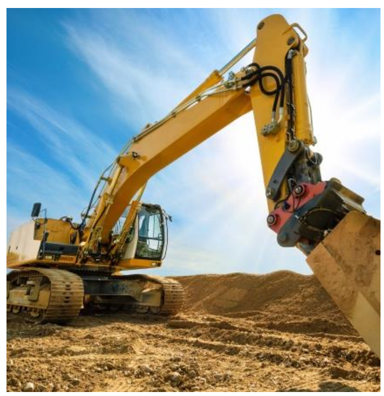
GEOTECHNICAL REPORT

McColls Bridge Replacement

Municipality of Highlands East, Ontario


June 2023

TULLOCH Project # 23-0840

LICENSED PROFESSIONAL ENGINEER
E. K. GILES
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JUN 23/2023
PROVINCE OF ONTARIO

June 23, 2023	0	Final	E. Mighton	E. Giles	F. Palmay
Date	**Rev.**	**Status**	**Prepared By**	**Checked By**	**Approved By**
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23-0840
June 23, 2023

Municipality of Highlands East
P.O. Box 295
2249 Loop Road
Wilberforce, Ontario
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Attention: Perry Kelly | Public Works Manager

RE: Geotechnical Report for the Replacement of McColls Road Bridge in the Municipality of Highlands East, Ontario

Dear Mr. Kelly,

Please find enclosed our Geotechnical Report for the proposed replacement of McColls Road Bridge located on McColls Road in the Municipality of Highlands East, Ontario.

This report outlines the results of the geotechnical and abutment coring investigation and provides geotechnical recommendations for the proposed replacement of the existing structure.

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

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1. INTRODUCTION AND SCOPE

TULLOCH Engineering Inc. (TULLOCH) was retained by The Municipality of Highlands East (Client) to complete a geotechnical site investigation for the proposed replacement of the bridge located on McColls Road in the Municipality of Highlands East, Ontario. The existing bridge is located approximately 550 m south of Highway 503. A site plan illustrating the borehole locations for the investigation can be found in Appendix A.

The purpose of the geotechnical investigation was to evaluate the subsurface conditions near the abutments of the existing bridge as well as to complete concrete core samples of the existing abutments. The report will provide commentary on the findings of the investigation, suitability of abutment re-use and provide foundation engineering recommendations for bridge replacement.

This report provides the factual geotechnical investigation data and geotechnical design recommendations, which are based on the site investigation data, our understanding of the project scope and engineering experience. Common terminology used in this report can be found in Appendix B and specific terminology is referenced in table notes or the report body.

2. REGIONAL GEOLOGY AND SITE

Based on our review of Bedrock Geology, Quaternary Geology and the Surficial Geology of Southern Ontario mapping as published by the Ontario Geological Survey (OGS) through online databases created by OGSEarth, the site's surficial geology is comprised of glaciofluvial deposits consisting of sands and gravels. The bedrock underlying the project site consists of two (2) major rock formations with metasedimentary rocks including marble, scarn and tectonic breccias of the Grenville Supergroup along the south shores of the Irondale River, and plutonic rocks including syenite, mafic and ultramafic bedrock on the north shores of the river. The site is relatively flat with the bridge spanning the Irondale River which was observed to have sandy banks and a sandy river bottom under the bridge.

The bridge itself consists of a steel truss bridge with a paved/concrete deck. The total length of the deck is approximately 18.6 m and the bridge is approximately 4.0 m wide. The steel truss is supported on concrete abutments. Significant corrosion has been observed on the steel and cracking has also been noted on the bridge deck during TULLOCH’s previous OSIM inspection conducted in 2022. Currently the bridge is load posted at 9 tons. A site photograph log of the bridge and site during the geotechnical investigation is provided in Appendix C.

3. SITE INVESTIGATION AND METHODOLOGY

The geotechnical investigation was completed from May 29th to May 30th, 2023. The investigation consisted of advancing two (2) boreholes referenced as BH-23-01 and BH-23-02 depth of approximately 14.0 m and 9.8 m below ground surface (mbgs) respectively. BH-23-01 and BH-23-02 were advanced to assess the subsurface soil conditions for foundation design. In addition to the geotechnical boreholes three (3) concrete cores were taken in the exposed abutment structures. Two (2) cores were advanced into the south abutment referenced as SA-01 and SA-02 and one (1) core was taken in the north abutment referenced as NA-01. Target core depth into the abutment was approximately 1.0 m. The cores taken to examine the condition of the existing concrete as well as to provide samples for Unconfined Compression Strength (UCS) testing.

