



August 2017

DESKTOP FOUNDATION INVESTIGATION AND DESIGN REPORT

**Teeswater River Bridge Rehabilitation
Site 2-256, Highway 9
Contract 5B Structure Replacements and Rehabilitation
GWP 3025-14-00
Ministry of Transportation, Ontario - West Region**

Submitted to:

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REPORT



Report Number: 1534424-5002C-R01

Geocres No.: 41A-244

Distribution:

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PART A

FOUNDATION INVESTIGATION REPORT

TEESWATER RIVER BRIDGE REHABILITATION, SITE 2-256

HIGHWAY 9

CONTRACT 5B STRUCTURE REPLACEMENTS AND REHABILITATION

GWP 3025-14-00

MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION



1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a desktop review of the subsurface information available for the Teeswater River bridge on Highway 9 as part of the detail design work for GWP 3025-14-00. The project involves the detailed design of the replacement and rehabilitation of thirty-nine structures along multiple highways in Southwestern Ontario. This report addresses the proposed rehabilitation of the Teeswater River bridge (Site 2-256) on Highway 9 in the Geographic Township of Greenock in Bruce County, Ontario.

The purpose of this report is to provide recommendations based on a site reconnaissance together with the available geotechnical and geological data available from the MTO Geocres Library. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P1534424 dated September 2, 2015. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated October 26, 2015.

2.0 SITE DESCRIPTION

The subject site is located on Highway 9 approximately 140 metres east of Union Street in the community of Riversdale, in the Geographic Township of Greenock, Bruce County, Ontario. The approximate location of the bridge site is shown on the Key Plan, Figure 1.

This section of Highway 9 is currently a two lane, undivided highway with gravel shoulders. The bridge is oriented approximately east-west. The Teeswater River flows beneath the bridge generally from south to north. Highway 9 has a pavement surface elevation at about 277.5 metres at the bridge site. The existing precast concrete girder structure was constructed in 1969 as a replacement for the original structure. According to MTO's Structure Maintenance and Repair History Report¹, the bridge was rehabilitated in 1975, 1986, 2008 and 2012. The bridge has three spans with a total length of about 43 metres. The area immediately surrounding the site generally consists of flat-lying to gently rolling agricultural and forested lands to the east and the community of Riversdale to the west. Site photographs are provided in Appendix A.

3.0 INVESTIGATION PROCEDURES

The sources of information for this report included a site reconnaissance carried out at the site on February 14, 2017 together with a review of the available information from the MTO Geocres Library and the Ontario Geological Survey (OGS). No additional foundation investigations have been carried out for this assignment.

Geocres Report No. 41A-019 titled "Foundation Investigation Report For Proposed New Structure at Crossing Of Teeswater River and Highway #9, District #5 (Owen Sound), W.J. 67-F-19 – W.P. 101-64", dated April 18, 1967 was reviewed in conjunction with the preparation of this report. The Records of Boreholes and related laboratory test results are attached to this report in Appendix B. The Geocres report presented the results of the laboratory testing on the borehole logs and did not include separate figures illustrating the results.

¹ MTO, undated report: Structure Maintenance/Repair History (MRH Report) for Site Number 2-256, Highway 9 Teeswater River Bridge. This report contains the dates and summary details of rehabilitation works and inspections carried out at this site.



FOUNDATION INVESTIGATION AND DESIGN REPORT TEESWATER RIVER BRIDGE REHABILITATION - SITE 2-256

The borehole locations and ground surface elevations at the borehole locations are shown on the original Record of Borehole sheets in Appendix A and on Drawing 1, prepared by Golder. The table below summarizes the coordinates, ground surface elevations and depths of these previous boreholes.

Borehole	Location (m)		Ground Surface / Riverbed Elevation (m)	Depth of Water (m)	Borehole Depth# (m)
	Northing	Easting			
1	4 883 883	397 997	272.49	-	15.45
2*	4 883 870	398 002	273.01	-	4.27
3	4 883 884	398 013	270.45	3.20	10.67
4	4 883 879	398 050	273.50	-	15.24
5	4 883 888	398 031	271.27	1.89	14.84
6*	4 883 876	398 035	271.88	1.28	4.72
7*	4 883 891	398 046	273.01	-	5.15

*These boreholes contain records of dynamic cone penetration testing only.

#For boreholes 3, 5 and 6, the borehole depth includes the water column above the riverbed.

4.0 SITE GEOLOGY

The site lies within the Horseshoe Moraines physiographic region. This region is characterized by irregular, stony knobs and ridges which are composed mostly of till with some sand and gravel deposits as well as sand and gravel terraces and swampy valley floors². The quaternary geology mapping indicates that the surficial soils in the area of the site generally consist of modern alluvium containing silt, sand and gravel.³ Based on the available topographic bedrock mapping, the underlying bedrock surface is found at about elevation 256 metres⁴ corresponding to an overburden thickness of about 14 to 18 metres. The rock formation is mapped and described as limestone, dolostone and shale of the Detroit River Group of the Onondaga Formation of Middle Devonian age⁵.

