FOUNDATION INVESTIGATION AND DESIGN REPORT

Whitefish River Bridge, Site No. 46X-0201/B0
Highway 7041, Mongowin Township, District of Sudbury
Ministry of Transportation, Ontario
GWP 5184-14-00, WP 5182-17-01

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PART A

FOUNDATION INVESTIGATION REPORT
WHITEFISH RIVER BRIDGE, SITE NO. 46X-0201/B0
HIGHWAY 7041, MONGOWIN TOWNSHIP, DISTRICT OF SUDBURY
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5184-14-00, WP 5182-17-01

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by D.M. Wills Associates Ltd. (D.M. Wills) on behalf of Ministry of Transportation, Ontario (MTO) to provide detail design Foundation Engineering services for the replacement of the Whitefish River Bridge (Site 46X-0201/B0). The Whitefish River Bridge is located on Highway 7041 at approximately Station 10+935 Mongowin Township, in the District of Sudbury, Ontario (i.e., about 0.4 km north of the Highway 6 south junction). The Key Plan of the general location of this section of Highway 7041 and the location of the investigated area are shown in Drawing 1.

The purpose of this subsurface exploration is to establish the subsurface conditions at the Whitefish River Bridge site and approach embankments adjacent to the bridge by borehole drilling and rock coring with laboratory testing carried out on selected soil and rock core samples.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated May 2018 and subsequent discussions with MTO and D.M. Wills. Golder's proposal dated August 13, 2018, for Foundation Engineering services associated with the replacement of this structure is contained in Section 7.7 of D.M. Wills' technical proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for Foundation Engineering services for this project, dated November 19, 2018.

2.0 SITE DESCRIPTION

It should be noted that the orientation (i.e., north, south, east, west) stated in the text of the report is referenced to project north and therefore may differ from magnetic north shown in Drawing 1. For the purpose of this report, Highway 7041 is oriented in a north-south direction with the Whitefish River flowing in an east-west direction at the bridge site.

The existing Whitefish River Bridge consists of a single-lane, single-span, steel truss permanent Modular Bridge (MB), which is approximately 4.2 m wide by 33.5 m long. The existing MB was reportedly installed in 1994 on existing cast-in-place concrete abutments/footings, that were originally constructed in 1929 and subsequently repaired in 1984. We also understand that the Highway 7041 was last rehabilitated under Contract 2017 and the rehabilitation work consisted of in-place full depth reclamation and subsequent paving with with 50 mm of hot-mix asphalt.

The bridge deck is at an approximate Elevation 181.9 m at both the north and south abutments. The existing approach embankments are approximately between 4 m and 5 m high and the embankment side slopes are inclined at about 2 Horizontal to 1 Vertical (2H:1V) or slightly steeper beyond the walls of the abutments.
In general, the topography of the site and surrounding area is comprised of gently rolling/undulating terrain with visible bedrock outcrops separated by vegetated zones of grass and small shrubs beyond the highway right-of-way. The land use in the area of the bridge is generally residential with some commercial properties nearby and a marina located about 300 m downstream (i.e., west) of the bridge. Ground surface conditions at the bridge location are shown in Photographs 1 to 6.

As outlined in the Ontario Structure Inspection Manual (OSIM) report dated November 30, 2016, the embankments were noted to be in good condition with no performance deficiencies at the time of the inspection. Based on our site observations and a review of the available site photographs, the existing approach embankments at the Whitefish River Bridge generally appear to be performing satisfactorily with no evidence of instability or settlement (i.e., soil movement). Although a buried layer of asphalt was encountered in the boreholes drilled at the abutments, given that the elevation of the top of this layer is at a relatively consistent elevation, it is believed that this is part of the former pavement structure prior to the 1984 bridge replacement.

As further noted in the 2016 OSIM, the existing Whitefish River Bridge was generally in moderate condition at the time of the inspection; however, the existing cast-in-place concrete abutments walls and wingwalls were noted to be severely corroded with extensive severe scaling, map cracking, and delaminations.

3.0 INVESTIGATION PROCEDURES

Field work for this subsurface exploration was carried out between June 24 and June 27, 2019, during which time eight boreholes (Boreholes WR-1 to WR-8) were advanced at the bridge abutments and approaches at the approximate locations shown in Drawing 1.

The boreholes were advanced from the existing roadway platform using a track-mounted drilling rig supplied and operated by Landcore Drilling of Chelmsford, Ontario. The boreholes were advanced using 150 mm outside diameter solid-stem augers, NW casing with wash boring, and NQ coring techniques (as required). Water utilized for NW casing and NQ coring advancement was obtained from the Whitefish River. Traffic control was performed by Golder in accordance with Ontario Traffic Control Manual Book 7 – Temporary Conditions. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic hammer, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)[1]. Field vane shear tests were carried out in the cohesive soils for assessment of undrained shear strengths in general accordance with Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils (ASTM D2573)[2] using MTO 'N'-size vanes. The groundwater level in the open boreholes was observed during the drilling operations. The boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended) and the existing asphalt on the roadway surface at the borehole locations was repaired using tamped cold patch asphalt.