For the purposes of this report, we have assumed that the section of McColls Road at the project site is oriented in a north-south direction. BH-23-01 and BH-23-02 were advanced in the north and south bound lane respectively with both holes advanced approximately 20 m from the abutment to allow for safe through traffic. Boreholes were advanced as close as possible to the bridge structure considering site logistics, maintaining the road without a detour, as well as the safety of the site crew including the locations of guiderails, road width accessibility and overhead and underground utilities. A summary of the investigation is shown below in Table 3-1. The borehole layout and coordinates can be viewed in the site plan in Appendix A. Borehole depths for this report are referenced from ground surface. Bedrock coring was not considered part of the scope of this investigation and was therefore not confirmed, DCPTs were advanced at both boreholes and refusal was noted on an inferred hard layer at the depths shown in the table below.

Table 0-1: Summary of Borehole Information

Abutment Location	Borehole No.	Northing (m)	Easting (m)	BH/DCPT Depth (mbgs)	Comments
North Abutment	BH-23-01	4976089	708672	14.02	Re-advanced after cobble/boulder refusal at approx. 1.5 mbgs
South Abutment	BH-23-02	4976034	708691	9.75	Re-advanced twice after cobble/boulder refusal at approx. 0.6 and 0.9 mbgs

Boreholes were advanced using a Diedrich D50Turbo track mounted drill rig owned and operated by Walker Drilling based in Utopia, Ontario. Both boreholes were advanced using 200 mm outside diameter (OD), continuous flight, hollow stem augers. BH23-02 was advanced using wash boring techniques past approximately 1.3 mbgs due to dense gravelly sands with cobbles and boulders.

The rig was equipped with standard soil sampling equipment including an automatic hammer. All depths for boreholes are referenced from ground surface. Detailed borehole logs are attached to this report in Appendix D

Concrete cores were drilled using a jig mounted Hilti coring drill which extracted continuous concrete cores approximately 100 mm in diameter which were examined photographed and logged by a TULLOCH technician. Photos of the retrieved concrete cores are shown in Appendix E.

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 the borehole and was conducted using an automatic hammer at 1.5 m intervals thereafter. In both boreholes, due to the presence of heaving sands causing auger and casing lock up, Dynamic Cone Penetration Testing (DCPT) was conducted down the boreholes to refusal.

The drilling and soil sampling programs were directed by a TULLOCH representative, who logged the drilling operations and identified the soil samples and concrete cores as they were retrieved. The recovered soil and rock samples were transported to TULLOCH's CCIL Certified Laboratory in Sault Ste Marie for detailed examination and testing.

4. LABORATORY TESTING PROGRAM

A geotechnical laboratory testing program was performed on representative samples at the bridge location in accordance with ASTM standards. Table 4-1 provides a list of the testing program. Detailed laboratory reports for grain size analysis, natural water content, unconfined compressive strength of the concrete and soil corrosivity can be found in Appendix F.

Table 0-1: Summary of Laboratory Testing Program

Test	Number of Tests	ASTM Standards
Washed Sieve Analysis	11	ASTM D422
Natural Water Content	11	ASTM D2216
Unconfined Compressive Strength (Concrete)	2	ASTM C39
Corrosivity Suite	2	Various

5. SUBSURFACE CONDITIONS

5.1 General

Subsurface conditions encountered at the McColls Bridge site are summarized below. Detailed borehole logs and laboratory testing summaries can be seen in Appendix D and F, 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. Soil layers encountered in the investigation are described below in the order they were encountered from ground surface.

5.2 Asphalt

A thin layer of asphalt was observed on the road surface at both borehole locations which were advanced through the lane of each road. The asphalt was found to be approximately 65 mm and 45 mm in thickness at BH23-01 and BH23-02 respectively.

5.3 Granular Road Base/ Subbase

Beneath the asphalt in all boreholes, an existing gravelly sand to sand fill was observed. The road base was observed to be approximately 690 mm thick in BH 23-01 but could not be differentiated from the road subbase in BH23-02. The thickness of the existing fill varied between approximately 1.5 m thick and 0.7 m thick between BH 23-01 and BH23-02 respectively. The existing fill material was noted to be well graded with trace non-plastic fines and was generally brown in colour and demonstrated non-cohesive behaviour. The material density of the fill was found to be compact with SPT ‘N’ values ranging between 19 and 25 blows per 30 cm of sampler advancement.

Grain size distribution testing of two (2) representative samples from this deposit were determined by laboratory sieve analysis which yielded the results shown below in Table 5-1. The material generally fell within the OPSS 1010 Granular B Type I gradation envelope with a small exceedance in fines content.

Table 0-1: Grain Size Distribution Summary – Sand

Borehole No.	Sample No.	Size Fraction (%)			
		Gravel	Sand	Silt	Clay
BH-23-01	SS1	21	68	11	
BH-23-01	SS2	23	67	10	

5.4 (SM) Silty Sand to Sand and Silt

IN BH 23-01 a native silty sand to sand and silt material was found beneath the existing fills. The material was found to contain medium grained sand and trace non-plastic fines. Varying degrees of silt and sand contents were observed across the retrieved samples. The deposit ranged in colour from dark grey to mottle grey/brown and generally displayed non-cohesive behaviour. Observed moisture conditions of the material from recovered split spoons varied from moist to wet with the material becoming noticeably wetter with depth. The deposit was approximately 7.6 m thick and the material density was found to range from loose to very loose with SPT ‘N’ values ranging from 9 blows per 30 cm of sampler advancement, to the sampler being advanced by the weight of hammer alone. Low SPT values were noted in heaving sands which may indicated the material had been disturbed by advancement of the borehole.