5.0 SUBSURFACE CONDITIONS

The subsurface conditions encountered in the boreholes previously drilled at the site (Geocres Report No. 41A-019) are shown in detail on the Record of Borehole sheets in Appendix A. The following discussion has been simplified in terms of major soil or rock strata for the purposes of geotechnical design. The soil boundaries discussed in this report and illustrated on the Records of Boreholes have been inferred from non-continuous samples and observations of drilling resistance and represent a transition from one soil or rock type to another. The boundaries should not necessarily be interpreted to represent exact planes of geological change. Further, subsurface conditions should be expected to vary between and beyond the borehole locations. In addition, post-

² Chapman, L.J., and Putnam, D.F., 1984: Physiography of Southern Ontario; Ontario Geological Survey, Special Volume 2, 270p.

³ Cowan, W.R., and Pinch, J.J., 1986: Quaternary Geology of the Walkerton-Kincardine Area. Southern Ontario; Ontario Geological Survey, Map P.2956. Geological Series-Preliminary Map, scale 1:50 000. Geology 1975-1979.

⁴ Karrow, P.F. 1962, Bedrock Topography Series, Kincardine-Walkerton Sheet. Ontario Department of Mines, Preliminary Map P.165, Scale 1:50 000.

⁵ Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1.



investigation construction activities at the site may have altered the subsurface conditions from those shown on the previous Records of Boreholes.

The soil descriptions on the previous Records of Boreholes have been interpreted to correspond to standard MTO soil descriptions, using the original laboratory data, where available. Although the majority of the descriptions provided on the Record of Boreholes from Geocres No. 41A-019 have been retained, materials described as sandy gravel have been classified as sand and gravel or silty sand and gravel for the purposes of this report. The sandy silt layer at elevation 264.6 metres in borehole 5 has been classified as silty sand. The standard MTO soil descriptions are used in the discussion below and in the profile on Drawing 1. The subsurface conditions encountered in the boreholes generally consisted of organic silt, granular deposits (silt, sandy silt, silty sand, and sand and gravel), interlayered with clayey silt and silty clay, overlying bedrock. Based on the elevation of the boreholes and dynamic cone penetration hole locations reported in Geocres No. 41A-019, the elevation of the ground surface at the abutments ranged from 272.5 to 273.5 metres at the time of the investigation. The pre-construction riverbed elevation ranged between elevation 270.4 and 271.8 metres at boreholes 3, 5 and 6. The depth of the river at these three borehole locations ranged between 1.3 and 3.2 metres.

Embankment fills were constructed when the existing structure was built. Currently, the nature of this fill is unknown but the grade was raised by approximately 5 m and 3 m to approximate elevation 277.3 and 276.8 metres at the west and east abutments, respectively.

5.1 Site Stratigraphy

A layer of sandy silt was found at ground surface or formed the river substrate at boreholes 1, 3, 4 and 5. The thickness of the sandy silt stratum varied between 0.8 and 2.1 metres. The sandy silt was very loose to compact with standard penetration test (SPT) N values ranging from 4 to 24 blows per 0.3 metres. The sandy silt is of low plasticity based on a single sample with liquid and plastic limits of 39 and 36 per cent, respectively.

Organic silt was encountered at elevation 271.5 metres in borehole 4 below the sandy silt deposit. The organic silt layer was about 1.7 metres thick and had SPT N values of 4 blows per 0.3 metres. The organic silt had water contents of 77 and 95 per cent.

Layers of clayey silt and/or silty clay were encountered beneath the sandy silt in boreholes 1, 3 to 5 and beneath the organic silt in borehole 4. These cohesive soils were encountered between elevation 269.2 and 271.5 metres and were 1.8 to 3.7 metres thick. The very stiff to hard cohesive soils had SPT N values of 15 to 41 blows per 0.3 metres and water contents of about 13 to 18 per cent. The cohesive deposits are of low to intermediate plasticity based on liquid limits of 18 to 38 per cent and plastic limits of 13 to 20 per cent.

Granular deposits, ranging in gradation from silt to sand and gravel, were encountered in boreholes 1 and 3 to 5 beneath the clayey silt from elevation 266.2 and 269.7 metres. The thickness of the granular layers ranged from about 6.9 to 7.3 metres thick.

Borehole 3 was terminated in a sand and gravel layer after exploring it for about 3.1 metres. The granular deposits had SPT N values of 15 to over 100 blows per 0.3 metres. The sand and gravel deposits below approximate elevation 263.7 metres were very dense while the overlying silt to silty sand deposits were loose to dense. Boulders were noted in the sand and gravel deposits. Water contents ranged between 8 to 25 per cent with silt layers having water contents of 21 per cent and above. The silt deposits contain clay layers and are of low plasticity based on liquid limits of 25 and 27 per cent and plastic limits of 19 and 20 per cent.



