terms have been included to expand the USCS:

Soil Classification		Terminology	Proportion
Clay	<0.002 mm	"trace", sand, etc.	1% to 10%
Silt	0.002 to 0.06 mm	"some"	10% to 20%
Sand	0.075 to 4.75 mm	Sandy, Gravelly, etc.	20% to 35%
Gravel	4.751 to 75 mm	"and" SAND, SILT, (non-cohesive)	>35%
Cobbles	75 to 300 mm	"with" SAND, SILT, (cohesive)	>35%
Boulders	>300 mm		

Notes:

1. Soil properties, such as strength, gradation, plasticity, structure, etc., dictate the soils engineering behaviour over the grain size fractions;
2. With the exception of soil samples tested for grain size distribution or plasticity, all soil sample classifications are based on visual and tactile observations and, therefore, constitute an approximate description.

The following table outlines the qualitative terms used to describe the relative density condition of cohesionless soils related to the SPT "N" value:

Cohesionless Soils

Compactness	SPT "N" Value (blows/30cm)
Very Loose	0 to 4
Loose	5 to 10
Compact	11 to 30
Dense	31 to 50
Very Dense	>50

The following table outlines the qualitative terms used to describe the consistency of cohesive soils related to undrained shear strength and SPT "N" value:

Cohesive Soils

Consistency	Undrained Shear Strength (kPa)	SPT "N" Value (blows/30 cm)
Very Soft	<12.5	< 2
Soft	12.5 to 25	2 to 4
Firm	25 to 50	5 to 8
Stiff	50 to 100	9 to 15
Very Stiff	100 to 200	16 to 30
Hard	> 200	>30

Note: Utilizing the SPT "N" value to correlate the consistency and undrained shear strength of cohesive soils is very approximate and needs to be used with caution.

Particle Sizes

Constituent	Description	Size (mm)	Size (in)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by Plasticity	< 0.075	< (200)

Note: Brackets () indicate US Standard Sieve Size Number

ROCK CORING

Rock Quality Designation (RQD) is an indirect measure of the number of fractures within a rock mass, Deere et al. (1967). It is the sum of sound pieces of rock core equal to or greater than 100 mm recovered from the core run, divided by the total length of the core run, expressed as a percentage. If the core section is broken during coring or handling, the pieces are fitted together and, if 100 mm or greater included in the total sum.

Intact Rock Strength

Intact Strength (MPa)	Description
< 1	Extremely low strength
1 to 5	Very low strength
5 to 25	Low strength
25 to 50	Medium strength
50 to 100	High strength
100 to 250	Very high strength
>250	Extremely high strength

Rock Mass Quality

RQD Classification	RQD Value (%)
Very Poor Quality	<25
Poor Quality	25 to 50
Fair Quality	50 to 75
Good Quality	75 to 90
Excellent Quality	90 to 100

Rock Mass Weathering

Term	Description
Unweathered (Fresh)	No visible sign of material weathering and slight discoloration on major discontinuity surfaces.
Slightly Weathered	Discoloration indicates the weathering of rock material and discontinuity of surfaces. All of the rock material may be discoloured by weathering and may be somewhat weaker than its fresh condition.
Moderately Weathered	Less than half the rock material is decomposed and/or disintegrates to soil. Fresh or discoloured rock is present either as a continuous framework of as core stones.
Highly Weathered	More than half the rock material is decomposed and/or disintegrated to soil. Fresh or discoloured rock is present either as a discontinuous framework or as core stones.
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely intact.
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

Joint and Foliation Spacing

Description	Spacing
Very Wide	Greater than 3 m
Wide	1 m to 3 m
Moderately Close	0.3 m to 1 m
Close	50 mm to 300 mm
Very Close	Less than 50 mm

Bedding Thickness

Description	Spacing
Very thick	Greater than 2 m
Thick	0.6 m to 2 m
Medium	0.2 m to 0.6 m
Thin	60 mm to 0.2 m
Very thin	20 mm to 60 mm
Laminated	6 to 20 mm
Thinly Laminated	Less than 6 mm

SYMBOLS

General

w_N	Natural water content within the soil sample
γ	Unit weight
γ'	Effective unit weight
γ_D	Dry unit weight
γ_{SAT}	Saturated unit weight
ρ	Density
ρ_s	Density of solid particles
ρ_w	Density of water
ρ_D	Dry density
ρ_{SAT}	Saturated density
e	Void ratio
n	Porosity
S	Degree of saturation
E_{50}	Fifty percent secant modulus

Consistency

w_L	Liquid Limit
w_P	Plastic Limit
I_P	Plasticity Index
w_S	Shrinkage Limit
I_L	Liquidity Index
I_C	Consistency Index
e_{max}	Void ratio in loosest state
e_{min}	Void ratio in densest state
I_D	Density Index (formerly relative density)

Shear Strength

S_u	Undrained shear strength parameter (total stress)
c'	Effective cohesion intercept
ϕ'	Effective friction angle
τ_P	Peak shear strength
τ_R	Residual shear strength
δ	Angle of interface friction
μ	Coefficient of friction = $\tan \phi'$

Consolidation

C_c	Compression index (normally consolidated range)
C_r	Recompression index (over consolidated range)
m_v	Coefficient of volume change
c_v	Coefficient of consolidation
T_v	Time factor (vertical direction)
U	Degree of consolidation
σ'_v	Effective overburden pressure
OCR	Overconsolidation ratio

APPENDIX C

PHOTOGRAPH LOG



Photo 1: Existing Bridge – North Side.



Photo 2: Existing Bridge – South Side.

CLIENT

The Municipality of Magnetawan

PROJECT

Bay Street Bridge Replacement – Geotechnical Investigation

CONSULTANT



YYYY-MM-DD 2026-03-18

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LM

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EM

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TITLE

Geotechnical Investigation - Site Photographs

PROJECT NO.
26-0649

Phase/Task
104

Rev.
0

FIGURE
1



Photo 3: Existing Bridge – East Abutment.



Photo 4: Existing Bridge – West Abutment.

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Phase/Task
104

Rev.
0

FIGURE
2



Photo 5: BH-26-01 during advancement. Photo taken facing northeast.



Photo 6: BH-26-01 backfilled. Photo taken facing east.

CLIENT

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PROJECT

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FIGURE
3



Photo 7: BH-26-02 during advancement. Photo taken facing northeast.



Photo 8: BH-26-02 backfilled. Photo taken facing southwest.

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104

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0

FIGURE
4

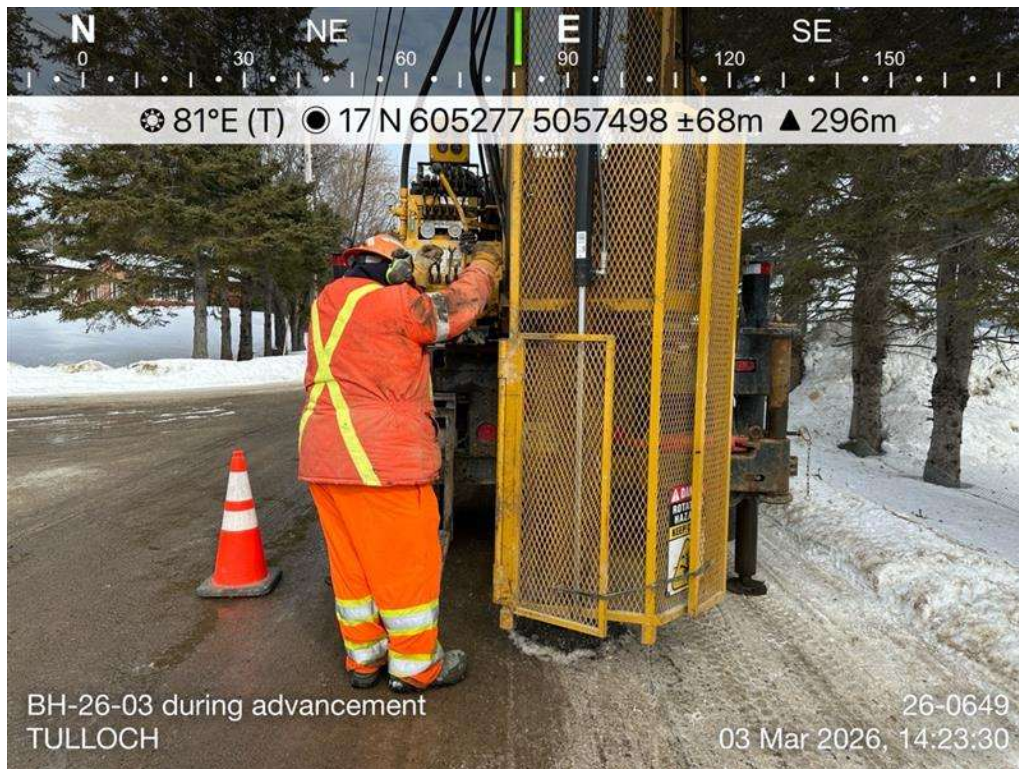


Photo 9: BH-26-03 during advancement. Photo taken facing east.



Photo 10: BH-26-03 backfilled. Photo taken facing northeast.

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PROJECT

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TITLE

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PROJECT NO.
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Phase/Task
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FIGURE
5

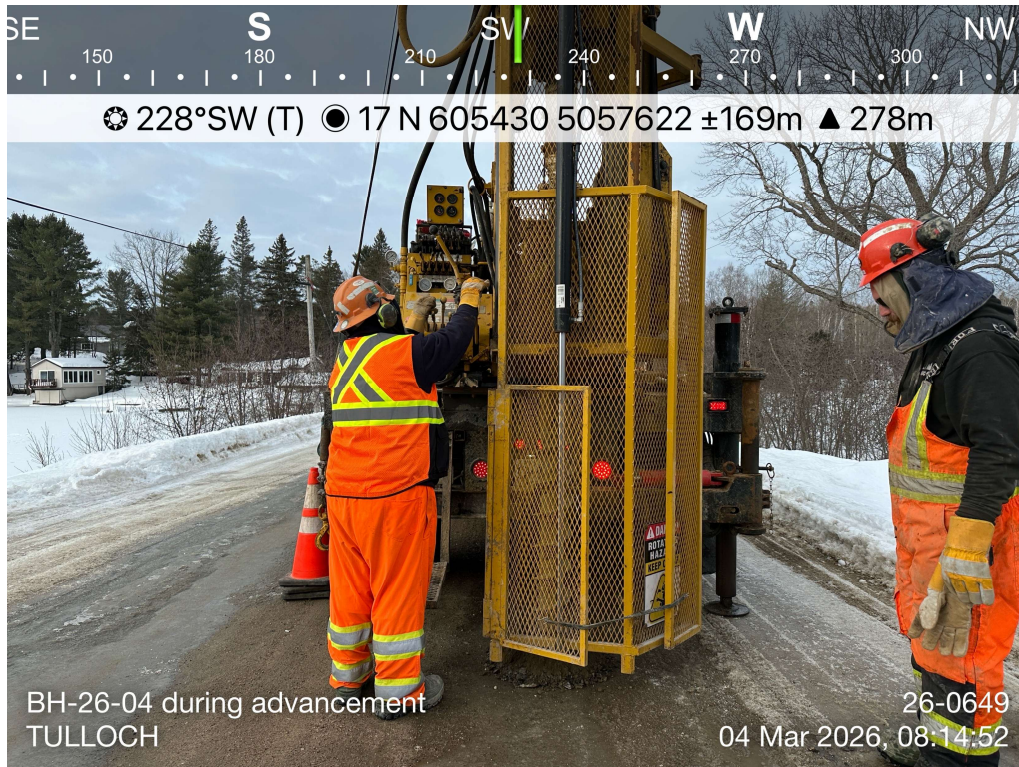


Photo 11 BH-26-04 during advancement. Photo taken facing southwest.



Photo 12: BH-26-04 backfilled. Photo taken facing southwest.