Field work was monitored on a full-time basis by members of Golder's technical staff who: located the boreholes in the field; arranged for the clearance of underground services; supervised the drilling and sampling operations; logged the boreholes; and examined the soil samples. The soil samples were identified in the field, placed in labelled containers, and transported to Golder's geotechnical laboratory in Sudbury for further examination and laboratory testing. Index and classification testing consisting of water content determinations, grain size distributions, Atterberg limits tests, and an organic content test were carried out on selected soil samples. In addition, unconfined compressive strength (UCS) testing was carried out on four selected specimens of the bedrock core recovered from the boreholes. The geotechnical laboratory testing on soil and rock core samples was completed according to ASTM and MTO LS standards, as applicable. In addition, a soil sample from one of the boreholes at each abutment was obtained using appropriate protocols and submitted to a specialist analytical laboratory under chain of custody procedures for testing for a suite of parameters including pH, resistivity, conductivity, sulphates, and chlorides.

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[1] ASTM D1586/D1586M-18 Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils.

[2] ASTM D2573/D2573M-18 Standard Test Method of Field Vane Shear Test in Saturated Fine-Grained Soils.

Classification of the rock mass quality of the bedrock is based on the Rock Quality Designation (RQD) as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006[3]). The degree of weathering of the bedrock samples (i.e., fresh to slightly weathered), as described in the Lithological and Geotechnical Rock Description Terminology included in Appendix A, is based on field identification and the strength classification of the intact rock mass (i.e., strong to very strong) is described in accordance with Table 3.5 of CFEM (2006).

The as-drilled borehole locations were measured by a member of our technical staff, referenced to the highway centerline and existing bridge structure using a measuring tape and the locations were converted into northing/easting coordinates on the plan drawing. Given the relatively short distances between the boreholes and the existing highway centerline/bridge, the measurements are considered accurate to within 0.5 m horizontally. The ground surface elevations at the borehole locations were obtained using a survey level and rod and the survey loop was closed to within 0.1 m vertically. The boreholes were surveyed relative to benchmark HCP-105 and the geodetic elevation of the benchmark (Elevation 181.802 m) was obtained from the General Arrangement (GA) drawing (4678-Whitefish River Bridge-01-General Arrangement.dwg) provided by D.M. Wills. The MTM NAD83 Zone 12 CSRS CBNV6-2010.0 northing and easting coordinates, World Geodetic System 1984 (WGS 84) geographical coordinates, ground surface elevations referenced to Geodetic datum, and borehole depths at each borehole location are presented on the Record of Borehole sheets in Appendix A and summarized below.

Borehole Number	MTM NAD83 Northing (Latitude)	MTM NAD83 Easting (Longitude)	Ground Surface Elevation (m)	Borehole Depth* (m)
WR-1	5108402.6 (46.113084)	248390.8 (-81.729771)	182.1	7.8
WR-2	5108394.0 (46.113007)	248399.1 (-81.729663)	181.9	6.0
WR-3	5108392.1 (46.112991)	248397.5 (-81.729683)	181.9	6.4
WR-4	5108367.6 (46.112772)	248429.6 (-81.729265)	181.9	4.8
WR-5	5108366.0 (46.112758)	248428.2 (-81.729283)	181.9	4.9
WR-6	5108358.1 (46.112687)	248435.7 (-81.729185)	181.6	7.7
WR-7	5108394.4 (46.113011)	248395.0 (-81.729716)	181.9	4.4
WR-8	5108366.6 (46.112764)	248432.4 (-81.729229)	181.8	1.8

* Includes coring of bedrock for lengths between 0.4 m and 3.5 m in boreholes WR-1 to WR-7.

Additionally, Golder surveyed the ground surface elevation at exposed bedrock outcrops in the vicinity of the abutments at the approximate locations shown in Drawing 1 as summarized below.

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[3] Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

Approximate Vicinity of Survey Location	Bedrock Surface Elevation (m)
West side of north abutment	176.3
East side of north abutment	176.9
West side of south abutment	180.9
East side of south abutment	179.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain Study (NOEGTS)[4] mapping, the Whitefish River Bridge site is located within a glaciolacustrine plain deposit consisting primarily of sands bordered by jagged, rugged and cliffed bedrock knobs.

Based on geological mapping by the Ontario Ministry of Northern Development and Mines (MNDM)[5], the site is underlain by bedrock comprised of sedimentary rocks such as conglomerate, wacke, arkose quartz arenite and argillite.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets contained in Appendix A. The detailed results of geotechnical laboratory testing are contained in Appendix B. The results of the in-situ field tests (i.e., SPT 'N'-values and field vane undrained shear strengths) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile and cross sections in Drawings 1 and 2 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

In summary, the subsoil conditions encountered at the site consist of asphalt and granular fill underlain by native deposits of silt, clayey silt to silt, and clayey silt to clay and/or bedrock. A more detailed description of the soil deposits and groundwater conditions encountered in the boreholes is provided below.