Laboratory testing results yielded natural water contents in this soil type ranging from approximately 19.4% to 38.0% and averaging 27.5%.

Grain size distribution testing of five (5) representative samples from this deposit were determined by laboratory sieve analysis which yielded the results shown below in Table 5-2.

Table 0-2: Grain Size Distribution Summary – Silty Sand to Sand and Silt

Borehole No.	Sample No.	Size Fraction (%)			
		Gravel	Sand	Silt	Clay
BH-23-01	SS3	3	19	78	
BH-23-01	SS4	0	45	56	
BH-23-01	SS6	5	81	14	
BH-23-01	SS7	0	56	44	
BH-23-01	SS8	0	64	35	

5.5 (SP) Sand

A thin layer of poorly graded sand was observed at BH23-01 on the north side of the bridge beneath the silty sand to sand and silt material at approximately 9.1 mbgs. The material extended to the depth of sampling at approximately 9.8 mbgs at which point a DCPT was advanced to refusal at an approximate depth of 14.0 mbgs. The poorly graded sand was observed to be medium grained in size, grey in colour and displayed non-cohesive behaviour. The material was observed to be wet and had a loose material density with an SPT ‘N’ value of 5 blows per 30 cm of sampler advancement.

5.6 (SW) Gravelly Sand to Sand

A well graded gravelly sand to sand was encountered beneath the fill and then again at approximately 6.2 mbgs at borehole BH23-02. The material contained fine to coarse grained sand particles, was gravelly to containing some gravel and trace non-plastic fines. Cobbles and boulders were also noted within the material deposit based on spoon interference and discontinuous auger advancement /grinding. The upper deposit was brown in colour whereas the lower deposit was grey. The deposit displayed non-cohesive behaviour and field moisture observations on retrieved split spoon samples indicated the material ranged from dry to wet with an increasing moisture content observed with depth. The upper deposit was approximately 2.3 m thick, and the lower deposit was approximately 1.4 m thick. The material density of the gravelly sand to sand was compact to very dense with SPT N values ranging from 19 to 27 blows per 30 cm of sampler advancement. Increased blows were noted in some samples but were likely caused by cobble/boulder interference and have therefore been discarded.

Laboratory testing results yielded natural water contents in this soil type ranging from approximately 2.6% to 12.3% and averaging 8.0%.

Grain size distribution testing of two (2) representative samples from this deposit were determined by laboratory sieve analysis which yielded the results shown below in Table 5-3.

Table 0-3: Grain Size Distribution Summary – Sand to Gravelly Sand

Borehole No.	Sample No.	Size Fraction (%)			
		Gravel	Sand	Silt	Clay
BH-23-02	SS2	27	67	6	
BH-23-02	SS4	21	71	8	

5.7 (SW/GW) Sandy Gravel to Sand and Gravel

A sand and gravel to sandy gravel was encountered between the sand to gravelly sand in BH23-02 between approximately 3.0 mbgs to 6.2 mbgs and then again at the bottom of the sampling interval at approximately 7.6 mbgs. The material deposit contained well graded sand and gravel particles as well as trace non-plastic fines. The material was grey in colour and field moisture observations on retrieved split spoon samples indicated the samples were wet with free standing water observed. The deposit displayed non-cohesive behaviour and had a material density of very dense to compact with SPT ‘N’ values ranging from 18 blows to over 50 blows per 30 cm of sampler advancement. High blow counts were noted within this deposit due to cobble and boulder obstructions.

Laboratory testing results yielded natural water contents in this soil type ranging from approximately 4.3% to 8.9% and averaging 6.6%.

Grain size distribution testing of two (2) representative samples from this deposit were determined by laboratory sieve analysis which yielded the results shown below in Table 5-4.

Table 0-4: Grain Size Distribution Summary – Sand to Gravelly Sand

Borehole No.	Sample No.	Size Fraction (%)			
		Gravel	Sand	Silt	Clay
BH-23-02	SS6	42	55	4	
BH-23-02	SS8	65	32	3	

Wash bore refusal was encountered within this layer at approximately 7.8 mbgs and a DCPT was advanced to refusal on an inferred hard layer at an approximate depth of 9.8 mbgs.

5.8 Bedrock

Bedrock was not confirmed within this investigation, refusal depths of the DCPT were noted in BH-23-01 and BH-23-02 at approximately 14.0 mbgs and 9.8 mbgs respectively. However, based on infield observations and blow counts the cone was not observed to be bouncing on a hard bedrock surface indicating a likely hard till layer or possibly weathered bedrock surface.