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PROJECT NO.
26-0649

Phase/Task
104

Rev.
0

FIGURE
6

APPENDIX D

BOREHOLE LOGS

JOB NUMBER 26-0649 LOCATION Bay Street Bridge, Magnetawan, ON ORIGINATED BY LM
 CLIENT Municipality of Magnetawan DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger/HW COMPILED BY EM
 DRILLER Landcore Drilling DATE 2026.03.02 NORTHING 5057511 EASTING 605257 CHECKED BY EG

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION (M)	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)			SHEAR STRENGTH kPa													
									○ POCKET PEN	+	FIELD VANE	● QUICK TRIAXIAL	×					LAB VANE	WATER CONTENT (%)			
							20	40	60	80	100								GR	SA	SI	CL
281.32								▽	281									Auger grinding during advancement from approx. 1.52 to 2.29 mbgs				
280.00	ASPHALT - 25 mm thick		01	AS			280												24	61	(15)	
	FILL - (SM) Gravelly SILTY SAND, fine to coarse grained, brown, non-cohesive, frozen to moist, compact to very dense		01	SS	128	63																
			02	SS	27	67																
279.03			03	SS	89/ 3"	38			279													
2.29	BRIDGE ABUTMENT - historic timber fragments with medium to coarse grained sand. Timber pieces appear weathered and consistent with remnants of the historic bridge abutment.		04	SS	20	13			278													
									277													
276.44			05A	SS	32	46			276													
4.88	(SM) SILTY SAND, fine grained, trace coarse grained gravel, non-plastic, brown, with oxidation, non-cohesive, wet, dense		05B	SS	32	46			275													
275.68									274													
5.64	(SP) SAND, fine to coarse grained, trace non-plastic fines, trace fine grained gravel, brown, non-cohesive, wet, compact		06	SS	27	21											1	95	(5)			
273.45			07	SS	102/ 4"	100																
273.24	COBBLES/BOULDERS																					
8.08	END OF SOIL BOREHOLE LOG - SEE RECORD OF ROCKCORE No. BH-26-01. Note(s): - Cave in encountered at approx. 4.40 mbgs upon completion of rock coring and removal of casing. - Groundwater mesaured at approx. 2.21 mbgs upon completion of rock coring and removal of casing. - It should be noted that groundwater conditions may not have stabilized upon completion of drilling.																					

3. SOIL REPORT+REC. (ELEVATION) 26-0649 BH LOGS SOIL.GPJ ONTARIO MTO.GDT 26-4-2

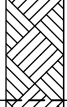
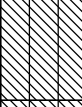
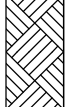


RECORD OF ROCKCORE No BH-26-01

1 OF 1

METRIC

JOB NUMBER 26-0649 LOCATION Bay Street Bridge, Magnetawan, ON ORIGINATED BY LM
CLIENT Municipality of Magnetawan DATUM Geodetic BOREHOLE TYPE HW/NQ COMPILED BY EM
DRILLER Landcore Drilling DATE 2026.03.02 NORTHING 5057511 EASTING 605257 CHECKED BY EG

Elev. Depth	Strat Plot	Description	Depth (m)	Ground Water Conditions	Sample No.	Recovery Parameters	RQD % 	Fracture index / ft	Discontinuity Data		Compressive Strength (MPa)	Remarks
									Dip Angles W.R.T Core Axis	Type and Description (see core log photos)		
281.3 0.00		Soil Overburden: Refer to RECORD OF BOREHOLE No. BH-26-01	1 2 3 4 5 6 7				25 50 75		30 60		50 100 150	
273.24 8.08		ORTHOgneiss - Metamorphic Rock, fine to coarse grained, red to pink, light to dark grey banding, drill induced fractures throughout, faintly weathered rock, strong rock, fair quality based on RQD	8 9		01	TCR= 72% SCR= 47% RQD= 72%		1 5				
272.02 9.30		ORTHOgneiss - same as above, fresh rock, excellent quality based on RQD	10		02	TCR= 100% SCR= 93% RQD= 100%					64.6	
270.50 10.82		ORTHOgneiss - same as above, fresh rock, excellent quality based on RQD	11		03	TCR= 100% SCR= 100% RQD= 100%						
269.86 11.46		END OF BOREHOLE - TARGET DEPTH ACHIEVED Note(s): - Cave in encountered at approx. 4.40 mbgs upon completion of drilling. - Groundwater measured at approx. 2.21 mbgs upon completion of drilling. - It should be noted that groundwater conditions may not have stabilized upon completion of drilling.										



Filling			Surface		Type		Aperture		Weathering	
Clean - CL	Clay - Cy	PL	Planar	BD	Bedding	Tight: 0.1 - 0.5mm (T)		FR	Fresh	
Iron Stained - Fe	Calcite - Cal	CU	Curved	JN	Joint	Moderately Open:		SW	Slightly	
Manganese Stained - Mn	Hematite - Hem	UN	Undulating	FLT	Fault	0.5-2.5mm (MO)		MW	Moderately	
Carbonate - C	Pyrite - Py	IR	Irregular	SH	Shear	Open: 2.5-10mm (O)		HW	Highly	
Gypsum - G	Serpentine - St			KV	Karstic Void	Very Open: > 10mm (VO)		EW	Extremely	
Silty/Clay - SC	Chlorite - Ch									

JOB NUMBER 26-0649 LOCATION Bay Street Bridge, Magnetawan, ON ORIGINATED BY LM
 CLIENT Municipality of Magnetawan DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger/HW COMPILED BY EM
 DRILLER Landcore Drilling DATE 2026.03.03 NORTHING 5057500 EASTING 605247 CHECKED BY EG

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION (M)	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)			SHEAR STRENGTH kPa								
281.10									20	40	60	80	100				
280.00	ASPHALT - 25 mm thick		01	AS													
	FILL - (SP-SW) SAND, fine to coarse grained, trace fine grained gravel, trace non-plastic fines, brown, non-cohesive, moist, compact		01	SS	19	46											
278.81																	
2.29	BRIDGE ABUTMENT - historic timber fragments with medium to coarse grained sand. Timber pieces appear weathered and consistent with remanants of the historic bridge abutment.		02	SS	6	0											SS02: Readvanced with no recovery
278.13																	
2.97	(SM) SILTY SAND, fine grained, non-plastic, dark brown, with wood debris, non-cohesive, wet, loose		03	SS	4	42											SS03: Readvanced
276.99																	
4.11	(CL) SILTY CLAY, low plasticity, grey, cohesive, w>PL, very soft to firm		04	SS	WH	21											Begin casing advancement with washboring method
			01	VANE													
275.00																	
6.10	(ML/SP) SILT and SAND, fine grained, trace fine grained gravel, grey, non-cohesive, wet, loose to compact		05	SS	7	21											1 45 (54)
			06	SS	27	0											
272.01																	
9.09	END OF SOIL BOREHOLE LOG - SEE RECORD OF ROCKCORE No. BH-26-02. Note(s): - Cave in encountered at approx. 2.31 mbgs upon completion of rock coring. - Groundwater mesaured at approx. 1.81 mbgs upon completion of rock coring. - It should be noted that groundwater conditions may not have stabilized upon completion of drilling.																

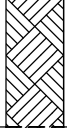
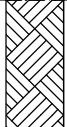


RECORD OF ROCKCORE No BH-26-02

1 OF 1

METRIC

JOB NUMBER 26-0649 LOCATION Bay Street Bridge, Magnetawan, ON ORIGINATED BY LM
CLIENT Municipality of Magnetawan DATUM Geodetic BOREHOLE TYPE HW/NQ COMPILED BY EM
DRILLER Landcore Drilling DATE 2026.03.03 NORTHING 5057500 EASTING 605247 CHECKED BY EG

Elev. Depth	Strat Plot	Description	Depth (m)	Ground Water Conditions	Sample No.	Recovery Parameters	RQD % 	Fracture index / ft	Discontinuity Data		Compressive Strength (MPa)	Remarks
									Dip Angles W.R.T Core Axis	Type and Description (see core log photos)		
281.1 0.00		Soil Overburden: Refer to RECORD OF BOREHOLE No. BH-26-02	1 2 3 4 5 6 7 8 9 10 11 12				25 50 75		30 60		50 100 150	
272.01 9.09		ORTHOGNEISS - Metamorphic Rock, fine to coarse grained, red, light grey and dark grey banding, fragmented zone with oxidation and trace sand infilling from 9.53 to 9.73 mbgs, fresh to faintly weathered, with drill induced breaks, strong rock, good quality based on RQD			01	TCR= 100% SCR= 72% RQD= 77%		1			67	
270.48 10.62		ORTHOGNEISS - same as above, no natural fractures, several drill induced breaks throughout the run, strong rock, excellent quality based on RQD			02	TCR= 100% SCR= 97% RQD= 100%					85.2	
268.96 12.14		END OF BOREHOLE - TARGET DEPTH ACHIEVED Note(s): - Cave in encountered at approx. 2.31 mbgs upon completion of drilling. - Groundwater measured at approx. 1.81 mbgs upon completion of drilling. - It should be noted that groundwater conditions may not have stabilized upon completion of drilling.										



Filling		Surface	Type	Aperture	Weathering
Clean - CL Iron Stained - Fe Manganese Stained - Mn Carbonate - C Gypsum - G Silty/Clay - SC	Clay - Cy Calcite - Cal Hematite - Hem Pyrite - Py Serpentine - St Chlorite - Ch	PL Planar CU Curved UN Undulating IR Irregular	BD Bedding JN Joint FLT Fault SH Shear KV Karstic Void	Tight: 0.1 - 0.5mm (T) Moderately Open: 0.5-2.5mm (MO) Open: 2.5-10mm (O) Very Open: > 10mm (VO)	FR Fresh SW Slightly MW Moderately HW Highly EW Extremely

5. ROCK CORE LOG 26-0649 BH LOGS ROCK.GPJ LOG A_RNGN01.GDT 26-4-2



RECORD OF BOREHOLE No BH-26-03

1 OF 1

METRIC

JOB NUMBER 26-0649 LOCATION Bay Street Bridge, Magnetawan, ON ORIGINATED BY LM
CLIENT Municipality of Magnetawan DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger COMPILED BY EM
DRILLER Landcore Drilling DATE 2026.03.03 NORTHING 5057510 EASTING 605266 CHECKED BY EG

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION (M)	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)			SHEAR STRENGTH kPa										
									20	40	60	80					100	WATER CONTENT (%)	
							○ POCKET PEN + FIELD VANE												
							● QUICK TRIAXIAL × LAB VANE												
281.28																GR	SA	SI	CL
280.00	ASPHALT - 25 mm thick (OH) ORGANIC CLAYEY SILT to (MH) CLAYEY SILT, high plasticity, dark brown to grey, cohesive, w~PL to w>PL		01	AS					281										AS01: transitions to (MH) with depth, transitions to grey with depth
279.76									280										
1.52	(CL) SILTY CLAY, low plasticity, grey, cohesive, w>PL, very stiff		01	SS	30	71													
279.07																			
2.21	(ML) SILT, trace fine grained sand, grey to brown, non-cohesive, wet, compact		02	SS	19	71			279										
278.31																			
2.97	(SM) SILTY SAND, fine to coarse grained, trace fine grained gravel, brown, non-cohesive, wet, compact		03	SS	12	54			278										Auger grinding during advancement from approx. 3.05 - 4.59 mbgs.
276.86																			
4.42	END OF BOREHOLE - AUGER AND SPLITSPOON REFUSAL ON INFERRED BEDROCK SURFACE Note(s): - Cave in encountered at approx. 3.35 mbgs upon completion of drilling and removal of augers. - Groundwater mesasured at approx. 3.05 mbgs upon completion of drilling and removal augers. - It should be noted that groundwater conditions may not have stabilized upon completion of drilling.								277										

3. SOIL REPORT+REC. (ELEVATION) 26-0649 BH LOGS SOIL.GPJ ONTARIO MTO.GDT 26-4-2

200 + : Numbers refer to Field Vane Over Limit + 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

JOB NUMBER 26-0649 LOCATION Bay Street Bridge, Magnetawan, ON ORIGINATED BY LM
 CLIENT Municipality of Magnetawan DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger COMPILED BY EM
 DRILLER Landcore Drilling DATE 2026.03.04 NORTHING 5057500 EASTING 605235 CHECKED BY EG