4.2.1 Asphalt

A 50 mm to 230 mm thick layer of asphalt was encountered at ground surface in all boreholes. A 150 mm thick layer of reclaimed asphalt pavement (RAP) was encountered below the asphalt in Borehole WR-1 and a 100 mm to 130 mm thick layer of buried asphalt was encountered within the granular fill in Boreholes WR-2, WR-3, WR-5, WR-7 and WR-8 as noted in the borehole records in Appendix A.

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[4] Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 41ISW.

[5] Ontario Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey – Map 2544.

4.2.2 Silty Sand (SM) to Sandy Gravel (GP) (Fill)

The asphalt in Boreholes WR-1 to WR-8 is underlain by granular fill varying in composition from silty sand (SM) to sandy gravel (GP). The granular fill layer was between 0.3 m and 3.9 m thick.

As noted above, the granular fill contains RAP and/or buried asphalt. In addition, cobbles and boulders ranging in size from 100 mm to 330 mm as noted on the borehole records was encountered within the fill at some locations. These zones of cobbles/boulders required NW casing and/or NQ coring techniques to advance the boreholes through the fill. Further, occasional instances of Standard Penetration Test (SPT) sampling resulted in small size samples (or empty split-spoon) being recovered in Boreholes WR-3, WR-4, WR-5, which is further indicative of the potential presence of cobble and/or boulder obstructions.

SPT ‘N’-values measured within the fill range from 9 blows to 47 blows per 0.3 m of penetration indicating a loose to dense state of compactness. In five instances, the split-spoon sampler did not penetrate the entire SPT depth due to refusal conditions (i.e., split-spoon bouncing) on inferred cobble/boulder obstructions or the underlying bedrock surface.

The water content measured on 23 samples of the fill range from about 1% to 19%. The organic content measure on one sample of the fill was 4%.

The results of grain size distribution tests completed on five samples of the granular fill are shown in Figures B-1 and B-2 in Appendix B.

4.2.3 Silt (ML)

Underlying the fill in Borehole WR-1 and underlying the clayey silt in Borehole WR-6, as discussed in the following section, a 1.4 m and 0.9 m thick layer of silt was encountered in the respective boreholes.

The water content measured on four samples of the silt deposit ranged from about 19% to 30%.

SPT ‘N’-values measured within the silt deposit range from 3 blows to 19 blows per 0.3 m of penetration indicating a very loose to compact state of compactness.

The results of the grain size distribution tests completed on two samples of the silt deposit are shown in Figure B-3 in Appendix B. One Atterberg limit test carried out on a sample of the silt deposit yielded a non-plastic test result.

4.2.4 Clayey Silt (CL) to Clay (CH)

Underlying the silt deposit in Borehole WR-1 and underlying the fill in Borehole WR-6, a 3.7 m and 0.4 m thick deposit of clay and clayey silt was encountered in the respective boreholes. The deposit was encountered at Elevation 180.3 m in Borehole WR-1 and Elevation 178.5 m in Borehole WR-6.

The water content measured on four samples of the clayey silt to clay deposit range from 28% to 63%.

SPT ‘N’-values measured within the clayey silt to clay deposit range from 1 blow to 14 blows per 0.3 m of penetration suggesting a very soft to stiff consistency. In-situ field vane tests carried out within the cohesive deposit measured undrained shear strengths between about 35 kPa and 63 kPa indicating a firm to stiff consistency.

The grain size distributions of two samples of the clayey silt to clay deposit are presented in Figure B-4.

Atterberg limits tests carried out on two selected samples of the clayey silt to clay deposit yielded liquid limits of about 33% and 67%, plastic limits of about 17% and 22% and plasticity indices of about 17% and 45%.
The results of the Atterberg limits tests are shown on the plasticity chart in Figure B-5 in Appendix B and indicate that the deposit consists of clayey silt of low plasticity to clay of high plasticity.

4.2.5 Clayey Silt – Silt (CL-ML)

A 0.7 m thick layer of cohesive clayey silt – silt was encountered underlying the clay deposit in Borehole WR-1 at Elevation 176.6 m.

The water content of a sample of clayey silt-silt deposit is about 29%.

An SPT was attempted within the clayey silt, measuring an ‘N’-value of 19 blows for 0.15 m of penetration, but the split-spoon sampler did not penetrate the entire SPT depth due to refusal (i.e., hammer bouncing) on the underlying bedrock surface.

In-situ field vane testing within the clayey silt-silt deposit measured an undrained shear strength of about 45 kPa indicating a firm consistency.

Atterberg limit testing carried out on a sample of the clayey silt – silt deposit yielded a liquid limit of about 21%, a plastic limit of about 16% and a corresponding plasticity index of about 5% indicating the deposit is on the borderline of a low plastic clayey silt to slightly plastic silt. The results of the Atterberg limits test is shown on the plasticity chart in Figure B-6 in Appendix B.

4.3 Bedrock and Refusal Conditions

Refusal conditions (auger and/or split-spoon refusal) were encountered in all boreholes and bedrock was confirmed via coring in Boreholes WR-1 to WR-7. The depth to / elevation of the inferred/confirmed bedrock surface is presented in the table below.