5.9 Concrete Abutments

A total of three (3) concrete cores were taken from the existing abutments with two (2) taken from the south abutment and one (1) taken from the north abutment. Generally, the cores were found to be intact and there was no indication of rehab within the abutments. Large boulders were frequently noted within the concrete of the abutments with particle sizes of over 300 mm and frequent large gravel particles exceeding 50 mm. The concrete itself appeared to be relatively intact with minimal signs of wear and/or weathering. One (1) viable sample was trimmed from the cores for each abutment and crushed to determine the compressive strength of the existing concrete. The results ranged from 11.3 MPa to 15.2 MPa. The results and implications of the concrete cores will be further discussed in the Recommendations section of this report in Section 6.0. Photograph logs of the recovered intact cores can be seen in Appendix E and the concrete UCS results can be seen in Appendix F.

8. CLOSURE

This geotechnical report has been prepared by TULLOCH for the exclusive use of The Municipality Highlands East. and their authorized agents for the replacement of the McColls Road Bridge. 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 I, Notice to Reader, which pertains to this report.

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

Erik Giles, P.Eng.
Geotechnical Engineer

Reviewed By:
Frank Palmay, P.Eng.
Project Manager

REFERENCES

Canadian Geotechnical Foundation Engineering Design Manual 4th Edition, 2006.

Canadian Highway Bridge Design Code (CHBDC 2014 CAN/CSA-S6-14), 2014.

Geotechnical Engineering Circular #3 Design Guidance: Geotechnical Earthquake Engineering for Highways, Volume I – Design Principles, 1997.

Ontario Geological Survey 2011. 1:250 000 Scale Bedrock Geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1.

Ontario Geological Survey 2010. 1:50 000 Scale Surficial Geology of Southern Onterio; Ontario Geological Survey, Miscellaneous Release---Data 128-Revised.

Occupational Health and Safety Act (OHSA), Ontario Regulation 213/9, Construction Projects, January 1, 2010, Part Ill - Excavations, Section 226.

National Building Code of Canada, NRC, 2015.

Pavement Design and Rehabilitation Manual, 2nd Edition, Chapter 3 Pavement Design, Ontario Ministry of Transportation, 2013

APPENDIX A

SITE LOCATION PLAN

BOREHOLE LOCATION PLAN

0 — 50
METERS

PROJECT LOCATION

COORDINATES		
NAME	EASTING	NORTHING
BH-23-01	708 672	4 976 089
BH-23-02	708 691	4 976 034

NOTES:

1. CO–ORDINATES ARE IN UTM ZONE 17 (NAD83 CSRS).

LEGEND:

BH–23–01 BOREHOLE LOCATION

No.	DATE	BY	ISSUES / REVISIONS
1	2023-06-23	EG	FINAL
0	2023-06-22	EG	ISSUED FOR REVIEW

TULLOCH

DRAWING: BOREHOLE LOCATION PLAN

PROJECT: **McCOLLS ROAD BRIDGE REPLACEMENT**

DRAWN BY: L. MENEGHETTI	CHECKED BY: J. MERCER	PROJECT No.: 23-0840
DESIGNED BY: L. MENEGHETTI	APPROVED BY: E. GILES	DRAWING No. 23-0840-001
SCALE: AS NOTED	DATE: 2023-06-22	REVISION No. 0

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APPENDIX B

TERMINOLOGY

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

BOREHOLES AND TEST PIT LOGS

Soils

AA	Auger Sample	w	Water Content
SS	Split Spoon	wP	Plastic Limit
TO	Tin-walled Tube	wL	Liquid Limit
TP	Thin-walled Piston	V(FV)	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
K	Slikensided	SR	Slightly Rough
BD	Bedding	R	Rough
FO	Foliation	VR	Very rough

IN SITU SOIL TESTING

Standard Penetration Test (SPT) "N" value. 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	
Clay	<0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.075 to 4.75 mm
Gravel	4.751o 75 mm
Cobbles	75 to 200 mm
Boulders	>200 mm

Terminology	Proportion
“trace”, sand, etc.	1%to 10%
"some"	10% to 20%
Sandy, Gravelly, etc.	20% to 35%
“and”	>35%
Ex., SAND, SILT, etc.	>35%

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 samples have been classified based on visual and tactile observations and is therefore an approximate description.