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION (M)	DYNAMIC CONE PENETRATION RESISTANCE PLOT					REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N° VALUES	RECOVERY RATIO (%)			20	40	60	80	100	
281.10	ASPHALT - 25 mm thick		01	AS				281						
280.00	FILL - (SW) SAND, fine to coarse grained, some non-plastic fines, some to trace fine to coarse grained gravel, brown to dark brown, non-cohesive, moist, dense		01	SS	38	88		280						
278.89	(PT) SILTY PEAT, dark brown to black, with wood debris, non-cohesive, wet, very loose		02	SS	3	42		279						
278.13	(SM) SILTY SAND, fine grained, grey, non-cohesive, wet, compact to very loose		03	SS	12	25		278						0 70 (30)
275.46	(ML) CLAYEY SILT, medium plasticity, grey, with wood debris, cohesive to non-cohesive, w>LL, very soft to firm		04	SS	3	0		277						
5.64			05	SS	1	100		276						
272.38			06	SS	WH	100		275						0 1 (99)
8.72	(ML) CLAYEY SILT, fine grained, medium plasticity, grey, cohesive, interlayered with silty sand seams, w>PL, very soft		01	VANE				274						
270.89	(SW) SAND, fine to coarse grained, some non-plastic fines, red to brown, non-cohesive, wet, compact		07	SS	WH	100		273						
269.37	(SM) SILTY SAND, fine to coarse grained, trace fine grained gravel, non-plastic, brown, non-cohesive, wet, dense		08	SS	11	58		272						
268.50	(SW/GW) SAND and GRAVEL, fine to coarse grained, trace non-plastic fines, brown, non-cohesive, wet, dense		09A	SS	32	100		271						4 80 (16)
268.00	END OF BOREHOLE - AUGER AND SPLITSPOON REFUSAL ON INFERRED BEDROCK SURFACE Note(s): - Cave in encountered at approx. 3.35 mbgs upon completion of drilling and removal of augers. - Groundwater measured at approx. 2.74 mbgs upon completion of drilling and removal of augers. - It should be noted that groundwater conditions may not have stabilized upon completion of drilling.		09B	SS	32	100		270						
12.75								269						

3. SOIL REPORT+REC. (ELEVATION) 26-0649 BH LOGS SOIL.GPJ ONTARIO MTO.GDT 26-4-2

APPENDIX E

LABORATORY DATA

WATER CONTENT TEST

TEST METHOD: LS 701 / ASTM C 566 / D 2216

CONTRACT NO: 26-0649

DATE SAMPLED: 2026-03-02

PROJECT: Bay St. Bridge Replacement

SOURCE: Boreholes

DATE TESTED: 2026-03-11

TESTED BY: T. Linley

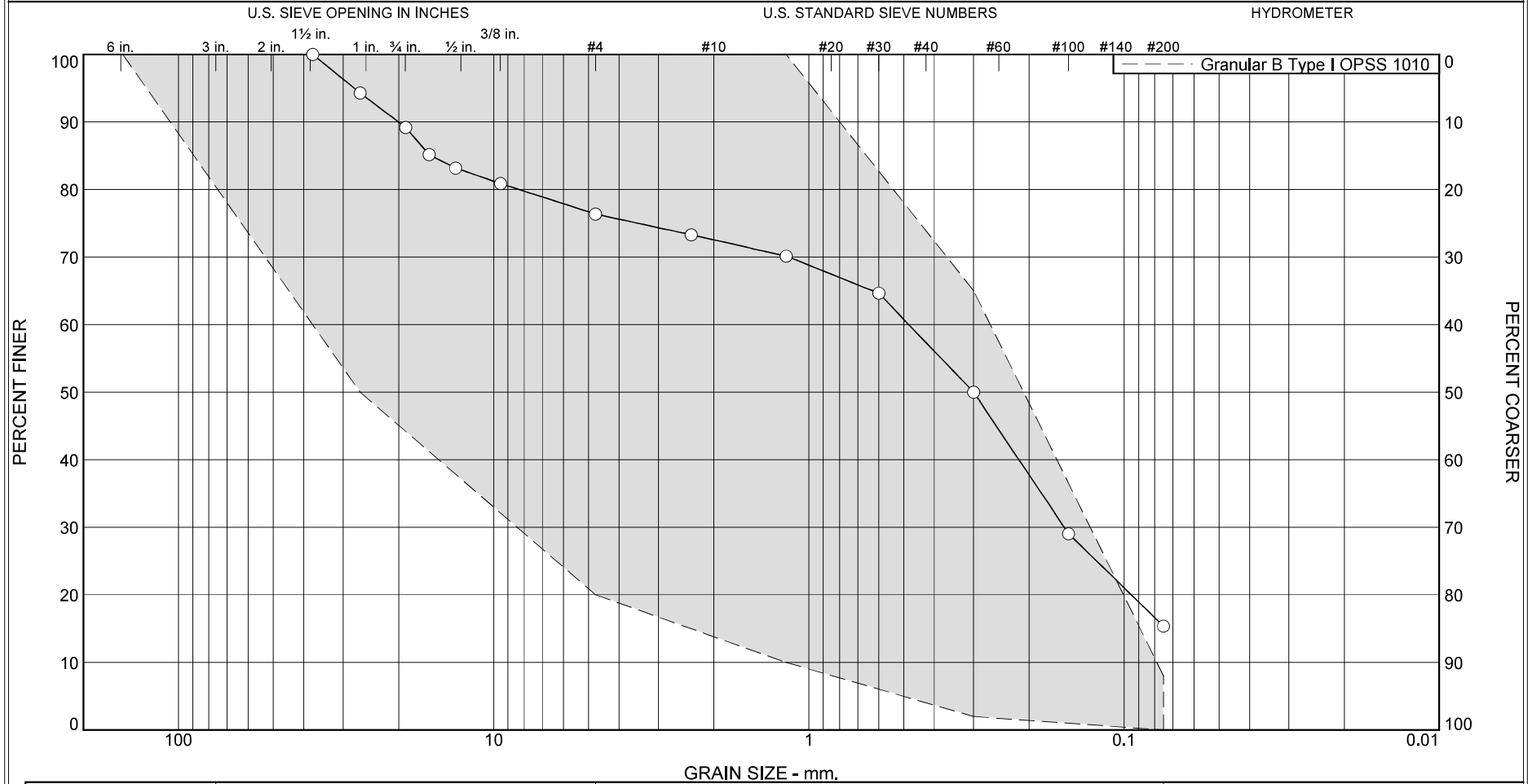
[illegible]

REMARKS:

CLIENT: Municipality of Magnetawan

COPIES TO:

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines
		Coarse	Fine	Coarse	Medium	Fine	Silt
○	0.0	10.8	12.8	3.9	15.2	41.9	15.4

Identification			Date Sampled	Date Received	Date Tested
Source of Sample: BH-26-01	Depth: 0.8m - 1.4m	Sample Number: SS01	Mar 2, 2026	Mar 9, 2026	Mar 13, 2026

Client Municipality of Magnetawan		 71 Black Road Unit 3 Sault Ste. Marie, ON P6B 0A3 T. 705 949.1457 F. 705 949.9606 TF. 866 806.6602 adam.byers@TULLOCH.ca
Project Bay St. Bridge Replacement		
Project No. 26-0649	Figure	

Tested By: T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2026-03-17

Client: Municipality of Magnetawan

Project: Bay St. Bridge Replacement

Project Number: 26-0649

Location: BH-26-01

Depth: 0.8m - 1.4m

Sample Number: SS01

Date Sampled: Mar 2, 2026

Date Received: Mar 9, 2026

Date Tested: Mar 13, 2026

Tested by: T. Linley

Material specification: Granular B Type I OPSS 1010

Sieve Test Data

Post #200 Wash Test Weights (grams): Dry Sample and Tare = 805.20

Tare Wt. = 166.20

Minus #200 from wash = 10.5%

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained	Lower Spec. Limit, %	Upper Spec. Limit, %	Deviation From Spec., %
879.90	166.20	37.5mm	0.00	0.00	100.0	0.0			
		26.5mm	40.80	0.00	94.3	5.7	50.0	100.0	
		19mm	36.60	0.00	89.2	10.8			
		16mm	28.70	0.00	85.1	14.9			
		13.2mm	14.00	0.00	83.2	16.8			
		9.5mm	16.40	0.00	80.9	19.1			
		#4	32.20	0.00	76.4	23.6	20.0	100.0	
		#8	22.00	0.00	73.3	26.7			
		#16	22.40	0.00	70.1	29.9	10.0	100.0	
		#30	39.30	0.00	64.6	35.4			
		#50	104.60	0.00	50.0	50.0	2.0	65.0	
		#100	149.40	0.00	29.0	71.0			
		#200	97.60	0.00	15.4	84.6	0.0	8.0	+7.4

Fractional Components

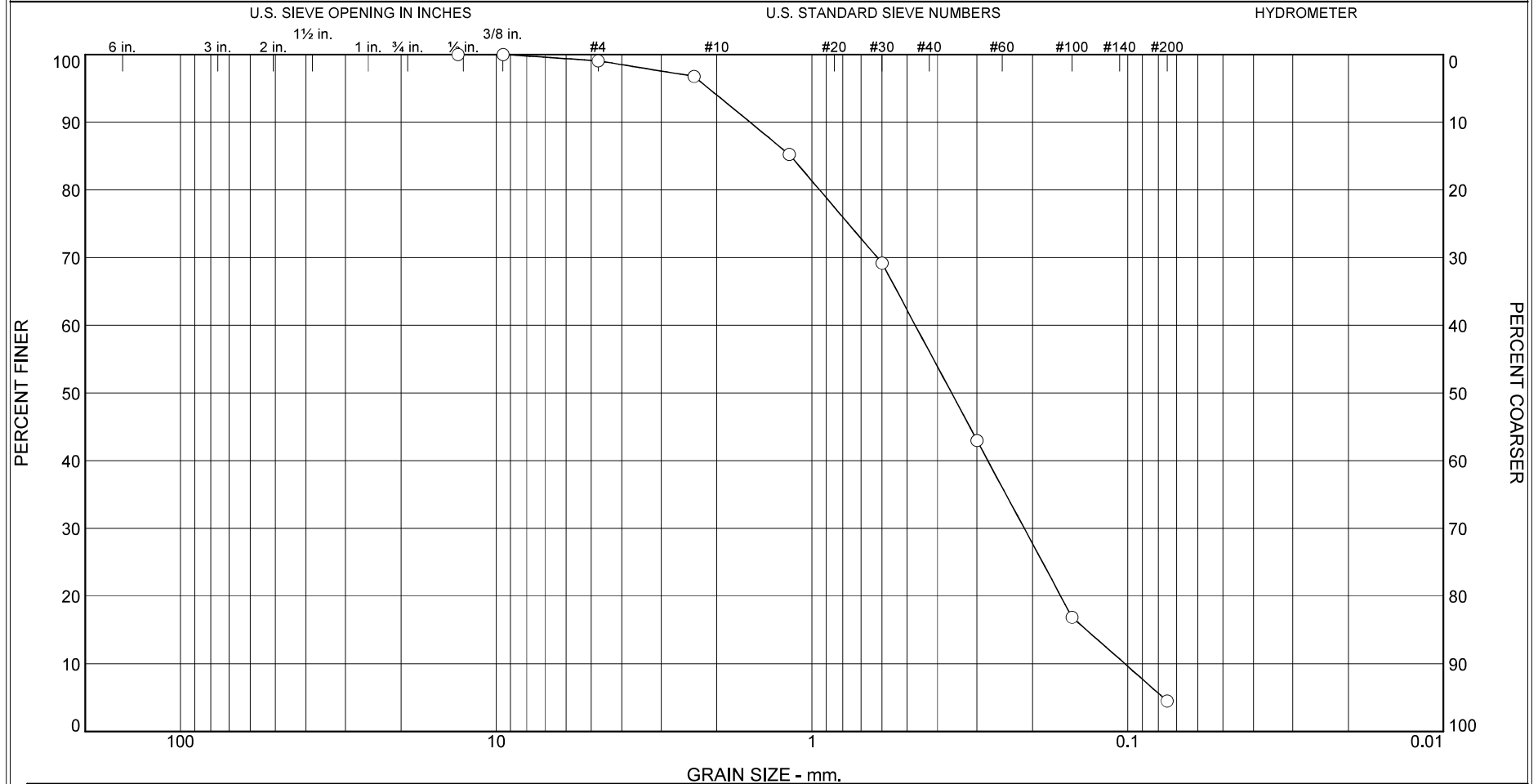
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	10.8	12.8	23.6	3.9	15.2	41.9	61.0			15.4

D ₅	D ₁₀	D ₁₅	D ₂₀	D ₃₀	D ₄₀	D ₅₀	D ₆₀	D ₈₀	D ₈₅	D ₉₀	D ₉₅
			0.0948	0.1548	0.2156	0.3003	0.4819	8.3059	15.7914	20.0706	27.6789

Fineness Modulus

2.66

Particle Size Distribution Report



GRAIN SIZE - mm.						
% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt
0.0	0.0	0.9	5.1	37.8	51.7	4.5

Identification			Date Sampled	Date Received	Date Tested
Source of Sample: BH-26-01	Depth: 6.1m - 6.7m	Sample Number: SS06	Mar 2, 2026	Mar 9, 2026	Mar 13, 2026