Borehole No.	Location	Depth to Inferred/Confirmed Bedrock Surface (m)	Inferred/Confirmed Bedrock Surface Elevation (m)	Core Length (m)
WR-1	North Approach	6.3	175.8	1.5
WR-2	North Abutment	3.0	178.9	3.0
WR-3	North Abutment	2.9	179.0	3.5
WR-4	South Abutment	1.7	180.2	3.1
WR-5	South Abutment	1.5	180.4	3.4
WR-6	South Approach	4.4	177.2	3.3
WR-7	North Abutment	4.0	177.9	0.4
WR-8	South Abutment	1.8	180.0	-

The retrieved bedrock core is described as fresh, grey to pink, fine grained, medium strong to very strong arkose. Additional details of the bedrock core are presented in the Record of Drillhole sheets in Appendix A, including data on the discontinuity frequency and type. Photographs of the retrieved bedrock core samples are shown in Figures B-7A and B-7B and a summary of the rock core test data (including unconfined compressive strength) is shown in Figure B-8, in Appendix B. The bedrock properties of the retrieved cores and the selected samples for laboratory testing are summarized below.

Borehole No.	Total Core Recovery (TCR)	Rock Quality Designation (RQD)	Quality Classification (Table 3.10 of CFEM 2006[6])	Uniaxial Compressive Strength (MPa)	Strength Classification (Table 3.5 of CFEM 2006)
WR-1	98%	98%	Excellent	-	-
WR-2	100%	100%	Excellent	170.1	(R5) Very Strong
WR-3	82% to 100%	82% to 100%	Good to Excellent	41.9	(R3) Medium Strong
WR-4	100%	100%	Excellent	200.3	(R5) Very Strong
WR-5	100%	100%	Excellent	145.7	(R5) Very Strong
WR-6	100%	100%	Excellent	-	-
WR-7	19%	0%	-	-	-

Note: The low RQD with Borehole WR-7 is likely due to the limited coring length, limited core recovery, and potential drilling disturbance during seating, and is therefore, not considered representative of the overall quality of the rock mass.

4.4 Groundwater Conditions

Unstabilized groundwater levels measured in the open boreholes upon completion of drilling/coring are summarized below. The river water level was surveyed by others at Elevation 176.45 m in January 2019 and by Golder at Elevation 177.1 m in June 2019 (i.e., at the time of the drilling investigation). Groundwater and river water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

Borehole No.	Depth to Groundwater Level* (m)	Groundwater Elevation* (m)
WR-1	2.4	179.7
WR-2	3.3	178.6
WR-3	2.9	179.0
WR-4	1.7	180.2
WR-5	1.5	180.4
WR-6	2.8	178.8
WR-7	3.8	178.1
WR-8	1.8	180.0

Note: Boreholes WR-1 to WR-7 were advanced using NW Casing and wash boring and NQ coring and as such, the measured groundwater level may not be representative of the stabilized, in-situ groundwater conditions.

4.5 Analytical Testing of Soil

The results of the analytical testing of one sample of gravelly sand fill obtained from Borehole WR-2 and one sample of gravelly sand to sand fill from Borehole WR-4, carried out by Bureau Veritas (formerly Maxxam Analytics) an accredited analytical testing laboratory, are detailed in the laboratory test report (Certificate of Analysis) included in Appendix C and summarized below.

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[6] Canadian Geological Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

Parameter	Units	Borehole WR-2, Sample 3	Borehole WR-4, Samples 2B and 3 Combined
Resistivity	ohm-cm	2900	1600
Conductivity	µmho/cm	342	622
pH	pH	7.28	6.78
Sulphate	µg/g	<20*	<20*
Chloride	µg/g	150	340
Sulphide	µg/g	<0.30*	<0.30*

Note: * Concentration is lower than the Reportable Detection Limit (RDL).

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Shane Albert, under the overall direction of Mr. Adam Core P.Eng. and Mr. David Muldowney, P.Eng. This Foundation Investigation Report was prepared by Mr. Adam Core P.Eng. and the technical aspects were reviewed by Mr. David Muldowney, P.Eng. Mr. Paul Dittrich, P.Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted an independent quality control review of this report.

Signature Page

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PART B

FOUNDATION DESIGN REPORT
WHITEFISH RIVER BRIDGE, SITE NO. 46X-0201/B0
HIGHWAY 7041, MONGOWIN TOWNSHIP, DISTRICT OF SUDBURY
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5184-14-00, WP 5182-17-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering design recommendations for the proposed replacement of the Whitefish River Bridge (Site 46X-0201/B0). The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface exploration. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess feasible foundation alternatives and to carry out the conceptual design of the temporary shoring/dewatering system(s). The foundation investigation report, discussion, and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO), and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

The Whitefish River Bridge is located on Highway 7041 at approximately Station 10+935 Mongowin Township, in the District of Sudbury, Ontario (i.e., about 0.4 km north of the Highway 6 south junction). The existing structure consists of a single-lane, single-span, steel truss permanent Modular Bridge (MB), which is approximately 4.2 m wide by 33.5 m long. Based on the encountered subsurface conditions in the boreholes, together with a visual inspection of the north and south abutments, the existing bridge appears to be founded directly on bedrock.