The following table outlines the qualitative terms used to describe the relative density condition of cohesionless soil:

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-Index:

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 to0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	< 0.075	< (200)

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 due to mechanical 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
1-5	Very low strength
5-25	Low strength
25-50	Medium strength
50-100	High strength
100-250	Very high strength
>250	Extremely high strength

Rock Mass Quality

RQD Classification	RQD Value (%)
Very Poor Quality	<25
Poor Quality	25 to 50
Fair Qualty	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 to discoloration on major discontinuity surfaces.
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity of surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than its fresh condition.
Moderatly Weathered	Less than half the rock material is decomposed and/or disintegrates to soil. Fresh or discolored rock is present either as a continuous frame work of as core stones.
Highly Weathered	More than half the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a discontinuous frame work 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 Plastric 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_R Peak shear strength
τ_R Residual shear strength
δ Angle of interface friction
μ Coefficient of friction = tan ϕ'

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' Effictive overburden pressure
OCR Overconsolidation ratio

APPENDIX C

PHOTOGRAPH LOG

Photo 1: BH-23-01 during advancement. Photo taken facing southeast.

Photo 2: BH-23-01 following completion of backfill. Photo taken facing southeast.

CLIENT
Municipality of Highlands East

PROJECT
McColls Bridge Replacement

CONSULTANT

TULLOCH

YYYY-MM-DD	2023-06-14
PREPARED	EM
DESIGNED	EM
REVIEWED	EG
APPROVED	EG

TITLE
Geotechnical Investigation BH-23-01 Site Photographs

PROJECT NO.	Phase/Task	Rev.	FIGURE
23-0840			1

Photo 3: BH-23-02 during advancement. Photo taken facing northwest.

Photo 4: BH-23-02 following completion of backfill. Photo taken face north.

CLIENT Municipality of Highlands East			PROJECT McColls Bridge Replacement	
CONSULTANT TULLOCH	YYYY-MM-DD	2023-06-14	TITLE **Geotechnical Investigation BH-23-02 Site Photographs**	
	PREPARED	EM		
	DESIGNED	EM		
	REVIEWED	EG	PROJECT NO. / Phase/Task / Rev. **23-0840**	FIGURE **2**
	APPROVED	EG		

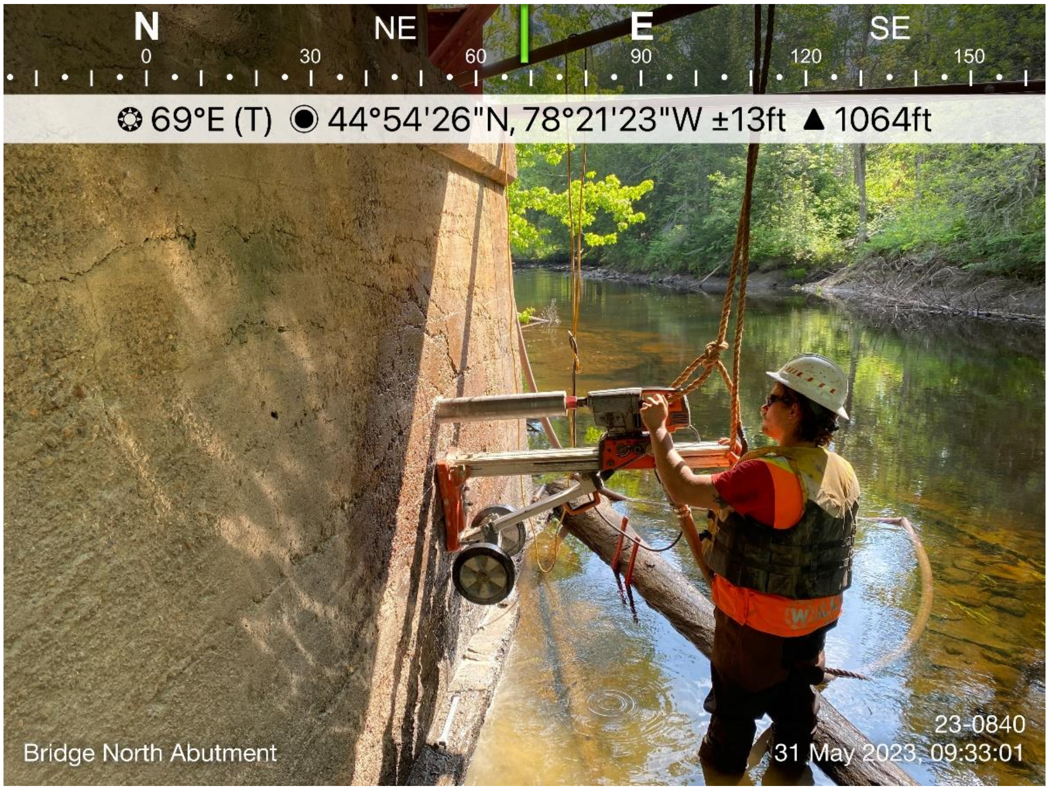

Photo 5: NA01 during advancement. Photo taken facing east.

Photo 6: SA01 and SA02 upon filling. Photo taken face south.