Client Municipality of Magnetawan		 71 Black Road Unit 3 Sault Ste. Marie, ON P6B 0A3 T. 705 949.1457 F. 705 949.9606 TF. 866 806.6602 adam.byers@TULLOCH.ca		
Project Bay St. Bridge Replacement				
Project No. 26-0649	Figure			

Tested By: T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2026-03-17

Client: Municipality of Magnetawan

Project: Bay St. Bridge Replacement

Project Number: 26-0649

Location: BH-26-01

Depth: 6.1m - 6.7m

Date Sampled: Mar 2, 2026

Tested by: T. Linley

Sample Number: SS06

Date Received: Mar 9, 2026

Date Tested: Mar 13, 2026

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
427.80	161.60	13.2mm	0.00	0.00	100.0	0.0
		9.5mm	0.00	0.00	100.0	0.0
		#4	2.50	0.00	99.1	0.9
		#8	6.10	0.00	96.8	3.2
		#16	30.70	0.00	85.2	14.8
		#30	42.70	0.00	69.2	30.8
		#50	69.80	0.00	43.0	57.0
		#100	69.50	0.00	16.9	83.1
		#200	32.90	0.00	4.5	95.5

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.9	0.9	5.1	37.8	51.7	94.6			4.5

D ₅	D ₁₀	D ₁₅	D ₂₀	D ₃₀	D ₄₀	D ₅₀	D ₆₀	D ₈₀	D ₈₅	D ₉₀	D ₉₅
0.0771	0.1021	0.1351	0.1630	0.2126	0.2772	0.3612	0.4705	0.9462	1.1683	1.5711	2.1219

Fineness Modulus	C _u	C _c
1.90	4.61	0.94

WATER CONTENT TEST

TEST METHOD: LS 701 / ASTM C 566 / D 2216

CONTRACT NO: 26-0649

DATE SAMPLED: 2026-03-02

PROJECT: Bay St. Bridge Replacement

SOURCE: Boreholes

DATE TESTED: 2026-03-11

TESTED BY: T. Linley

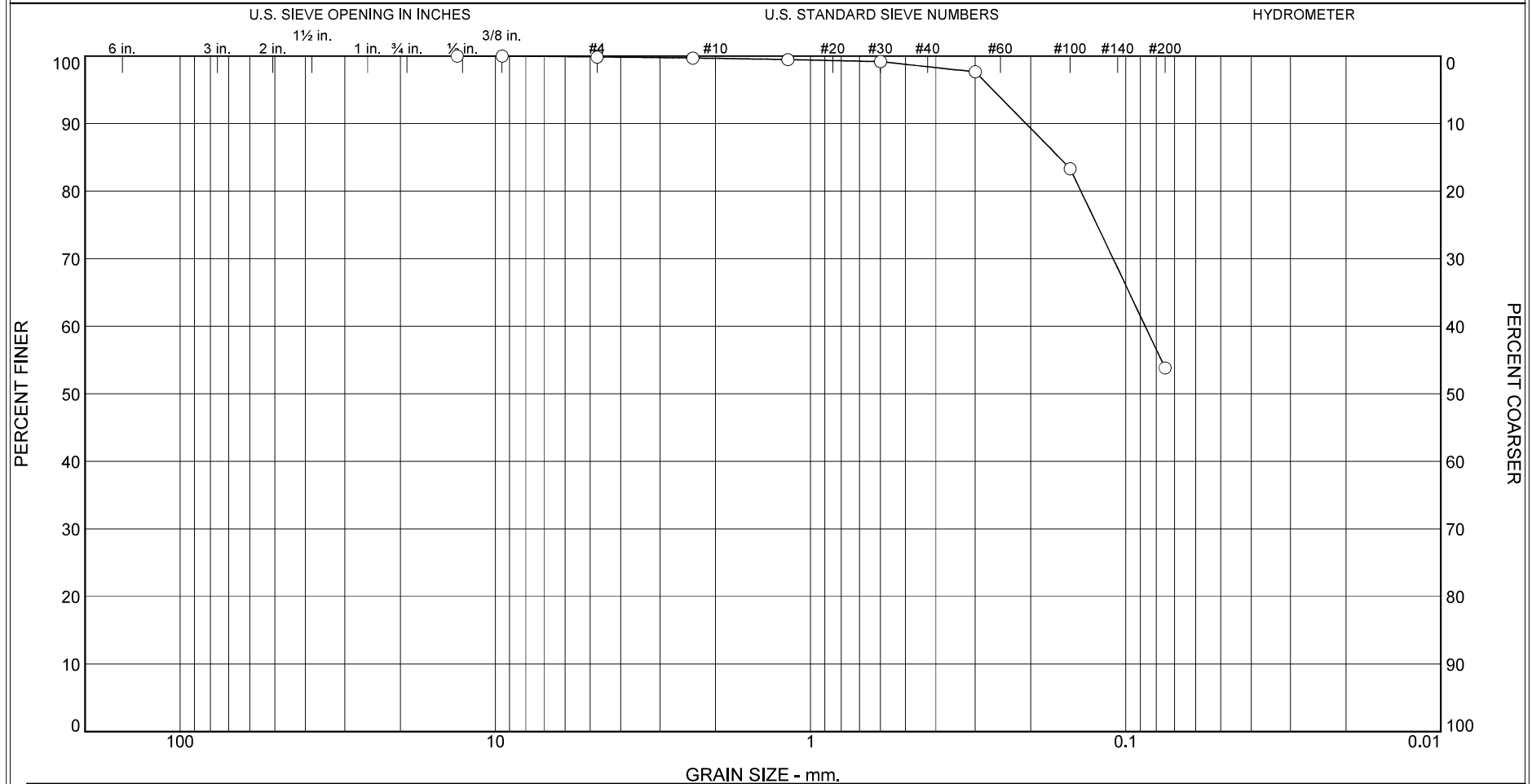
[illegible]

REMARKS:

CLIENT: Municipality of Magnetawan

COPIES TO:

Particle Size Distribution Report



GRAIN SIZE DISTRIBUTION TEST DATA

2026-03-17

Client: Municipality of Magnetawan

Project: Bay St. Bridge Replacement

Project Number: 26-0649

Location: BH-26-02

Depth: 6.1m - 6.7m

Sample Number: SS05

Date Sampled: Mar 2, 2026

Date Received: Mar 9, 2026

Date Tested: Mar 13, 2026

Tested by: T. Linley

Sieve Test Data

Post #200 Wash Test Weights (grams): Dry Sample and Tare = 302.90
Tare Wt. = 221.40
Minus #200 from wash = 38.2%

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
353.30	221.40	13.2mm	0.00	0.00	100.0	0.0
		9.5mm	0.00	0.00	100.0	0.0
		#4	0.20	0.00	99.8	0.2
		#8	0.20	0.00	99.7	0.3
		#16	0.30	0.00	99.5	0.5
		#30	0.40	0.00	99.2	0.8
		#50	2.00	0.00	97.6	2.4
		#100	18.90	0.00	83.3	16.7
		#200	38.90	0.00	53.8	46.2

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.2	0.2	0.2	1.2	44.6	46.0			53.8

D ₅	D ₁₀	D ₁₅	D ₂₀	D ₃₀	D ₄₀	D ₅₀	D ₆₀	D ₈₀	D ₈₅	D ₉₀	D ₉₅
							0.0867	0.1387	0.1627	0.2072	0.2639

Fineness Modulus
0.21

WATER CONTENT TEST

TEST METHOD: LS 701 / ASTM C 566 / D 2216

CONTRACT NO: 26-0649

DATE SAMPLED: 2026-03-03

PROJECT: Bay St. Bridge Replacement

SOURCE: Boreholes

DATE TESTED: 2026-03-11

TESTED BY: T. Linley

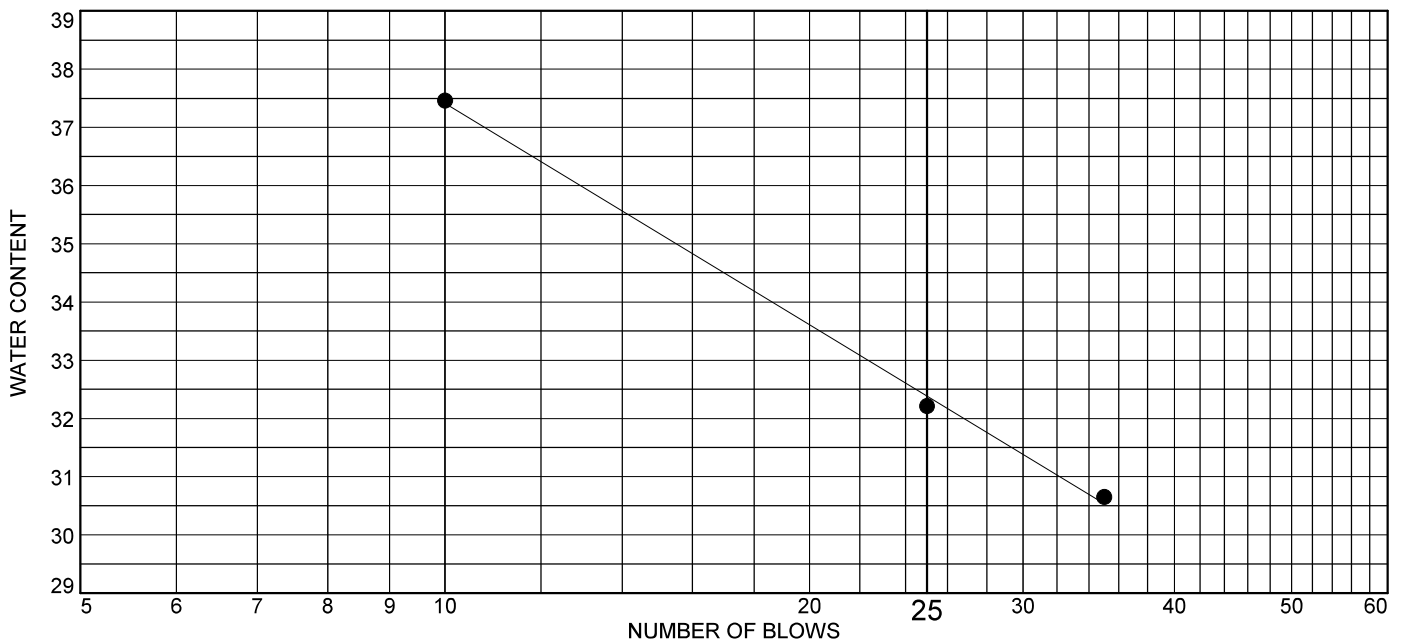
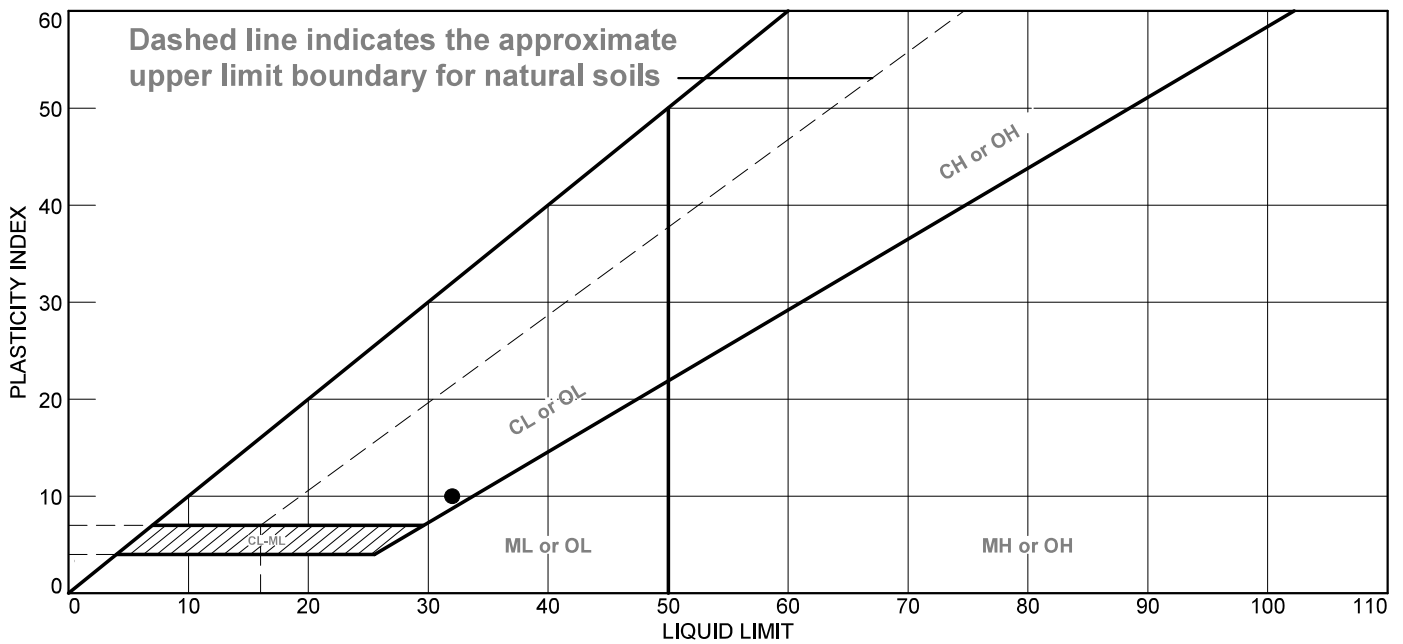
[illegible]