Based on the General Arrangement (GA) drawing provided by D.M. Wills, we understand that the preferred replacement option is a two-lane, single-span, MB structure, which is 7.5 m wide by 39.6 m in length (approximately 6 m longer than the existing). Based on the encountered subsurface conditions, we understand the replacement bridge is to be founded on cast-in-place shallow footings bearing directly on bedrock, with the proposed footings being located approximately 3 m behind the existing abutments. Given that the replacement bridge will be a wider two-lane structure, we also understand that a slight embankment widening is required locally at the bridge abutments; however, the existing highway alignment and grade is to generally remain unchanged with potentially a small grade raise up to 200 mm) as part of the embankment reinstatement following the bridge construction. Further, we understand that a full road closure will be utilized to facilitate construction.

Based on the proposed bridge geometry and the encountered subsurface condition at this site, conventional shallow footings are considered to be the most practical and most economical foundation option for the proposed replacement structure. Given the shallow depth to bedrock, and the modular nature of the proposed replacement bridge, deep foundation options are not considered to be practical and are not discussed further in this report.

6.2 Consequence and Site Understanding Classification

Given that Highway 7041 carries a low volume of traffic, with a projected average annual daily traffic (AADT) of 290 to 350 vehicles over a 20-year period from 2020 to 2040 (based on the draft Design Criteria for GWP 5184-14-00, dated August 2019), and has limited potential to impact alternative transportation corridors, consideration could be given to adopting a “low” consequence level as outlined in Section 6.5 of the *Canadian Highway Bridge Design Code* (CHBDC 2014) and its *Commentary*. However, given the favourable subsurface conditions encountered at this site, a more conventional “typical” consequence level has been utilized for foundation design purposes. Further, given the scope of work of the foundation field exploration and laboratory testing program as presented in Sections 3.0 and 4.0, a “typical degree of site and prediction model understanding" has been utilized. Accordingly, the appropriate corresponding Ultimate Limit State (ULS) and Serviceability Limit Stat (SLS) consequence factor, Ψ, and geotechnical resistance factors, Φ_{gu} and Φ_{gs}, from Tables 6.1 and 6.2 of the CHBDC have been used for design.

6.3 Modular Bridge Foundation Recommendations

6.3.1 Founding Level and Geotechnical Resistances

The subsoils encountered in the vicinity of the bridge abutments consist of granular embankment fill, which is about 2.9 m to 4.0 m thick at the north abutment and about 1.5 m to 1.8 m thick at the south abutment, underlain by arkose bedrock. Given the shallow depth to the encountered bedrock at the proposed foundation locations, it is recommended that the replacement bridge be supported by shallow footings founded directly on the bedrock surface, or on mass concrete on the bedrock surface. The recommended footing founding levels/elevations at the north and south abutments are summarized below.

Location	Boreholes	Anticipated Bedrock Elevation* (m)
North Abutment	WR-2, WR-3 and WR-7	177.9 – 179.0
South Abutment	WR-4, WR-5 and WR-8	180.0 – 180.4

*It should be noted that the bedrock elevation is anticipated to vary beyond the borehole locations based on the observed variations in the exposed bedrock surface within the vicinity of the existing bridge structure.

The footings may be designed to be founded within the range of elevations indicated in the table above. It is noted that the footings should be founded on a horizontal bearing surface; as such, if the lowest elevation in the range is adopted as the founding level, a hydraulic rock-breaker (i.e., hoe-ram) and/or controlled blasting with line drilling will be required to remove a portion of the bedrock within the footprint. If the highest elevation in the range is adopted as the founding level, mass concrete (possibly with the need for rock dowels to satisfy lateral/sliding resistance) will be required to fill in the lower portions of the bedrock and achieve a level surface. Alternatively, a combination of rock excavation and mass concrete could be adopted if an intermediate founding elevation is selected for design.

For the anticipated 1.5 m to 2.5 m wide spread footings constructed on the properly cleaned and prepared bedrock surface at the approximate elevations noted above, a factored ultimate geotechnical axial resistance at ULS of 20 MPa may be used for design.

The factored serviceability geotechnical axial resistance at SLS, for 25 mm of settlement, will be greater than the factored ultimate geotechnical axial resistance, as the bedrock is considered to be an unyielding material; as such, ULS conditions will govern the footing design.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the base of the footings. Where loads are not applied perpendicular to the base of the footings, inclination of the loads should be taken into account in accordance with Section 6.10.4 and Section C6.10.4 of the CHBDC (2014) and its Commentary. The geotechnical resistances provided above also assume that any mass concrete utilized to level the footing bearing surface will have a compressive strength no less than 20 MPa.

6.3.2 Frost Protection

The estimated depth of frost penetration in the area of the Whitefish River bridge is about 2.0 m, as interpreted from OPSD 3090.100 (Foundation, Frost Penetration Depths for Northern Ontario). However, for footings founded on bedrock, soil cover for protection from frost penetration is not considered necessary.