Limestone bedrock was encountered beneath the sand and gravel layer in borehole 1 at elevation 260.8 metres or a depth of 11.7 metres. Borehole 1 was terminated in the bedrock after it was cored to a depth of 3.7 metres below the bedrock surface. The bedrock in borehole 1 was described as weathered from elevation 259.0 to 260.8 metres, and sound from elevation 257.0 to 259.0 metres. Boreholes 4 and 5 were terminated in inferred bedrock at elevations 259.0 and 259.5 metres after penetrating the rock for 0.8 and 1.2 metres, respectively.

5.2 Groundwater Conditions

Based on the information provided in the Geocres report, artesian groundwater conditions were encountered in boreholes 1, 3, 4 and 5. The encountered groundwater elevation was not indicated on the borehole logs or in the report except to indicate that the water rose 1.2 m in borehole 1 and the water rose only 0.3 m in the other three boreholes. Based on our review of the borehole logs the groundwater level cannot be confirmed.

Borehole 3 was drilled in the river and the reported river water level during drilling on March 30, 1967 was elevation 273.6 metres. Boreholes 5 (drilled March 28 and 29, 1967) and 6 (drilled March 23, 1967) were advanced in the river floodplain and reported a river water level and an ice level above the ground surface, respectively, both corresponding to elevation 273.2 metres. A river water level of 271.8 metres in January 1967 was also reported in the Geocres report. The preliminary General Arrangement drawing provided by Stantec indicated a river water level of 272.4 metres in April 2016.

The reported water levels are not necessarily considered to be representative of the long-term, stabilized groundwater conditions. The groundwater levels should be expected to fluctuate seasonally and be higher during periods of sustained precipitation or during spring snow melt conditions.

6.0 SITE RECONNAISSANCE

A site reconnaissance was carried out at the site on February 14, 2017 by a member of our geotechnical engineering staff. In general, the approach embankments and abutment foreslopes appeared to be stable with no visible signs of movement. The foreslopes were covered with rip rap largely consisting of concrete pieces. The abutments appeared to be performing adequately with no observable movement(s) noted, although the east pile cap is partially exposed. Some concrete spalling and exposed rebar was noted at the abutment locations. The piers were in generally good condition although there appeared to be some exposed reinforcing steel in one of the columns at the east pier. Areas of delaminations and spalling were noted at the ends of some of the concrete girders. The bridge soffit is in generally good condition although some concrete spalling and exposed rebar was noted. The asphalt riding surface was in fair condition at the approaches, with some minor to moderate cracking, and in poor condition in the area of the bridge deck with more severe cracking. Photographs from the site reconnaissance are shown in Appendix B.



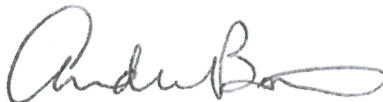
7.0 MISCELLANEOUS

The site reconnaissance and preparation of the report was carried out by Mr. Daniel Hyland, E.I.T. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. André Bom, P.Eng., an Associate with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.


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**FOUNDATION INVESTIGATION AND DESIGN REPORT
TEESWATER RIVER BRIDGE REHABILITATION - SITE 2-256**

PART B

FOUNDATION DESIGN REPORT

TEESWATER RIVER BRIDGE REHABILITATION, SITE 2-256

HIGHWAY 9

CONTRACT 5B STRUCTURE REPLACEMENTS AND REHABILITATION

GWP 3025-14-00

MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION



8.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the foundation aspects of the design of the proposed bridge rehabilitation at Site 2-256 on Highway 9 in the Geographic Township of Greenock in Bruce County, Ontario.

The recommendations are based on our interpretation of the available factual data and are intended to provide the designers with sufficient information to design the bridge rehabilitation. As such, where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, and scheduling.

The existing precast concrete girder structure was constructed in 1969 as a replacement for the original structure. According to the Structure Maintenance and Repair History, the bridge was rehabilitated in 1975, 1986, 2008 and 2012. The bridge has three spans with a total length of about 43 metres. The Department of Highways Ontario (DHO) design drawings for Contract No. 68-145 dated December 1967 identify that the piers are supported on 12 BP 74 H-piles (HP 310 x 110) and the abutments are supported on 12 BP 53 H-piles (HP 310 x 79). The design drawings also indicate that there are seven piles at each pier (one for each column) and eight piles at each abutment. The pier columns consist of 0.6 m diameter concrete encased sections. The design cut off elevations are 275.2 metres at the east pier, 275.4 metres at the west pier and 273.6 metres at the abutments. Geocres Report No. 41A-019 indicated that the piles could either be driven into the very dense sand and gravel or driven to bedrock. The design loads were 70 tons (623 kN) per pile at the abutments and 95 tons (845 kN) per pile at the piers using working stress design.

The 1967 General Drawing and Geocres Report No. 41A-019 specify that the 1.7 metre thick layer of organic silt encountered in borehole 4 was to be removed at the east approach. The area was to be sub-excavated to elevation 269.1 metres or lower, inferred to be sloping down towards the river. The sub-excavated area was to be backfilled with 'suitable granular fill' prior to construction of the east abutment.