REMARKS:

CLIENT: Municipality of Magnetawan

COPIES TO:

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
•	32	22	10			

Project No. 26-0649 **Client:** Municipality of Magnetawan

Project: Bay St. Bridge Replacement

Source of Sample: BH-26-03 **Depth:** 1.5m - 2.1m

Sample Number: SS01

Remarks:



71 Black Road
Unit 3
Sault Ste.
Marie, ON
P6B 0A3

T. 705 949.1457
F. 705 949.9606
TF. 866 806.6602
adam.byers@TULLOCH.ca

Figure

Tested By: T. Linley

LIQUID AND PLASTIC LIMIT TEST DATA

2026-03-17

Client: Municipality of Magnetawan

Project: Bay St. Bridge Replacement

Project Number: 26-0649

Location: BH-26-03

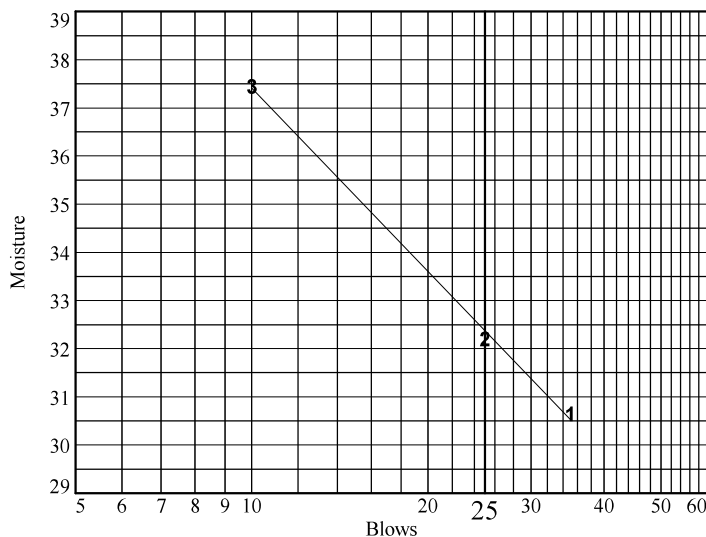
Depth: 1.5m - 2.1m

Sample Number: SS01

Tested by: T. Linley

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	38.53	39.00	34.81			
Dry+Tare	33.65	33.16	30.24			
Tare	17.73	15.03	18.04			
# Blows	35	25	10			
Moisture	30.7	32.2	37.5			



Liquid Limit= 32
 Plastic Limit= 22
 Plasticity Index= 10
 Natural Moisture= 25.4
 Liquidity Index= 0.3

Plastic Limit Data

Run No.	1	2	3	4	
Wet+Tare	31.70	30.06			
Dry+Tare	29.01	27.80			
Tare	16.74	17.35			
Moisture	21.9	21.6			

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
816.06	690.80	197.46	25.4

WATER CONTENT TEST

TEST METHOD: LS 701 / ASTM C 566 / D 2216

CONTRACT NO: 26-0649

DATE SAMPLED: 2026-03-04

PROJECT: Bay St. Bridge Replacement

SOURCE: Boreholes

DATE TESTED: 2026-03-11

TESTED BY: T. Linley

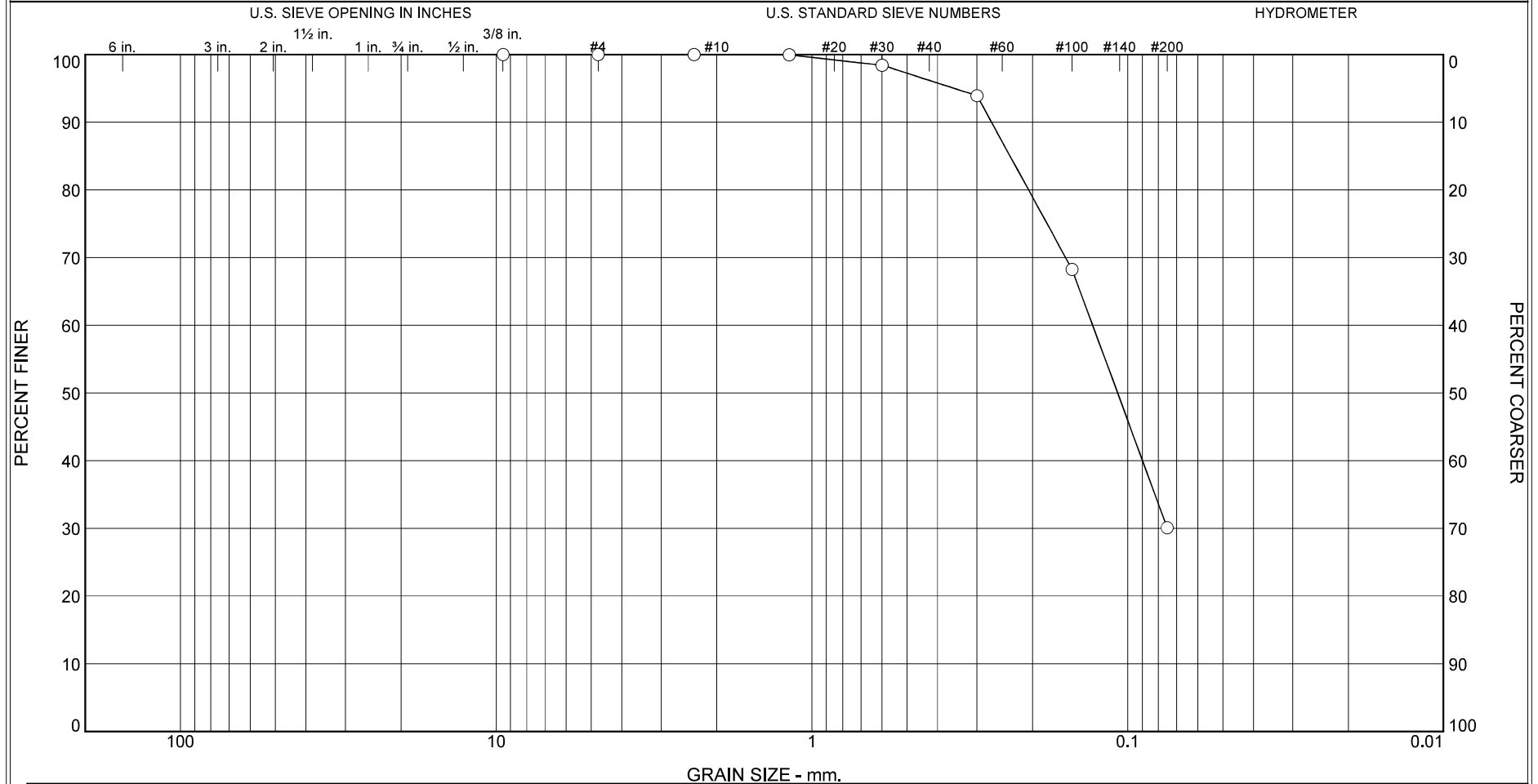
[illegible]

REMARKS:

CLIENT: Municipality of Magnetawan

COPIES TO:

Particle Size Distribution Report



GRAIN SIZE DISTRIBUTION TEST DATA

2026-03-17

Client: Municipality of Magnetawan
Project: Bay St. Bridge Replacement
Project Number: 26-0649
Location: BH-26-04
Depth: 3.0m - 3.7m
Date Sampled: Mar 4, 2026
Tested by: T. Linley

Sample Number: SS03
Date Received: Mar 9, 2026
Date Tested: Mar 16, 2026

Sieve Test Data

Post #200 Wash Test Weights (grams): Dry Sample and Tare = 408.70
Tare Wt. = 225.60
Minus #200 from wash = 17.4%

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
447.20	225.60	9.5mm	0.00	0.00	100.0	0.0
		#4	0.00	0.00	100.0	0.0
		#8	0.00	0.00	100.0	0.0
		#16	0.10	0.00	100.0	0.0
		#30	3.40	0.00	98.4	1.6
		#50	10.00	0.00	93.9	6.1
		#100	56.90	0.00	68.2	31.8
		#200	84.50	0.00	30.1	69.9

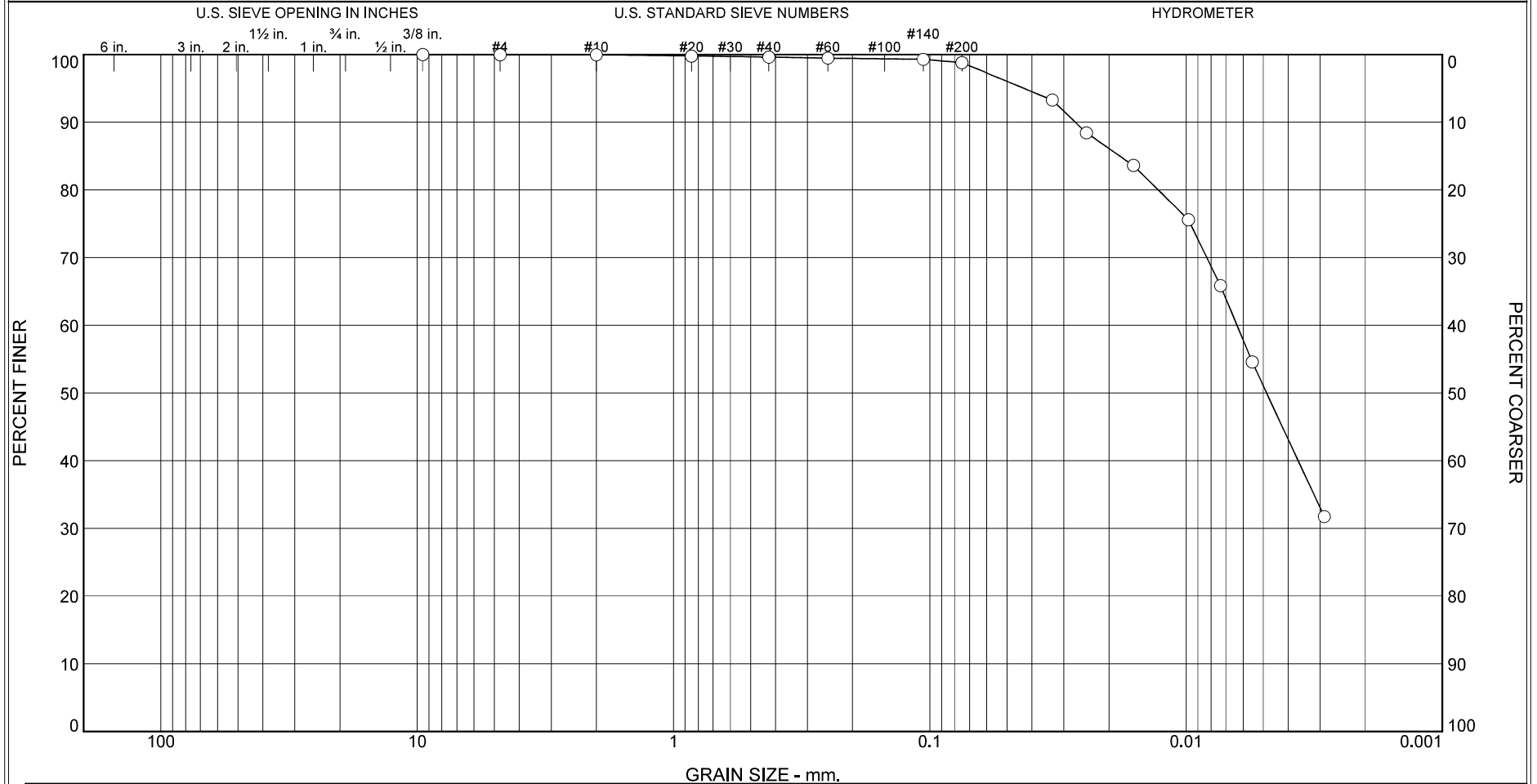
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	0.0	3.8	66.1	69.9			30.1