6.3.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the concrete strip footings and/or mass concrete and the bedrock surface should be calculated in accordance with Section 6.10.5 of the CHBDC (2014) applying the appropriate consequence and degree of site understanding factors as noted in Section 6.2. For cast-in-place concrete footings founded directly on the bedrock, the coefficient of friction (tan δ) may be taken as 0.7 (NAVFAC, 1986).

Dowels connecting the concrete footing (or mass concrete) / bedrock should be incorporated into the design where bedrock is found to be sloping at greater than 10 degrees and/or if additional horizontal resistance is required. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout, and steel. The factored lateral ultimate geotechnical resistance of the rock mass at this site is 3 MPa. The factored strength of the grout or concrete must be considered relative to the factored strength of rock mass and the critical (i.e., lower) value must be used for design.

The dowels should have a minimum 1.5 m embedment into fair quality bedrock (i.e., RQD >50%) and the structural strength of the dowels and compressive strength of the grout should not be exceeded. If dowelling into bedrock is incorporated into the design at this site, Non-Standard Special Provision (NSSP) FOUN0002 (Dowels Into Rock) should be included in the Contract Documents to specify the installation, materials and testing of the dowels, a copy of which is included in Appendix D.

6.3.4 Seismic Considerations

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole exploration program. Based on the subsurface conditions, the site may be classified as Site Class B “rock” in accordance with Table 4.1 of the CHBDC (2014). Geophysics testing (i.e., shear wave velocity measurements), if carried out, could potentially provide a more favourable Site Class A “hard rock” designation.

Based on the information obtained from the NRCan (2015) Hazard Calculator for this site located at Latitude 46.112902 and Longitude -81.729466, the following values were obtained for the spectral acceleration for a return period of 2,475 years:

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
Sa (0.2) (g)	0.085
Sa (1.0) (g)	0.039

Based on the values noted above and in accordance with Table 4.10 of the CHBDC (2014), this site should be considered to be located in Seismic Performance Zone 1 for “major-route and other bridges". In accordance with Section 4.4.5.1 of the CHBDC (2014), no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.4 Reconstructed Embankment Stability and Settlement

6.4.1 Embankment Stability

Based on our site observations at the time of the subsurface exploration and available site photographs, and as noted in Section 2.0, the existing highway embankments in the vicinity of the bridge appear to be performing satisfactorily with no evidence of instability or settlement (i.e., soil movement).

Based on discussions with D.M. Wills, we understand that the proposed approach embankments will be reconstructed as part of the overall bridge replacement strategy. The grades at the bridge approaches will generally be maintained with relatively minor grade changes of less than about ±200 mm; however, localized embankment widening of about 2 m is being proposed at the approach embankment to accommodate the wider replacement bridge structure. For the subsurface conditions present at this site and the approximately 2.5 m to 4 m reconstructed embankments, relative to the embankment toes of slope at the proposed south and north abutments, respectively, will have an estimated factor of safety greater than 1.54 if constructed with side slopes inclined at 2 horizontal to 1 vertical (2H:1V) or flatter and assuming that all organics and any softened/loose materials within the footprint of the narrow widening have been removed prior to new fill placement. New fill materials placed adjacent to the existing earth fill embankments for construction of the new widened approach embankment platform should be benched into the existing embankment slopes in accordance with OPSD 208.010.

6.4.2 Embankment Settlement

Given that the existing highway grade and alignment is generally being maintained, and considering the limited overburden thickness and shallow depth to bedrock at this site, settlement of the new slightly higher and widened approach embankments is anticipated to be relatively minor (i.e., less than 25 mm).

6.4.3 Lateral Earth Pressures

The lateral earth pressures acting on the bridge abutment walls and any associated wing walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of walls for this site. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting) as amended by Standard Special Provision (SSP) 105S22. Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and OPSD 3190.100 (Walls, Retaining and Abutment, Wall Drain).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northern Region Directive

(2002) for backfill to structures adjacent to rockfill embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501, as amended by SSP105S22. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 2.0 m behind the back of the wall (in accordance with Figure C6.20 (a) of the Commentary to the CHBDC). For unrestrained walls, granular fill should be placed within the wedge-shaped zone defined by a line drawn at or flatter than 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing in accordance with Figure C6.20(b) of the Commentary to the CHBDC (2014).
- The lateral earth pressures are based on the proposed backfill and embankment reconstruction fill material and the following parameters (unfactored) may be used:

Fill Type	Internal Angle of Friction, ϕ	Unit Weight	Coefficients of Static Lateral Earth Pressure		
			Active, Ka	At Rest, Ko	Passive, Kp
New Granular 'A' (compacted)	35°	22 kN/m^3	0.27	0.43	3.69
New Granular 'B' Type I or II (compacted)	35°	21 kN/m^3	0.27	0.43	3.69

The total passive resistance in front of the walls may be calculated based on the values of Kp indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

6.5 Construction Considerations

6.5.1 Sub-Excavation, Subgrade Preparation and Embankment Re-Construction

Based on discussions with D.M. Wills, we understand that a full road closure is being proposed with temporary open-cut excavations for the construction of the replacement bridge foundations. Temporary excavations for the bridge footing construction will extend through the existing embankment granular fill and native soils (where present) down to the underlying bedrock surface. All excavations must be carried out in accordance with Ontario Regulation 213, Ontario Occupational Health and Safety Act for Construction Projects (as amended).