CLIENT	PROJECT
Municipality of Highlands East	McColls Bridge Replacement

CONSULTANT

TULLOCH

YYYY-MM-DD	2023-06-14
PREPARED	EM
DESIGNED	EM
REVIEWED	EG
APPROVED	EG

TITLE
Geotechnical Investigation Concrete Coring Photographs

PROJECT NO.	Phase/Task	Rev.	FIGURE
23-0840			3

Photo 7: View of bridge and river banks facing southwest

Photo 8: Typical view of river from bridge location. Photo taken facing southwest

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Municipality of Highlands East

PROJECT
McColls Bridge Replacement

CONSULTANT

TULLOCH

YYYY-MM-DD	2023-06-14
PREPARED	EM
DESIGNED	EM
REVIEWED	EG
APPROVED	EG

TITLE
Geotechnical Investigation Concrete Coring Photographs

PROJECT NO.	Phase/Task	Rev.	FIGURE
23-0840			**4**

Photo 9: Bridge south approach. Photo taken facing northwest.

Photo 10: Bridge north approach. Photo taken face south.

CLIENT
Municipality of Highlands East

PROJECT
McColls Bridge Replacement

CONSULTANT

TULLOCH

YYYY-MM-DD	2023-06-14
PREPARED	EM
DESIGNED	EM
REVIEWED	EG
APPROVED	EG

TITLE
Geotechnical Investigation Site Photographs

PROJECT NO.	Phase/Task	Rev.	FIGURE
23-0840			**5**

APPENDIX D

BOREHOLE LOGS

TULLOCH ENGINEERING

RECORD OF BOREHOLE No 23-01

METRIC

JOB NUMBER 23-0840 LOCATION McColl's Bridge North End, Gooderham ORIGINATED BY JM

CLIENT HIghlands East DATUM Geodetic BOREHOLE TYPE HSA/DCPT COMPILED BY EM

DRILLER Walker DATE 2023.05.29 NORTHING 4976089 EASTING 708672 CHECKED BY JM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	GR	SA	SI	CL	REMARKS
0.00	ASPHALT - 65 mm										
0.07	Fill - Gravelly SAND, fine to coarse grained sand, fine to coarse grained gravel, trace to some non-plastic fines, (Pavement Structure Base), brown, non-cohesive, moist, compact	SS1	SS	25	42		21	68	(11)		
0.76	FILL - SAND, fine to coarse grained, some fine to coarse grained gravel, trace to some non-plastic fines, (Pavement Structure SubBase), dark brown, non-cohesive, moist, compact	SS2	SS	19	63		23	67	(10)		Grinding during advancement from 0 m to 1.524 m
1.52	(SM) SILTY SAND to SAND and SILT, fine to medium grained sand, some to trace non-plastic fines, dark grey to mottled brown/grey, non-coheisve, moist to wet, very loose to loose	SS3	SS	9	67		3	19	(78)		Refusal at 1.47 m, moved 1.2 m north and re-drill from ground surface
		SS4a	SS	2	100		0	45	(56)		
		SS4b	SS	2	100						
		SS5	SS	2	67	▽					
		SS6	SS	3	63		5	81	(14)		
	- material increase in moisture content becoming wet	SS7a	SS	WH	100		0	56	(44)		
		SS7b	SS	WH	100						
		SS8	SS	7	92		0	64	(35)		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

DEPTH (M): 1 2 3 4 5 6 7 8

Continued Next Page

200 + : Numbers refer to Field Vane Over Limit $+^3, \times^3$: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