The boreholes advanced for Geocres Report No. 41A-019 pre-dated construction of the approach embankments and existing structure. Therefore the type of fill material used to construct the approach embankments and its consistency/density is unknown.

Based on the information provided to Golder from Stantec the proposed rehabilitation work will consist of replacement of the deck and girders, conversion to semi-integral abutments, new parapet walls and other repairs. There will be an increase in axial loading at the abutments by about 100 kilonewtons (kN) and a slight decrease in loading at the piers. We understand from Stantec that the change in lateral loading of the piles due to the proposed rehabilitation is expected to be negligible, and that seismic input (from a geotechnical perspective) is not required.

We understand that a portion of the existing approach embankments will be sub-excavated for conversion of the existing abutments to semi-integral abutments. We further understand that the reconstructed approach embankments will be consistent with the existing embankment (i.e. neither grade raise nor widening) and that the new side slopes will not be steeper than 2 Horizontal to 1 Vertical (2H:1V). If a grade raise or widening is being considered at this site, the geotechnical engineer should be allowed to review and provide additional input.



8.1 Foundations

8.1.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC version S6-14) and its Commentary, the proposed bridge and its foundation system is considered to be classified as having a "typical consequence level" associated with exceeding limits states design. This consequence classification should be confirmed by Stantec and the MTO.

The degree of understanding, based on the scope of the current foundation desktop study and proximity of the previously advanced boreholes to the structure, is considered 'typical' as described in Clause 6.5.3.2 of the 2014 CHBDC. The appropriate corresponding Ultimate Limit States (ULS) and Serviceability Limit States (SLS) consequence factors, Ψ , geotechnical resistance factors at ULS (ϕ_{gu}) and SLS (ϕ_{gs}), respectively, from Tables 6.1 and 6.2 of the CHBDC should be used for design.

8.1.2 Geotechnical Axial Resistance – Driven Piles

Based on Golder's review of the design drawings and subsurface conditions, although the existing steel H-piles were specified to be driven to bedrock, it is quite probable that the H-piles may have experienced refusal in the very dense sand and gravel or may have terminated in this deposit if the design loads were achieved. No as-built records or pile driving records were provided for our review and these records may not be available. The following table summarizes the available factored geotechnical resistances at Ultimate Limit States (ULS) for piles driven to bedrock as well as terminating within the sand and gravel and geotechnical resistance values at Serviceability Limit States (SLS) for piles terminating within the sand and gravel. The SLS value for the sand and gravel is applicable for 25 millimetres of settlement. Since the bedrock is considered to be effectively unyielding an SLS value is not provided.



FOUNDATION INVESTIGATION AND DESIGN REPORT TEESWATER RIVER BRIDGE REHABILITATION - SITE 2-256

Pile Location	Pile Type	Cut-off Elevation (m)	Assumed Founding Strata	Assumed Founding Elevation Bedrock / Sand and Gravel (m)	Factored Geotechnical Resistance at ULS Bedrock / Sand and Gravel (kN)	Geotechnical Reaction at SLS Bedrock ¹ / Sand and Gravel (kN)
West Abutment	HP 310 x 79	273.6	Bedrock / Sand and Gravel	260.8 / 262.7	1,200 / 1,000	- / 925
East Abutment				259.5 / 262.1	1,200 / 1,000	- / 925
West Pier	HP 310 x 110	275.4		259.5 / 263.0	1,800 / 1,400	- / 925
East Pier		275.2		259.5 / 259.74	1,800 / 1,400	- / 925

Note: ¹ For piles founded on bedrock, the SLS reaction for 25 mm of settlement is greater than the ULS resistance and therefore ULS governs.

Based on Stantec's increase in loading of 100 kN at each abutment, the new load on each of the eight piles will be increased by about 13 kN, corresponding to a load of approximately 636 kN per pile based on the information available in the existing design drawings. We understand the loads at the piers will decrease slightly and will therefore be slightly less than 845 kN per pile. The existing foundations at the abutments and piers are considered to be adequate to support the nominal changes in loading.

8.2 Excavations and Backfill

Excavations for repairs to the concrete in the wingwalls and abutments and conversion of the conventional abutments to semi-integral abutments are anticipated to terminate within the embankment fill near elevation 276 metres. There was insufficient information in Geocres Report No. 41A-019 to ascertain the range of elevations corresponding to the reported artesian groundwater levels. Artesian conditions are not expected to influence construction since excavations will not penetrate the native deposits. The groundwater level has been inferred to correspond with the water level in the Teeswater River. This was reported to range from elevation 271.8 to 273.6 metres and will be at least 2 metres below the base of the proposed excavations. The nature of the embankment fill is unknown as Geocres Report No. 41A-019 predated construction of the approach embankments. However if the granular roadbase is underlain by less permeable material, or there are layers of lower permeability materials surrounding granular zones within the fill, infiltrated water may be perched or accumulated in these areas particularly after periods of heavy precipitation. Any seepage may be adequately controlled using properly



constructed and filtered sumps. The work area should be graded to prevent ingress of surface runoff into the excavations. Temporary open cut slopes within the fill materials should be maintained no steeper than 1 horizontal to 1 vertical. All excavations should be carried out in accordance with the current edition of the Occupational Health and Safety Act. The fill material above the water level is considered to be a Type 3 soil.