D ₅	D ₁₀	D ₁₅	D ₂₀	D ₃₀	D ₄₀	D ₅₀	D ₆₀	D ₈₀	D ₈₅	D ₉₀	D ₉₅
					0.0898	0.1077	0.1292	0.2061	0.2359	0.2700	0.3548

Fineness Modulus
0.39

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.3	0.8	98.8	

Identification			Date Sampled	Date Received	Date Tested
Source of Sample: BH-26-04 Depth: 7.6m - 8.2m Sample Number: SS06			Mar 4, 2026	Mar 9, 2026	Mar 13, 2026
Client Municipality of Magnetawan			 71 Black Road Unit 3 Sault Ste. Marie, ON P6B 0A3 T. 705 949.1457 F. 705 949.9606 TF. 866 806.6602 adam.byers@TULLOCH.ca		
Project Bay St. Bridge Replacement					
Project No. 26-0649	Figure				

Tested By: T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2026-03-17

Client: Municipality of Magnetawan

Project: Bay St. Bridge Replacement

Project Number: 26-0649

Location: BH-26-04

Depth: 7.6m - 8.2m

Sample Number: SS06

Date Sampled: Mar 4, 2026

Date Received: Mar 9, 2026

Date Tested: Mar 13, 2026

Tested by: T. Linley

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
1193.90	349.60	9.5mm.	0.00	0.00	100.0	0.0
		#4	0.30	0.00	100.0	0.0
		#10	0.20	0.00	99.9	0.1
61.40	0.00	#20	0.10	0.00	99.8	0.2
		#40	0.10	0.00	99.6	0.4
		#60	0.10	0.00	99.5	0.5
		#140	0.10	0.00	99.3	0.7
		#200	0.30	0.00	98.8	1.2

Hydrometer Test Data

Hydrometer test uses material passing #10

Percent passing #10 based upon complete sample = 99.9

Weight of hydrometer sample = 61.4

Automatic temperature correction

Composite correction (fluid density and meniscus height) at 20 deg. C = -5

Meniscus correction only = -1.0

Specific gravity of solids = 2.70

Hydrometer type = 152H

Hydrometer effective depth equation: $L = 16.294964 - .164 \times R_m$

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer	Percent Retained
1.00	19.9	63.0	57.9	0.0135	62.0	6.1	0.0333	93.3	6.7
2.00	19.9	60.0	54.9	0.0135	59.0	6.6	0.0245	88.4	11.6
5.00	19.9	57.0	51.9	0.0135	56.0	7.1	0.0161	83.6	16.4
15.00	19.9	52.0	46.9	0.0135	51.0	7.9	0.0098	75.6	24.4
30.00	19.8	46.0	40.9	0.0135	45.0	8.9	0.0073	65.9	34.1
60.00	19.7	39.0	33.9	0.0135	38.0	10.1	0.0055	54.6	45.4
250.00	23.2	24.0	19.7	0.0129	23.0	12.5	0.0029	31.7	68.3

Fractional Components

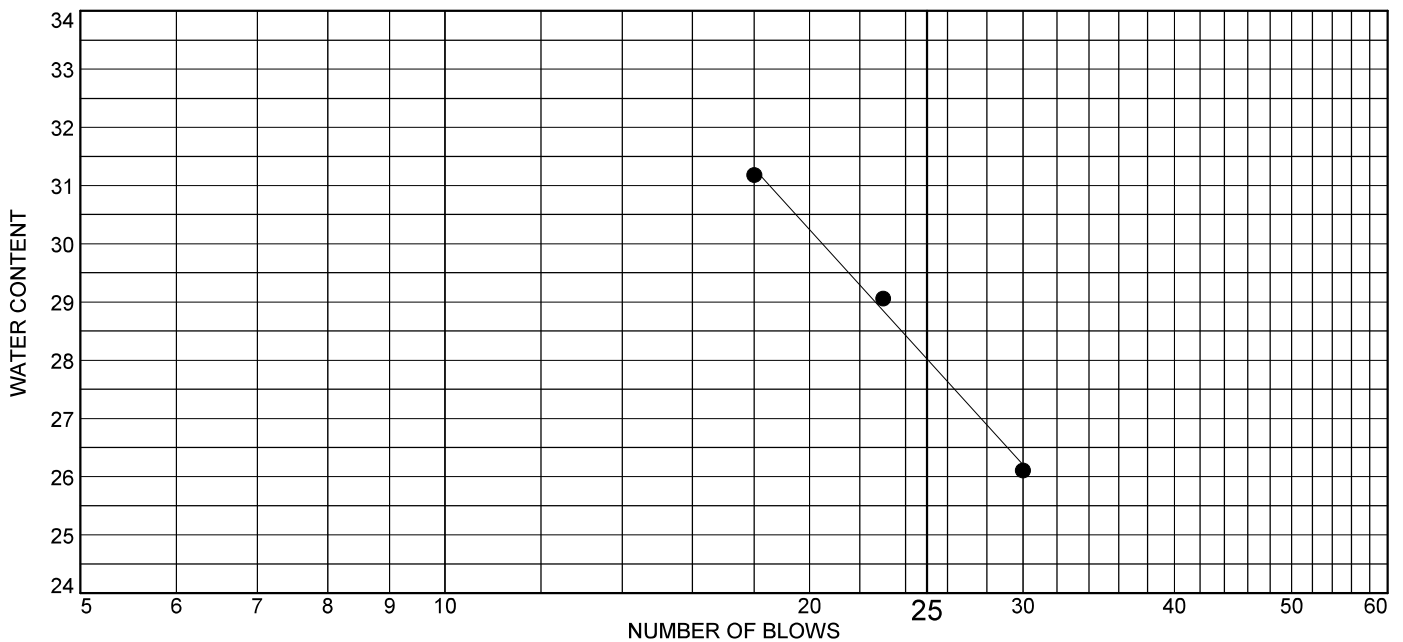
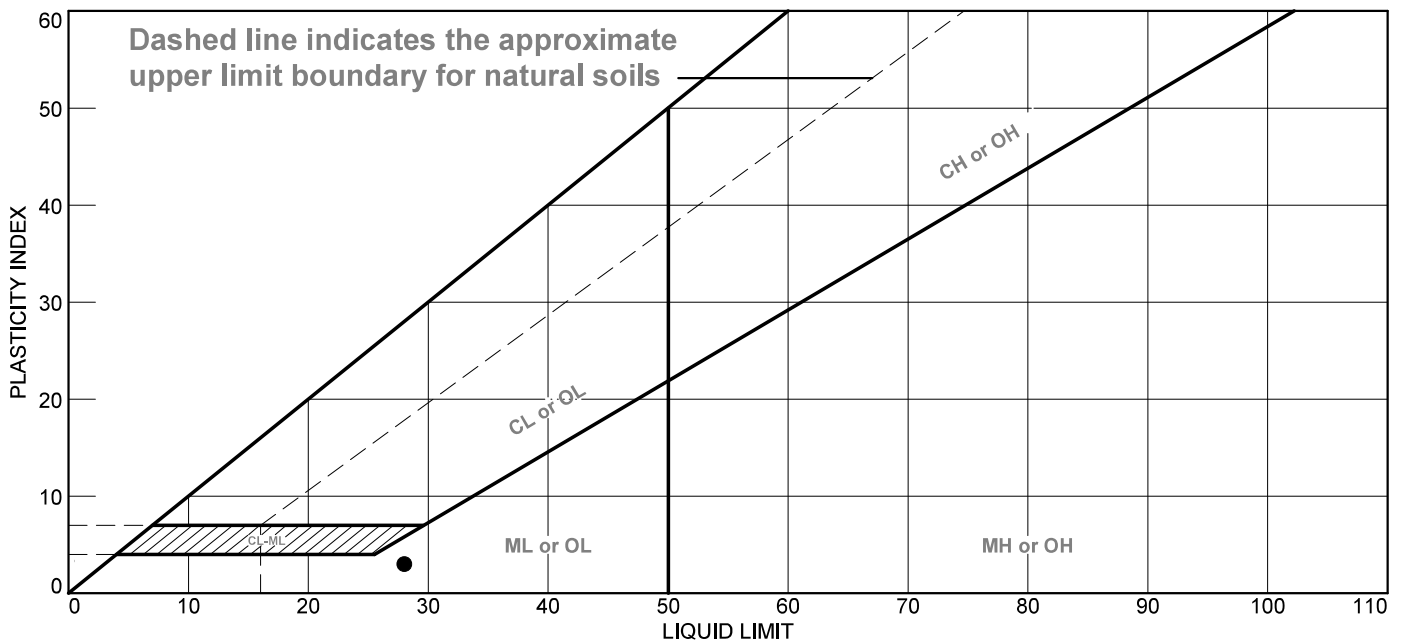
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	0.1	0.3	0.8	1.2			98.8

D ₅	D ₁₀	D ₁₅	D ₂₀	D ₃₀	D ₄₀	D ₅₀	D ₆₀	D ₈₀	D ₈₅	D ₉₀	D ₉₅
					0.0037	0.0049	0.0063	0.0129	0.0181	0.0271	0.0430

Fineness Modulus
0.02

Tulloch Engineering Inc.

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	28	25	3	99.6	98.8	ML

Project No. 26-0649 **Client:** Municipality of Magnetawan

Project: Bay St. Bridge Replacement

Source of Sample: BH-26-04 **Depth:** 7.6m - 8.2m

Sample Number: SS06

Remarks:



71 Black Road
Unit 3
Sault Ste.
Marie, ON
P6B 0A3

T. 705 949.1457
F. 705 949.9606
TF. 866 806.6602
adam.byers@TULLOCH.ca

Figure

Tested By: T. Linley

LIQUID AND PLASTIC LIMIT TEST DATA

2026-03-17

Client: Municipality of Magnetawan

Project: Bay St. Bridge Replacement

Project Number: 26-0649

Location: BH-26-04

Depth: 7.6m - 8.2m

Sample Number: SS06

%<#40: 99.6

%<#200: 98.8

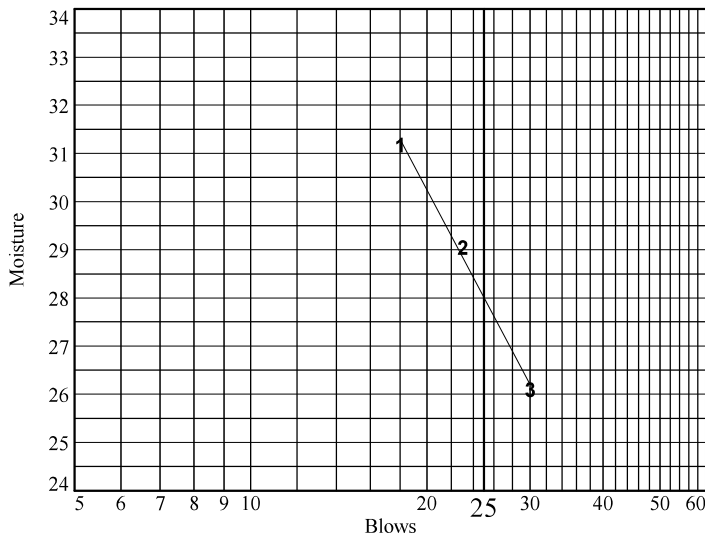
USCS: ML

AASHTO: A-4(3)

Tested by: T. Linley

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	40.58	36.40	42.34			
Dry+Tare	34.79	31.78	36.79			
Tare	16.22	15.88	15.53			
# Blows	18	23	30			
Moisture	31.2	29.1	26.1			



Liquid Limit= 28
 Plastic Limit= 25
 Plasticity Index= 3
 Natural Moisture= 37.3
 Liquidity Index= 4.1