TULLOCH ENGINEERING

RECORD OF BOREHOLE No 23-01

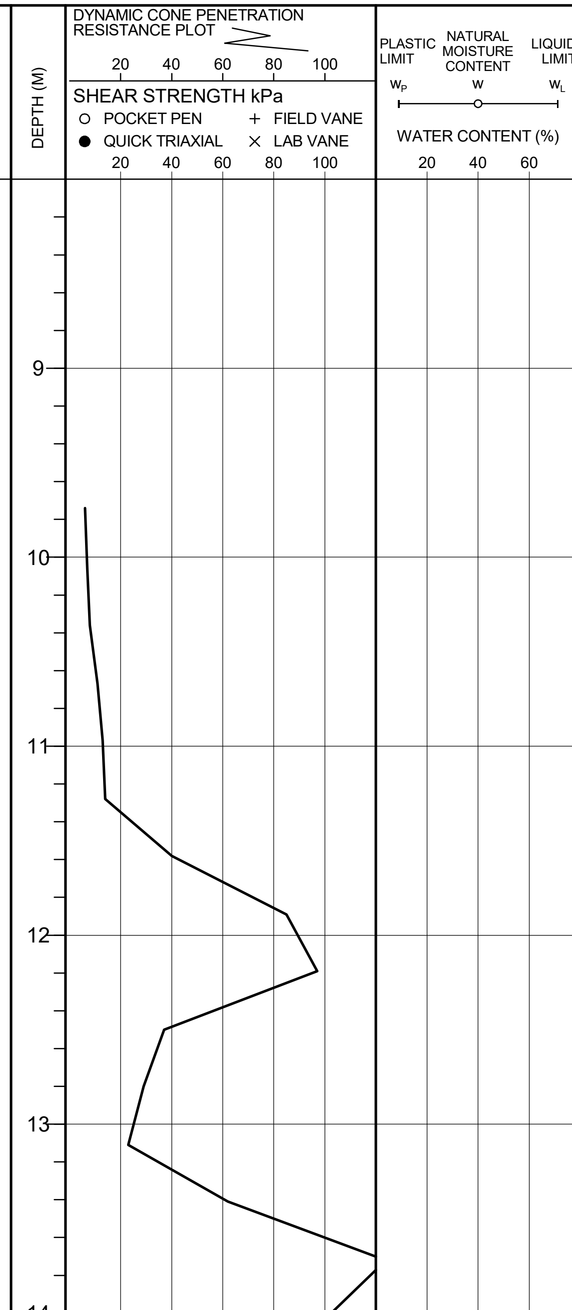
METRIC

JOB NUMBER 23-0840 LOCATION McColl's Bridge North End, Gooderham ORIGINATED BY JM

CLIENT HIghlands East DATUM Geodetic BOREHOLE TYPE HSA/DCPT COMPILED BY EM

DRILLER Walker DATE 2023.05.29 NORTHING 4976089 EASTING 708672 CHECKED BY JM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(SM) SILTY SAND to SAND and SILT, fine to medium grained sand, some to trace non-plastic fines, dark grey to mottled brown/grey, non-coheisve, moist to wet, very loose to loose *(continued)*							Coarser grained sand encontered at bottom of SS08 Heaving sand encountered at approx. 8.5 m
9.14	(SP) - SAND, medium grained, trace non-plastic fines, grey, non-cohesive, wet, loose		SS9	SS	5	75		
9.75								Switched to DCPT at approx. 9.75 m to coninue advancement
14.02	END OF BOREHOLE Note(s): 1. Groundwater measured at 3.33 mbgs upon compleiton of drilling. 2. Groundwater may not have stabalized upon completion of borehole. 3. Borehole cave-in to approx. 8.2 mbgs upon removal of casing. 4. It should be noted that the natural moisture contents may be higher due to water injected during advancement in heaving sandsections.							DCPT refusal



200 + : Numbers refer to Field Vane Over Limit $+^3, \times^3$: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

TULLOCH ENGINEERING

RECORD OF BOREHOLE No 23-02

METRIC

JOB NUMBER 23-0840 LOCATION McColl's Bridge South End, Gooderham ORIGINATED BY EM

CLIENT HIghlands East DATUM Geodetic BOREHOLE TYPE HSA/WB/DCPT COMPILED BY EM

DRILLER Walker DATE 2023.05.30 NORTHING 4976034 EASTING 708691 CHECKED BY JM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.00	ASPHALT - 45 mm						
0.05	FILL - SAND, fine to medium grained, trace non-plastic fines, (Pavement Structure Base/Subbase) trace gravel, brown, non-cohesive, dry, compact; trace to some coarse gravel, moist	SS01	SS	25	46		grinding from 0 m to 0.91 m; Auger refusal at approx. 0.6 m inferred boulder move rig and re advance
0.76	(SW) - Gravelly SAND, fine to coarse grained, trace non-plastic fines, presence of COBBLES and BOULDERS, brown, non-cohesive, dry to wet, compact	SS02	SS	21	38		27 67 (6); Auger refusal at approx 0.9m on inferred boulder move rig and re-advance 2nd time.
	becoming wet	SS03	SS	19	38		Auger refusal at approx. 1.32 m, switched to core barrel and wash boring to advance
	pocket of mica, very dense	SS04	SS	98	54	∇ (approx. 2.7 m)	21 71 (8); SS04 high blow count due to interference i.e. cobble/boulder
3.05	(SW/GW) - SAND and GRAVEL, fine to coarse grained sand, coarse to fine grained gravel, trace non-plastic fines, grey, non-cohesive, wet, very dense to compact	SS05	SS	57	46		SS05 high blow count due to interference i.e. cobble/boulder, no spoon recovery, spoon readvanced
		SS06	SS	18	30		42 55 (4)
		SS07a	SS	27	38		
6.20	(SW) - SAND, fine to coarse grained, some gravel, trace non-plastic fines, grey, non-cohesive, wet, compact	SS07b	SS	27	38		

DEPTH (M): 1, 2, 3, 4, 5, 6, 7

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

Continued Next Page

200 + : Numbers refer to Field Vane Over Limit $+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

TULLOCH ENGINEERING

RECORD OF BOREHOLE No 23-02

METRIC

JOB NUMBER 23-0840	LOCATION McColl's Bridge South End, Gooderham		ORIGINATED BY EM
CLIENT HIghlands East	DATUM Geodetic	BOREHOLE TYPE HSA/WB/DCPT	COMPILED BY EM
DRILLER Walker	DATE 2023.05.30	NORTHING 4976034 EASTING 708691	CHECKED BY JM