Excavation and backfilling of the abutments should be carried out in accordance with Ontario Provincial Standard Specification (OPSS) 902 and Ontario Provincial Standard Drawing (OPSD) 3101.150. Select, free-draining granular fill meeting the specifications of OPSS.PROV 1010 Granular A or Granular B Type II or III should be used as backfill behind the abutments and wingwalls. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading). The fill should be compacted in loose lifts not greater than 300 millimetres in thickness. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3190.100.

8.3 Temporary Roadway Protection

It is understood that temporary roadway protection is required should a single lane of traffic need to be maintained on Highway 9 at the bridge location during construction. Temporary support systems could consist of cantilevered soldier piles and lagging or steel sheet piles. Installation of steel sheets into the sand and gravel deposits may not be feasible due to the relative density of the layer and the presence of cobbles and boulders.

Excavation support systems should be designed and constructed in accordance with OPSS 539 and the design should limit the lateral movement of the temporary shoring system to meet Performance Level 2. The contractor is responsible for the complete detailed design of the protection system.

Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line, or point loads as well as the impact of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter.

The support systems may be designed using the parameters provided in the table below.

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Bulk Unit Weight γ (kN/m ³)
	Active, K_a	At Rest, K_o	Passive, K_p		
Fill (assumed to be compact granular fill)	0.36	0.53	2.8	28	19
Clayey Silt-Silty Clay	0.33	0.50	3.0	30	19



FOUNDATION INVESTIGATION AND DESIGN REPORT TEESWATER RIVER BRIDGE REHABILITATION - SITE 2-256

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Bulk Unit Weight γ (kN/m ³)
	Active, K_a	At Rest, K_o	Passive, K_p		
Silt	0.33	0.50	3.0	30	19
Sandy Silt to Silty Sand	0.32	0.49	3.1	31	19
Sand and Gravel	0.27	0.43	3.7	35	22

These parameters are provided to assist with design for the unfactored ultimate resistance and loading conditions and may not result in a temporary support design that adequately controls ground and structure displacements. Achieving adequate displacement control in accordance with the MTO performance criteria may require designs that result in a system that is stiffer than might otherwise be required based on the soil parameters provided in the table above. The earth pressure coefficients identified above may be applied assuming a horizontal ground surface behind the retaining structure. Where the ground surface behind the retaining structure is sloped, the earth pressure coefficients provided in the table above must be increased accordingly.



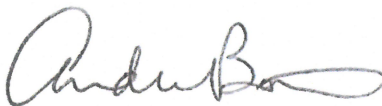
9.0 MISCELLANEOUS


This report was prepared by Mr. Daniel Hyland, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. André Bom, P.Eng., an Associate with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.


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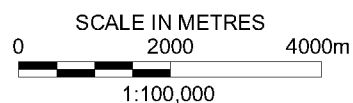
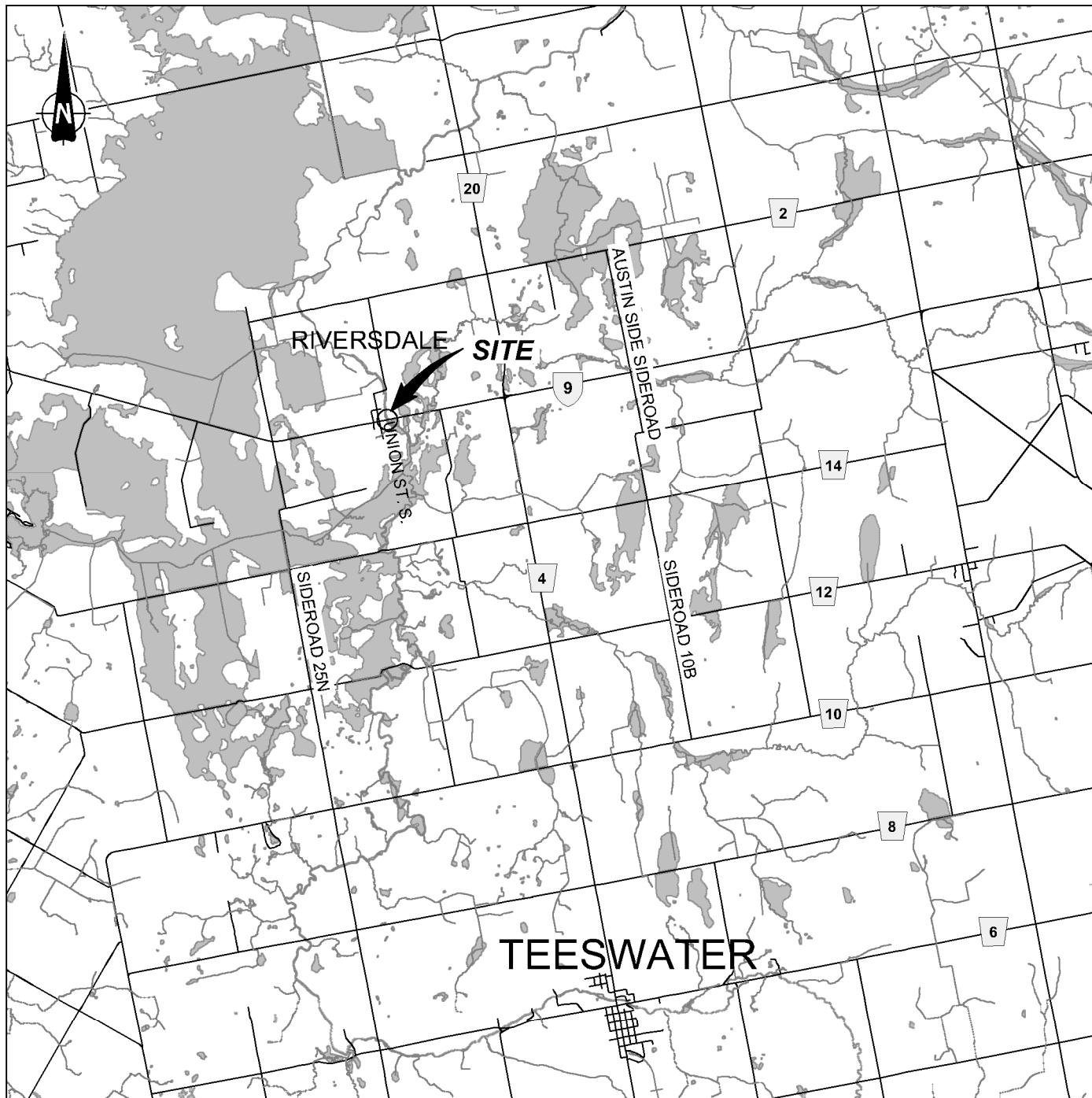