The granular fill and native soils are considered to be Type 3 soil above the groundwater table and Type 4 soil below the groundwater table. Temporary open-cut excavations in Type 3 soils should remain stable if side slopes are formed no steeper than 1H:1V. In Type 4 soils, the side slopes should be formed no steeper than 3H:1V.

Prior to construction of the footings, the existing fill should be sub-excavated to expose the bedrock surface within the plan limits of the footings. The exposed bedrock surface should be inspected by a qualified Foundation Engineering Specialist following sub-excavation to ensure that the bedrock surface is properly cleaned, scaled and all loosened debris has been removed prior to pouring concrete in accordance with OPSS 902 (Excavating and Backfilling Structures).

The bedrock surface was noted to be sloping across the footprint of the proposed abutments with a top of bedrock elevation difference of about 1.1 m across the north abutment and about 0.4 m across the south abutment. A sample Notice to Contractor regarding the subsurface conditions (i.e., variable bedrock surface elevations) is provided in Appendix D for inclusion into the Contract Documents. As such, consideration may need to be given to dowelling the footings and/or levelling the bedrock surface with mass concrete to create a horizontal bearing surface for the footings. Dowels connecting the footing/bedrock should be incorporated into the design where bedrock is found to be sloping at greater than 10 degrees and/or if additional horizontal resistance is required. Alternatively, consideration could be given to lowering the footing founding elevation to the lowest point of bedrock within the footing footprint and sub-excavating the upper portion of the exposed bedrock, as required. The bedrock is classified as medium strong to very strong and pre-drilling and hoe ramming techniques alone may not be adequate to excavate the bedrock at this site. As such, consideration could be given to controlled blasting excavation techniques as per OPSS.PROV 120 (Explosives), as amended by SSP101F02, SSP101S04, and SSP112S10, and OPSS.PROV 202 (Rock Removal - Manual or Blasting), as amended by SSP 202S01 in order to preserve the integrity of the rock mass in the area of the footing excavation. Pre-shearing, line-drilling or other specialized techniques may be required to maintain the excavation lines and preserve the integrity of the rock mass along the footprint of the footings. The effect of blasting on the existing roadway, existing bridge and other nearby infrastructure should be considered by the blasting contractor.

The backfill requirements for the bridge abutments should be in accordance with OPSS 902 (Excavating and Backfilling – Structures) as amended by NSSP FOUN0003 (Dewatering Structure Excavations), a copy of which is provided in Appendix D for inclusion into the contract documents. Backfill material should consist of granular fill meeting the specifications for OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type I or II. The granular backfill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) as amended by SSP105S22. Approach embankment reconstruction/widening adjacent to the bridge abutments should be consistent with OPSD 208.010 (Benching of Earth Slopes).

Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations to ensure that appropriate materials are used, and that adequate levels of compaction have been achieved.

6.5.2 Control of Groundwater and Surface Water

Temporary excavations will be required to sub-excavate the existing embankment fill to expose the bedrock surface at the proposed abutment locations. As a result of the excavation, groundwater seepage into the excavation should be expected due to the relatively permeable nature of the adjacent embankment fill and surface water should be directed away from the excavation areas to prevent ponding of water that could impede footing construction in the dry.

The river water level was surveyed by others at Elevation 176.45 m in January 2019 and by Golder at Elevation 177.1 m in June 2019. Based on the surveyed river water levels and the unstabilized water levels measured in the open boreholes upon completion of drilling/coring operations, the encountered bedrock surface elevations (i.e., about Elevation 179.0 m at the north abutment and 180.4 m at the south abutment), excavations for the proposed bridge footings are anticipated to be above the river water level but may be impacted by perched groundwater conditions, if present at the time of construction. Therefore, it is anticipated that the dewatering at this site will be minimal and can likely be controlled by pumping from properly constructed sumps.

Dewatering should be in accordance with OPSS 902 (Excavation for Structure), as amended by Non-Standard Special Provision (NSSP) FOUN0003 (Dewatering of Structure Excavation), a copy of which is provided in Appendix D. Given the limited anticipated dewatering efforts required for construction of the replacement bridge footings and the relatively shallow depth to bedrock, the Foundation fill-in in Section 902.04.02.01 of NSSP FOUN0003 should indicate that a preconstruction survey is not applicable (N/A).

Based on the subsurface conditions at this site and the anticipated bridge foundation elevations, pumping volumes to unwater/dewater the excavation areas are anticipated to be less than 50,000 L/day and as such, an Environmental Activity Section Registry (ESAR) is not considered to be required. However, the Contractor should be required to evaluate the estimated seepage and groundwater removal quantity, based on their proposed construction methods/procedures and the groundwater conditions at the time of construction, to make the final assessment/determination whether an EASR (or Permit to Take Water (PTTW)) is ultimately required.