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	DEPTH (M)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(SW) - SAND, fine to coarse grained, some gravel, trace non-plastic fines, grey, non-cohesive, wet, compact *(continued)*								
7.62	(GW) - Sandy GRAVEL, trace non-plastic fines, fine to coarse grained gravel, fine to coarse grained sand grey, non-cohesive, wet, very dense							o	65 32 (3)
7.82		SS08	SS	157	21		8		Wash bore refusal, switch to DCPT to advance
							9		
9.75	END OF BOREHOLE Note(s): 1. Groundwater measured at 2.80 mbgs upon completion of borehole 2. Groundwater may not have stabalized upon completion of borehole. 3. Borehole cave-in to approx. 2.9 mbgs upon removal of casing. 4. It should be noted that the natural moisture contents may be higher due to water injected during advancement in heaving sections.								DCPT refusal

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH kPa (20 40 60 80 100): O POCKET PEN, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

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

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

APPENDIX E

CONCRETE CORE PHOTOS

Retrieved Concrete Core at Bridge South Abutment

SA-01

Core Length: 1.12 m

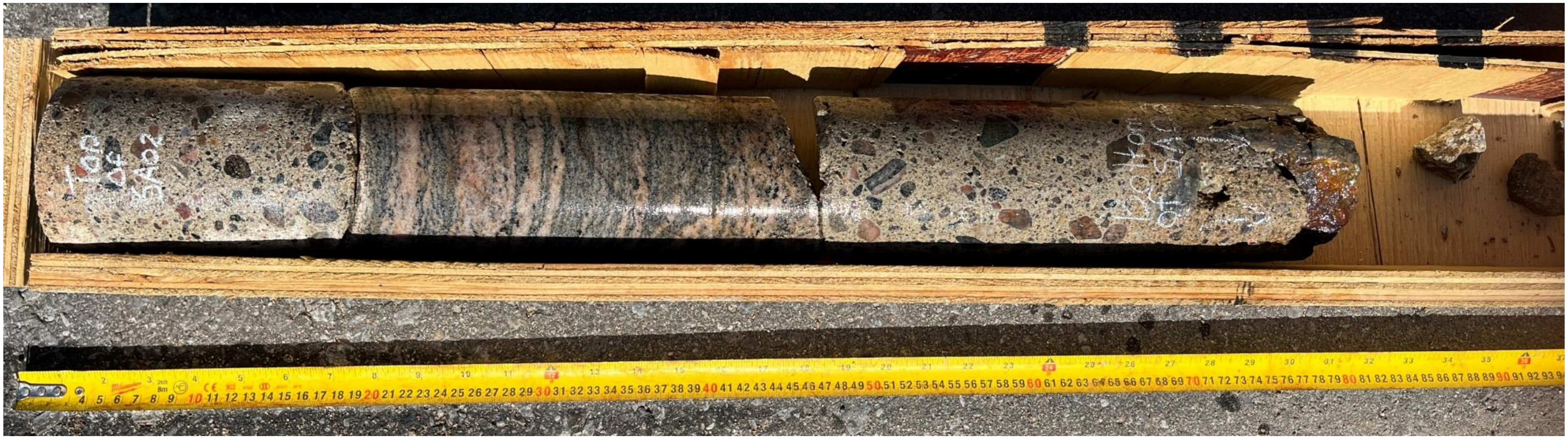
CLIENT	PROJECT
Municipality of Highlands East	McColl's Bridge Replacement

CONSULTANT		
TULLOCH	YYYY-MM-DD	2023-06-14
	PREPARED	EM
	DESIGN	EM
	REVIEW	EG
	APPROVED	EG

TITLE			
Concrete Core Photos – SA-01			
PROJECT No.	Phase / Task	Rev.	Figure
23-0840	100	0	A-1

Retrieved Concrete Core at Bridge South Abutment

SA-02



Core Length: 0.80 m

CLIENT		PROJECT
Municipality of Highlands East		McColl's Bridge Replacement

CONSULTANT		
TULLOCH	YYYY-MM-DD	2023-06-14
	PREPARED	EM
	DESIGN	EM
	REVIEW	EG
	APPROVED	EG

TITLE

Concrete Core Photos – SA-02

PROJECT No.	Phase / Task	Rev.	Figure
23-0840	**100**	**0**	**A-2**

Retrieved Concrete Core at Bridge North Abutment

NA-01

Core Length: 1.19 m

CLIENT	PROJECT
Municipality of Highlands East	McColl's Bridge Replacement

CONSULTANT		
TULLOCH	YYYY-MM-DD	2023-06-14
	PREPARED	EM
	DESIGN	EM
	REVIEW	EG
	APPROVED	EG

TITLE

Concrete Core Photos – NA-01

PROJECT No.	Phase / Task	Rev.	Figure
23-0840	100	0	A-3

APPENDIX F

LABORATORY RESULTS