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REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.4.

NOTES

THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING TEXT.
ALL LOCATIONS ARE APPROXIMATE ONLY.

PROJECT

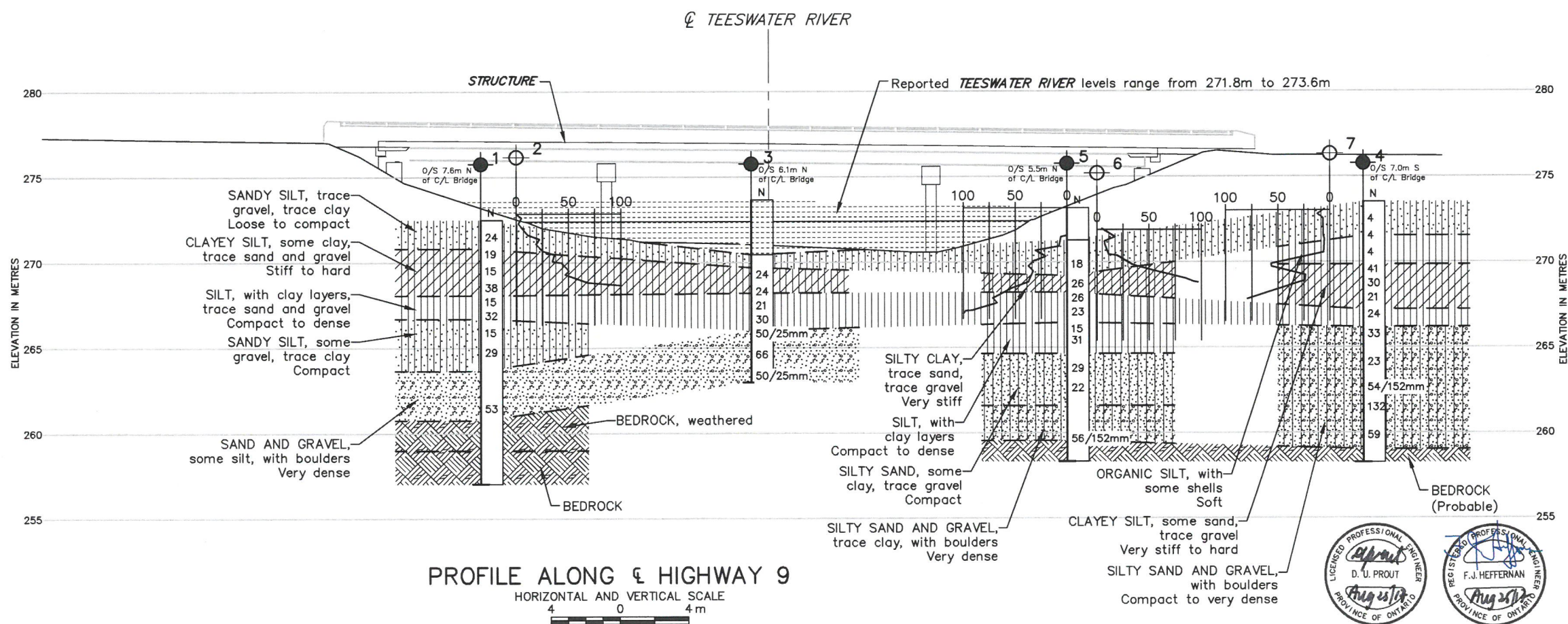
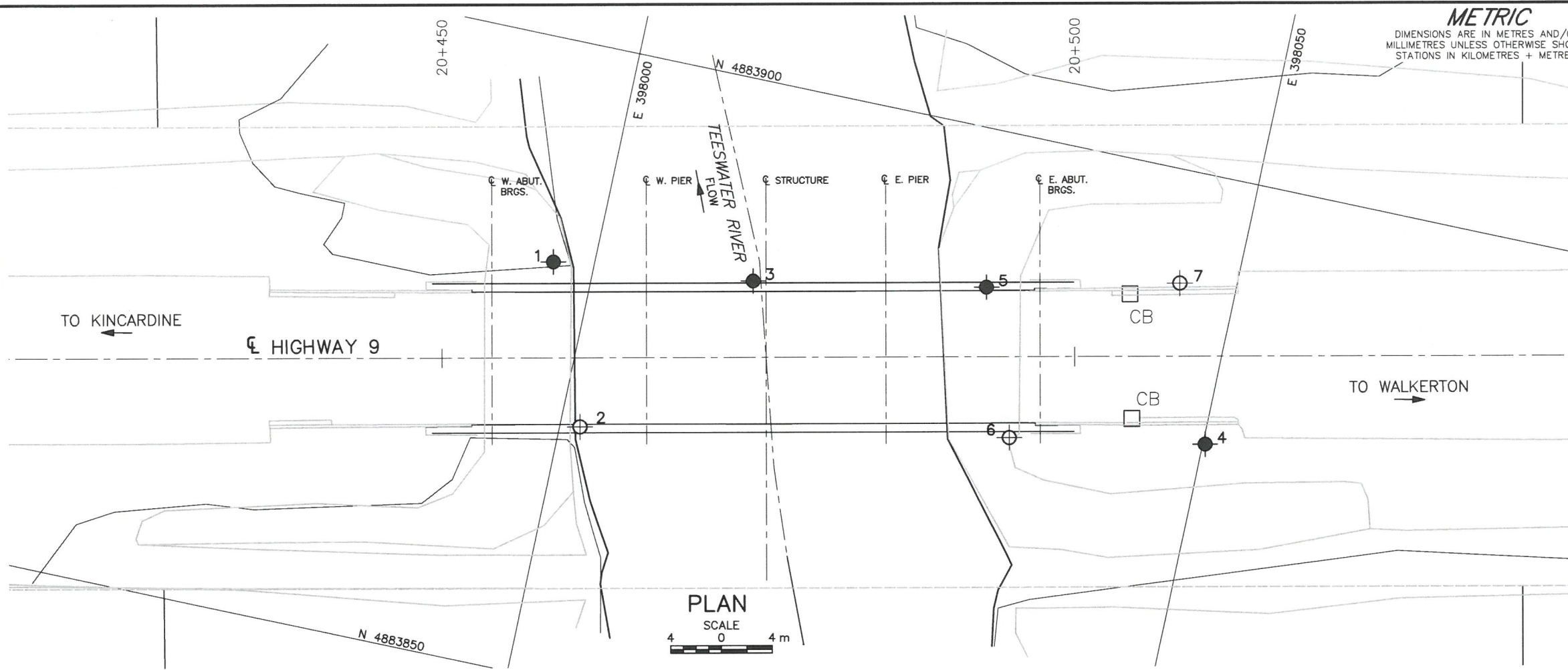
TEESWATER RIVER BRIDGE REHABILITATION, SITE 2-256
HIGHWAY 9
GWP 3025-14-00

TITLE

KEY PLAN



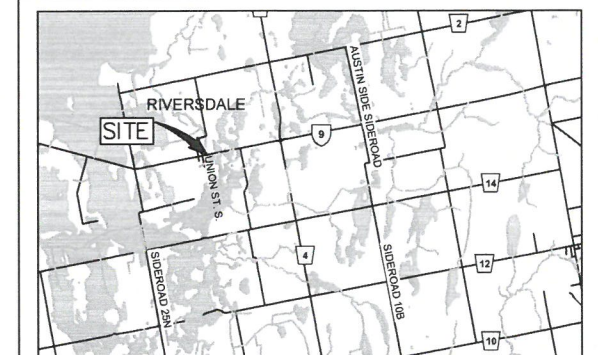
PROJECT No. 1534424			FILE No. 1534424-5002C-F01001	
CADD	LMK	May 30/17	SCALE AS SHOWN	REV. 0
CHECK			FIGURE 1	

CONT No.
WP No. 3025-14-00

TEESWATER RIVER BRIDGE

STRUCTURE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

**Golder Associates Ltd.**
LONDON, ONTARIO, CANADA

LEGEND

- Borehole (from Geocres Report 41A-019)
- Cone Penetration (from Geocres Report 41A-019)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Dynamic Cone Penetration Test

No.	ELEVATION	CO-ORDINATES (MTM NAD83 ZONE 11)	
		NORTHING	EASTING
GEOCRES NO. 41A-019			
1	272.49	4 883 882.5	397 996.8
2	272.86	4 883 870.1	398 001.6
3	273.65	4 883 884.3	398 012.5
4	273.50	4 883 879.1	398 050.2
5	273.16	4 883 887.6	398 030.7
6	271.88	4 883 876.3	398 034.9
7	273.01	4 883 891.1	398 045.6

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

Base plans provided by Stantec.