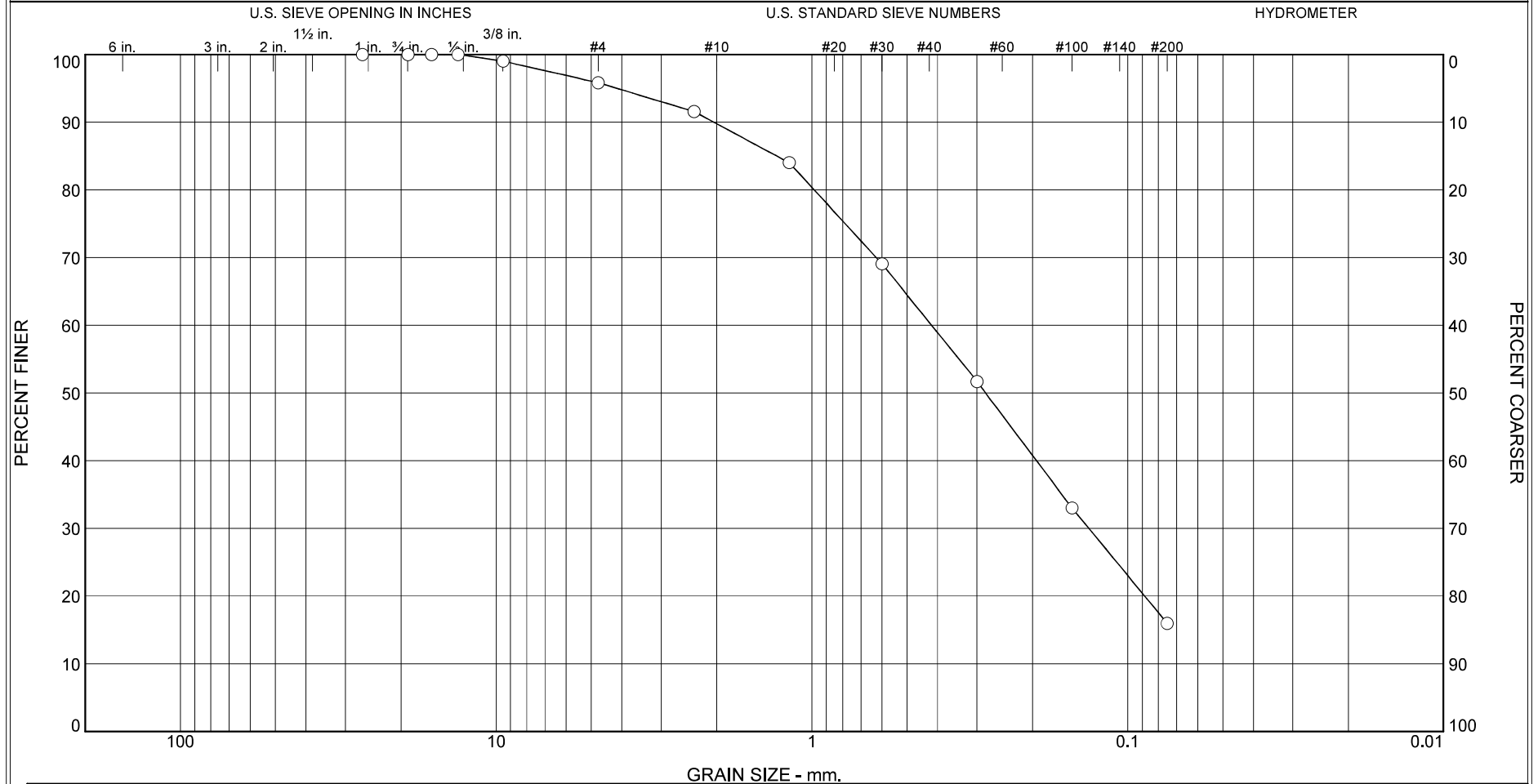
Plastic Limit Data

Run No.	1	2	3	4	
Wet+Tare	30.65	34.08			
Dry+Tare	27.85	30.67			
Tare	16.39	16.83			
Moisture	24.4	24.6			

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
1508.44	1193.87	349.61	37.3

Particle Size Distribution Report



GRAIN SIZE - mm.						
% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt
0.0	0.0	4.2	6.0	29.4	44.4	16.0

Identification			Date Sampled	Date Received	Date Tested
Source of Sample: BH-26-04	Depth: 12.2m - 12.6m	Sample Number: SS09A	Mar 4, 2026	Mar 9, 2026	Mar 16, 2026

Client Municipality of Magnetawan		 71 Black Road Unit 3 Sault Ste. Marie, ON P6B 0A3 T. 705 949.1457 F. 705 949.9606 TF. 866 806.6602 adam.byers@TULLOCH.ca		
Project Bay St. Bridge Replacement				
Project No. 26-0649	Figure			

Tested By: T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2026-03-17

Client: Municipality of Magnetawan

Project: Bay St. Bridge Replacement

Project Number: 26-0649

Location: BH-26-04

Depth: 12.2m - 12.6m

Sample Number: SS09A

Date Sampled: Mar 4, 2026

Date Received: Mar 9, 2026

Date Tested: Mar 16, 2026

Tested by: T. Linley

Sieve Test Data

Post #200 Wash Test Weights (grams): Dry Sample and Tare = 784.40
Tare Wt. = 232.10
Minus #200 from wash = 12.0%

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
860.00	232.10	26.5mm	0.00	0.00	100.0	0.0
		19mm	0.00	0.00	100.0	0.0
		16mm	0.00	0.00	100.0	0.0
		13.2mm	0.00	0.00	100.0	0.0
		9.5mm	6.20	0.00	99.0	1.0
		#4	20.00	0.00	95.8	4.2
		#8	26.80	0.00	91.6	8.4
		#16	47.40	0.00	84.0	16.0
		#30	93.80	0.00	69.1	30.9
		#50	109.10	0.00	51.7	48.3
		#100	117.40	0.00	33.0	67.0
		#200	106.90	0.00	16.0	84.0

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	4.2	4.2	6.0	29.4	44.4	79.8			16.0

D ₅	D ₁₀	D ₁₅	D ₂₀	D ₃₀	D ₄₀	D ₅₀	D ₆₀	D ₈₀	D ₈₅	D ₉₀	D ₉₅
			0.0884	0.1328	0.1945	0.2817	0.4178	0.9841	1.2923	2.0452	4.1477

Fineness Modulus
1.76



CSA A283 Certified Laboratory for Concrete Testing
CCIL Certified Laboratory for Aggregates and Asphalt Testing
CSA/CCIL Certified Technicians

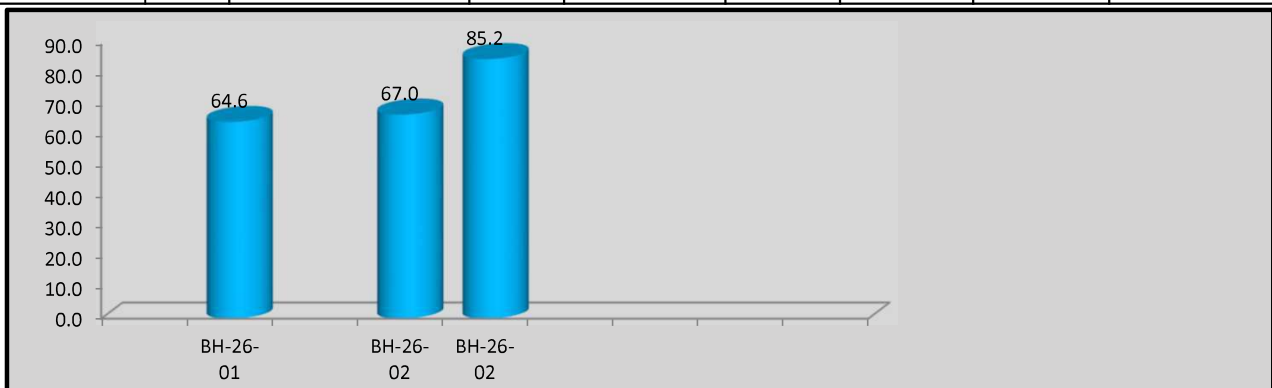


Rock Core Compressive Strength Report

PROJECT: Bay St. Bridge Replacement
DATE SAMPLED: 2026-03-02
DATE TESTED: 2026-03-17

CONTRACT: 26-0649
RUN BY: S. Campbell
SOURCE: Boreholes

Sample Location	Run #	Distance from top of run (cm)	Height (mm)	Diameter (mm)	L/D Ratio	Correction Factor	Peak Load (lbs)	Compressive Strength (Mpa)
BH-26-01	2	29	94.4	47.2	2.0	1.0	25400	64.6
BH-26-02	1	33	94.7	47.0	2.0	1.0	26100	67.0
BH-26-02	2	26	94.6	47.0	2.0	1.0	33200	85.2



REMARKS:

CLIENT: Municipality of Magnetawan

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CERTIFICATE OF ANALYSIS

Client: Trevor Linley
Company: Tulloch Engineering - Sault Ste. Marie
Address: 71 Black Road Unit 8
Sault Ste. Marie, ON, P6B 0A3
Phone:
Email: trevor.linley@tulloch.ca

Work Order Number: 614622
PO #:
Regulation: Information not provided
Project #: 26-0649
DWS #:
Sampled By: Tulloch

Date Order Received: 3/17/2026
Arrival Temperature: 5.4 C

Analysis Started: 3/17/2026
Analysis Completed: 3/25/2026

WORK ORDER SUMMARY

ANALYSES WERE PERFORMED ON THE FOLLOWING SAMPLES. THE RESULTS RELATE ONLY TO THE ITEMS TESTED.

Sample Description	Lab ID	Matrix	Type	Comments	Date Collected	Time Collected
BH-26-01 SS02	2260300	Soil	Grab		3/2/2026	
BH-26-02 SS03	2260301	Soil	Grab		3/2/2026	

METHODS AND INSTRUMENTATION

THE FOLLOWING METHODS WERE USED FOR YOUR SAMPLE(S):

Method	Lab	Description	Reference
Anions Soil (A5)	Garson	Determination of Anions in Soil	Modified from SW846-9056A
Cond Soil (A12)	Garson	Determination of conductivity in soil (1:2)	Modified from EPA SW846-9050A
Moisture (A99)	Garson	Determination of Percent Moisture	In-House
pH Soil (A2.0)	Garson	Determination of soil pH by Ion Selective Electrode	Modified from EPA SW-846 9045D
RedOx - Soil (T06)	Mississauga	Determination of RedOx Potential of Soil	Modified from APHA-2580B
Resistivity Soil (R12)	Garson	Determination of Resistivity in Soil (1:2)	Modified from Carter 18.3
Special Testing (A99)	Garson	Special Testing	In-House
Sulphide/S (R98)	Garson	Determination of Sulphide in Soil	In-House



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Tulloch Engineering - Sault Ste. Marie

Work Order Number: 614622

REPORT COMMENTS

RedOx - Soil (A6): Hold time exceeded for methods BEFORE receipt at lab

The corrosivity Index (AWWA C-105) has been determined as follows:

Sample 2260300, Good drainage, generally dry: 0

Sample 2260300, Fair drainage, generally moist: 1

Sample 2260300, Poor drainage, continuously wet: 2

Sample 2260301, Good drainage, generally dry: 0

Sample 2260302, Fair drainage, generally moist: 1

Sample 2260303, Poor drainage, continuously wet: 2

This report has been approved by:

Aline de Cheigny

Production Coordinator CET

WORK ORDER RESULTS

Sample Description	BH - 26 - 01 SS02		BH - 26 - 02 SS03		
Sample Date	3/2/2026 12:00 AM		3/2/2026 12:00 AM		
Lab ID	2260300		2260301		
Anions (Soil)	Result	MDL	Result	MDL	Units
Bromide	<0.2	0.2	<0.2	0.2	µg/g
Chloride	30.1	0.4	57.6	0.4	µg/g
Fluoride	0.3	0.2	0.5	0.2	µg/g
Nitrate (as N)	<0.2	0.2	<0.2	0.2	µg/g
Nitrite (as N)	<0.1	0.1	<0.1	0.1	µg/g
Sulphate	33	2	26	2	µg/g

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Tulloch Engineering - Sault Ste. Marie

Work Order Number: 614622

Sample Description	BH - 26 - 01 SS02		BH - 26 - 02 SS03		
Sample Date	3/2/2026 12:00 AM		3/2/2026 12:00 AM		
Lab ID	2260300		2260301		
Custom Analysis	Result	MDL	Result	MDL	Units
Custom Analysis	SeeComments	N/A	SeeComments	N/A	NA

Sample Description	BH - 26 - 01 SS02		BH - 26 - 02 SS03		
Sample Date	3/2/2026 12:00 AM		3/2/2026 12:00 AM		
Lab ID	2260300		2260301		
General Chemistry	Result	MDL	Result	MDL	Units
% Moisture	33.8	0.1	5.6	0.1	%
Conductivity	154 [149]	1	221	1	µS/cm
pH	4.65 [4.54]	N/A	6.88	N/A	pH
RedOx (vs. S.H.E.)	348	N/A	367 [369]	N/A	mV
Resistivity (Calc.)	6490	N/A	4520	N/A	ohm-cm
Sulphide	<0.4 [<0.4]	0.4	<0.3	0.3	µg/g



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Tulloch Engineering - Sault Ste. Marie

Work Order Number: 614622

LEGEND

Dates: Dates are formatted as mm/dd/year throughout this report.

MDL: Method detection limit or minimum reporting limit.

[]: Results for laboratory replicates are shown in square brackets immediately below the associated sample result for ease of comparison.

Quality Control: All associated Quality Control data is available on request.

LCL: Lower Control Limit.

UCL: Upper Control Limit.

QAQCID: This is a unique reference to the quality control data set used to generate the reported value. Contact our lab for this information, as it is traceable through our LIMS.

Field Data: Reports containing Field Parameters represent data that has been collected and provided by the client. Testmark is not responsible for the validity of this data which may be used in subsequent calculations.

Sample Condition Deviations: A noted sample condition deviation may affect the validity of the result. Results apply to the sample(s) as received.

Reproduction of Report: Report shall not be reproduced, except in full, without the approval of Testmark Laboratories Ltd.

Regulation Comparisons: Disclaimer: Please note that regulation criteria are provided for comparative purposes, however the onus on ensuring the validity of this comparison rests with the client.

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Tulloch Engineering - Sault Ste. Marie

Work Order Number: 614622

QUALITY CONTROL DATA

THIS SECTION REPORTS QC RESULTS ASSOCIATED WITH THE TEST BATCH; THESE ARE NOT YOUR SAMPLE RESULTS.

QAQC details include only values where sufficient sample data allowed measurement.

Anions (Soil)

Blank: LRB-6 (Blank) (6)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Nitrite (as N)	0.1	µg/g	0	<0.1	0.18	20260319.A5D
Sulphate	0.4	µg/g	0	<0.4	6	20260319.A5D
Bromide	0.2	µg/g	0	<0.2	0.6	20260319.A5D
Chloride	0.4	µg/g	0	<0.4	1.2	20260319.A5D
Fluoride	0.02	µg/g	0	<0.02	0.6	20260319.A5D
Nitrate (as N)	0.2	µg/g	0	<0.2	0.6	20260319.A5D

Positive Control: LFB-7 (0.2/0.1/0.02 mg/g equiv) (7)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Nitrite (as N)	N/A	%	80	94.2	120	20260319.A5D
Nitrate (as N)	N/A	%	80	98.7	120	20260319.A5D
Sulphate	N/A	%	80	102	120	20260319.A5D
Fluoride	N/A	%	80	98.8	120	20260319.A5D
Chloride	N/A	%	80	104	120	20260319.A5D
Bromide	N/A	%	80	94.7	120	20260319.A5D

Positive Control: LFB-5 (0.1/0.02/0.002 mg/g equiv) (5)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Nitrate (as N)	N/A	%	80	102	120	20260319.A5D
Nitrite (as N)	N/A	%	80	94.8	120	20260319.A5D
Sulphate	N/A	%	80	99.7	120	20260319.A5D
Fluoride	N/A	%	80	92	120	20260319.A5D
Chloride	N/A	%	80	97	120	20260319.A5D
Bromide	N/A	%	80	94.4	120	20260319.A5D

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Tulloch Engineering - Sault Ste. Marie

Work Order Number: 614622

Sample Replicate: % RPD (8)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Sulphate	N/A	%	0	3.2	35	20260319.A5D
Chloride	N/A	%	0	4.6	35	20260319.A5D

General Chemistry**Method Blank: Method Blank (1)**

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Conductivity	1	µS/cm	0	<1	5	20260319.TM-G.A12B

Sample Replicate: % RPD (3)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
pH	N/A	pH	0	0.11	0.3	20260318.TM-G.R2D

Sample Replicate: % RPD (8)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Conductivity	N/A	%	0	3.3	10	20260319.TM-G.A12B

Positive Control: Lab Control - 200 (7)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
RedOx (vs. S.H.E.)	N/A	mV	175	206	225	20260323.TM-M.A6B

Positive Control: LRB-6 (Blank) (6)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Sulphide	0.02	µg/g	0	<0.02	0.06	20260324.R98B

Positive Control: ORP - Soil Control 90 (8)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
RedOx (vs. S.H.E.)	N/A	mV	75	103	105	20260323.TM-M.A6B

Calibration Check: Lab Control Sample (2)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Conductivity	N/A	%	475	501	525	20260319.TM-G.A12B

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Tulloch Engineering - Sault Ste. Marie

Work Order Number: 614622

Positive Control: LFB-7 (7)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Sulphide	0.05	µg/g	0.24	0.271	0.36	20260324.R98B

Positive Control: LCS (pH 8) (2)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
pH	N/A	pH	7.8	8.01	8.2	20260318.TM-G.R2D

Sample Replicate: % RPD (9)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
RedOx (vs. S.H.E.)	N/A	%	0	0.5	10	20260323.TM-M.A6B

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Tulloch Engineering - Sault Ste. Marie

Work Order Number: 614622

THIS INDEX SHOWS HOW YOUR SAMPLES ARE ASSOCIATED TO THE CONTROLS INCLUDED IN THE IDENTIFIED BATCHES.

Sample Description	Lab ID	Method	QAQCID	Prep QAQCID
BH - 26 - 01 SS02	2260300	Anions Soil (A5)	20260319.A5D	20260318.A109B
BH - 26 - 01 SS02	2260300	Cond Soil (A12)	20260319.TM-G.A12B	20260318.A109B
BH - 26 - 01 SS02	2260300	Moisture (A99)	20260318.TM-G.A99B	
BH - 26 - 01 SS02	2260300	pH Soil (A2.0)	20260318.TM-G.R2D	20260318.A109C
BH - 26 - 01 SS02	2260300	RedOx - Soil (T06)	20260323.TM-M.A6B	20260323.A109B
BH - 26 - 01 SS02	2260300	Resistivity Soil (R12)	20260324.TM-G.R12B	20260318.A109B
BH - 26 - 01 SS02	2260300	Special Testing (A99)	20260325.TM-G.A99CmethB	
BH - 26 - 01 SS02	2260300	Sulphide/S (R98)	20260324.R98B	20260318.A109C
BH - 26 - 01 SS02	2260300r	Cond Soil (A12)	20260319.TM-G.A12B	20260318.A109B
BH - 26 - 01 SS02	2260300r	pH Soil (A2.0)	20260318.TM-G.R2D	20260318.A109C
BH - 26 - 01 SS02	2260300r	Sulphide/S (R98)	20260324.R98B	20260318.A109C
BH - 26 - 02 SS03	2260301	Anions Soil (A5)	20260319.A5D	20260318.A109B
BH - 26 - 02 SS03	2260301	Cond Soil (A12)	20260319.TM-G.A12B	20260318.A109B
BH - 26 - 02 SS03	2260301	Moisture (A99)	20260318.TM-G.A99B	
BH - 26 - 02 SS03	2260301	pH Soil (A2.0)	20260318.TM-G.R2D	20260318.A109C
BH - 26 - 02 SS03	2260301	RedOx - Soil (T06)	20260323.TM-M.A6B	20260323.A109B
BH - 26 - 02 SS03	2260301	Resistivity Soil (R12)	20260324.TM-G.R12B	20260318.A109B
BH - 26 - 02 SS03	2260301	Special Testing (A99)	20260325.TM-G.A99CmethB	
BH - 26 - 02 SS03	2260301	Sulphide/S (R98)	20260324.R98B	20260318.A109C
BH - 26 - 02 SS03	2260301r	RedOx - Soil (T06)	20260323.TM-M.A6B	20260323.A109B

APPENDIX F

ROCK CORE PHOTOS

Retrieved Rock Core at Borehole Location BH-26-01

Top of Bedrock Elevation: 8.08 mbgs / 273.24 masl



Bottom of Core Elevation: 11.46 mbgs / 269.86 masl

CLIENT
Municipality of Magnetawan .

PROJECT
Bay Street Bridge Rock Core Photos

CONSULTANT



YYYY-MM-DD	2026-04-01
PREPARED	EM
DESIGN	EM
REVIEW	EG
APPROVED	GL

TITLE

Rock Core Photos – BH-26-01

PROJECT No.
26-0649

Phase / Task
104

Rev.
0

Figure
F-1

Photos of Bedrock Core – Detailed Photos

BH-26-01

1



2



3



4



CLIENT
Municipality of Magnetawan .

CONSULTANT



YYYY-MM-DD	2026-04-01
PREPARED	EM
DESIGN	EM
REVIEW	EG
APPROVED	GL

PROJECT
Bay Street Bridge Rock Core Photos

TITLE
Rock Core Photos – BH-26-01

PROJECT No.	Phase / Task	Rev.	Figure
26-0649	104	0	F-2

Retrieved Rock Core at Borehole Location BH-26-02

Top of Bedrock Elevation: 9.09 mbgs / 272.01 masl



Bottom of Core Elevation: 12.14 mbgs / 268.96 masl

CLIENT
Municipality of Magnetawan .

PROJECT
Bay Street Bridge Rock Core Photos

CONSULTANT



YYYY-MM-DD 2026-04-01

PREPARED EM

DESIGN EM

REVIEW EG

APPROVED GL

TITLE

Rock Core Photos – BH-26-02

PROJECT No.
26-0649

Phase / Task
104

Rev.
0

Figure
F-3

Photos of Bedrock Core – Detailed Photos

BH-26-02

1



2



3



4



CLIENT
Municipality of Magnetawan .

CONSULTANT



YYYY-MM-DD	2026-04-01
PREPARED	EM
DESIGN	EM
REVIEW	EG
APPROVED	GL

PROJECT
Bay Street Bridge Rock Core Photos

TITLE
Rock Core Photos – BH-26-02

PROJECT No.	Phase / Task	Rev.	Figure
26-0649	104	0	F-4

APPENDIX G

NOTICE TO READER

NOTICE TO READER

This Report has been prepared by TULLOCH Engineering Inc. ('TULLOCH') for the sole and exclusive use of The Municipality of Magnetawan (the 'Client') to support design for the proposed Bay Street Bridge Replacement Project (the 'Development') located 435 m west of Sparks Street and Bay Street in the community of Magnetawan, Ontario (the 'Site'). The Report shall not be used for any other purpose, or provided to, relied upon or used by any third party without the express written consent of TULLOCH.

A limited number of boreholes were advanced at the Site; and as such, the information collected and presented herein applies to the borehole locations only. The subsurface conditions between boreholes can change and accordingly any use of the data contained in this Report should take into consideration the nature of the materials and potential variation between boreholes.

This Report contains opinions, conclusions and recommendations made by TULLOCH using professional judgment and reasonable care for the purpose of foundation design for the Development. Use of or reliance on this report by the Client is subject to the following conditions:

- a) the report being read in the context of and subject to the terms of the Engineering Services Agreement for the Work, including any methodologies, procedures, techniques, assumptions and other relevant terms or conditions specified or agreed therein;
- b) the report being read in its entirety. TULLOCH is not responsible for the use of portions of the report without reference to the entire report;
- c) the conditions of the site may change over time or may have already changed due to natural forces or human intervention, and TULLOCH takes no responsibility for the impact that such changes may have on the accuracy or validity of the observations, conclusions and recommendations set out in this report;
- d) the classification of soils and rocks in this report is based on commonly accepted methods. However, the classification of geologic materials and the boundaries between subsurface layers involves judgement. Boundaries between different soils layers may also be transitional rather than abrupt. TULLOCH does not warrant or guarantee the exactness of these descriptions and boundaries.
- e) the subsurface conditions must be verified by a qualified geotechnical engineer during construction to ensure that the borehole data presented herein is representative of the actual site conditions so that the design recommendations contained herein remain valid; and
- f) the report is based on information made available to TULLOCH by the Client or by certain third parties; and unless stated otherwise in the Agreement, TULLOCH has not verified the accuracy, completeness or validity of such information, makes no representation regarding its accuracy and hereby disclaims any liability in connection therewith.

This Report has been prepared with the degree of care, skill and diligence normally provided by engineers in the performance of comparable services for projects of similar nature. The scope of this report includes foundation engineering design only and it specifically excludes investigation, detection, prevention and assessment of the presence of subsurface contaminants. No conclusions or inferences should be drawn regarding contamination at the site including but not limited to molds, fungi, spores, bacteria, viruses, soil gases such as Radon, PCBs, petroleum hydrocarbons, inorganic and volatile organic compounds, polycyclic aromatic hydrocarbons and or any by products thereof.