NO.	DATE	BY	REVISION
Geocres No. 41A-244			
HWY.	9	PROJECT NO. 1534424	DIST.
SUBM'D. DH	CHKD. DH	DATE: Aug. 21/17	SITE: 2-256
DRAWN: LMK	CHKD. DUP	APPD. FJH	DWG. 1



APPENDIX A

Site Photographs



APPENDIX A

Site Photographs



Photograph 1: East end of bridge, looking west.



Photograph 2: East end of bridge, looking west.



APPENDIX A

Site Photographs



Photograph 3: South elevation.



Photograph 4: Embankment sideslope in northeast quadrant.



APPENDIX A

Site Photographs



Photograph 5: West abutment, looking north.



Photograph 6: Looking east under bridge.



APPENDIX A

Site Photographs



Photograph 7: East abutment, looking north.



Photograph 8: Partially exposed pile cap at east abutment.

n:\active\2015\3 proj\1534424 stantec_mega 5 3015-e-0008_swo\ph 5000 cont 5 gwp 3022-14-00\ph 5002 fdns\2-cor\5-rpts\r01-ph 5002c site 2-256\1534424-5002c-r01 aug 22 17 (final)
app a photos.docx



APPENDIX B

Records of Boreholes - Geocres Report No. 41A-019

FOUNDATION SECTION

JOB 67-F-19 LOCATION Sta. 66 + 42 @ Hwy. #9 and 24' o/s to Rt. ORIGINATED BY AMS
W. P. 101-64 BORING DATE March 14, 15, 16, 17 & 20, 1967 COMPILED BY AMS
DATUM Geodetic BOREHOLE TYPE NX, BX & AX Casing, AXT, R.C. CHECKED BY LL

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 67-F-19 LOCATION Sta. 65 / 91 & Hwy. #9 and 20' o/s to Rt. ORIGINATED BY AMS
W. P. 101-64 BORING DATE March 30, 1967 COMPILED BY AMS
DATUM Geodetic BOREHOLE TYPE NX Casing and Washbore CHECKED BY [Signature]

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOB 67-F-19

LOCATION Sta. 64 + 73 @ Hwy. #9 and 20' o/s to Lt.

ORIGINATED BY AMS

W.P. 101-64

BORING DATE March 21, 22 and 23, 1967

COMPILED BY AMS

DATUM Geodetic

BOREHOLE TYPE NX, BX & AX Casing, BXL & AXT. R.C.

CHECKED BY AK

[illegible]

FOUNDATION SECTION

DATUM Geodetic BOREHOLE TYPE NX & EX Casing, Washbore, EXL R.C. CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	20	40	60	80	100	WATER CONTENT % 15 30 45			
896.2	WATER LEVEL														
0.0	273.16m														
	271.27m (1.89m)														
890.0	GROUND LEVEL					890									
6.2	Sandy silt with traces of clay and gravel. Compact.														
883.2	269.20m. (3.96m)		1	SS	18										Gr. 3, Sa. 30 Si. 59, Cl. 8
13.0	Clayey silt trace of sand and gravel.		2	SS	26	880									
	Very stiff		3	SS	26										Gr. 0, Sa. 9 Si. 77, Cl. 14
874.2	266.46m 6.71m)		4	SS	23										
22.0	Silt with occasional layers of clay		5	SS	15										
868.2	Compact to dense.		6	SS	31	870									
28.0	Sandy silt with some clay and trace of gravel.		7	SS	29										
	Compact.		8	SS	22	860									Gr. 9, Sa. 41 Si. 40, Cl. 10
858.2	261.58m (11.58m)														
38.0	Sandy gravel, some silt, occ. boulders. Very Dense														
851.5	259.54m. (13.62m)		11	SS	56/61	850									Gr. 26, Sa. 39 Si. 29, Cl. 6
44.7	Probable Bedrock		12	BXL RC	Rec 50%										
817.5	258.32m. (14.84m)														
48.7	End of Borehole														

JOB 67-F-19 LOCATION Sta. 65 + 24 @ Hwy. #9 and 19' o/s to Lt. ORIGINATED BY AMS
W.P. 101-64 BORING DATE March 23, 1967 COMPILED BY AMS
DATUM Geodetic BOREHOLE TYPE Dynamic Cone Penetration Test CHECKED BY AK

[illegible]

FOUNDATION SECTION

[illegible]

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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