6.5.3 Erosion Protection

Embankment restoration after completion of the bridge replacement should be carried out in accordance with OPSS.PROV 206 (Grading), as amended by SSP206F04 and SSP206F06.The requirements for and design of scour and erosion protection measures for the abutments and embankment side slopes should be assessed by the hydraulics design engineer.

Based on the GA drawing provided, we understand that the existing footings are proposed to be left in place (i.e., in front of the proposed (new) footings) and the proposed front face of the abutments will be connected to the proposed footings / underlying bedrock. As such an exposed, granular fill front slope will not be present. Beyond the abutment walls / wingwalls, provisions should be made to provide erosion protection for the exposed, re-constructed embankment side slopes. As a minimum, the exposed embankment side slopes should be covered with topsoil in accordance with OPSS.PROV 802 (Topsoil) and seed and cover in accordance with OPSS.MUNI 804 (Seed and Cover), as amended by SSP804F02. If additional erosion protection is required, consideration could be given to the use rip-rap, rock protection, or granular sheeting, meeting the requirements of OPSS.PROV 1004 (Aggregates – Miscellaneous), as amended by SSP 110S16, which is placed/constructed in accordance with OPSS.PROV 511 (Rip-Rap, Rock Protection and Granular Sheeting)

If this slope protection is not in place before winter, then alternate erosion protection measures, such as covering the slope with straw will be required to reduce the potential for the requirement of remedial works on the side slopes in the spring.

6.5.4 Obstructions

The contractor should be alerted to the presence of cobbles and boulders within the embankment fill as encountered and confirmed by coring in Boreholes WR-2, WR-3, WR-6 and WR-7 and as inferred to be present due to instances of split-spoon refusal (i.e., hammer bouncing) and/or limited split-spoon sample recoveries. The extent and depth of the cobble and boulder obstructions may vary beyond and between the borehole locations. A sample Notice to Contractor regarding the subsurface conditions (i.e., obstructions) is provided in Appendix D for inclusion into the Contract Documents.

6.5.5 Analytical Testing for Construction Materials

The results of analytical tests on two samples of gravelly sand fill obtained near the bedrock interface at the north and south abutments are presented in Section 4.5. The suite of parameters tested is intended to allow the design engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection of steel reinforcing elements.

The analytical test results were compared to Table 3 in CSA A23.1-14. The sulphate concentration measured in the fill samples is less than 0.002% (<20 µg/g), which is below the exposure class S-3 “moderate”; and may be considered negligible according to Table 7.2 of the MTO Gravity Pipe Design Guidelines (2004). However, given that the location of the bridge location is on Highway 7041 and will be exposed to de-icing salts, it is recommended that a “C” type exposure class concrete, as defined by CSA A23.1 -14 Table 1, be considered.

The samples exhibited a pH of 6.8 and 7.3 and resistivity of 1,600 ohm-cm and 2,900 ohm-cm. According to the MTO Gravity Pipe Design Guidelines (2014), pH levels between 5.5 and 8.5 are not considered detrimental to durability. The resistivity is less than 4,500 ohm-cm, which indicates that the soil corrosiveness is “moderate” (4,500>R>2,000) to “severe” (2,000>R) as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014).

It should be noted that the river water levels in the area are subject to seasonal fluctuations and variations, due to the precipitation events, and the water chemistry could also be variable. These recommendations are provided as guidance only. The structural designer should take the results of the laboratory testing, the potential for corrosion, and the corrosion susceptibility of reinforcing steel into consideration as part of the ultimate material selection.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Adam Core, P.Eng. and the technical aspects were reviewed by Mr. David Muldowney, P. Eng. Mr. Paul Dittrich, P. Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted a technical review and independent quality control review of this report.

Signature Page

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
D. Muldowney
D. A. MULDOWNEY
100164716
Sept. 3, 2021
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J. P. DITTRICH
SEPT. 3, 2021
PROVINCE OF ONTARIO

David Muldowney, P.Eng.
Senior Geotechnical Engineer

Paul Dittrich, Ph.D., P.Eng., FEIC
MTO Foundations Designated Contact, Principal

JGH/AC/JMAC/DAM/JPD/hp

https://golderassociates.sharepoint.com/sites/28229g/deliverables/foundations/2. reporting/r01-whitefish river bridge/3. final/18103241-r01-rev0 final fidr mto 5018-e-0009 whitefish river bridge 03sept_2021.docx

REFERENCES

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National Resources Canada Earthquake Hazard Website. http://earthquakescanada.nrcan.gc.ca/hazard-alea/index-eng.php. Accessed on September 2019.

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Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D2573	Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soil

Ontario Provincial Standard Drawings

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirement
OPSD 3101.200	Walls, Abutment, Backfill, Rock
OPSD 3121.150	Walls, Retaining, Backfill Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Ontario Provincial Standard Specifications

OPSS.PROV 120	General Specification for the Use of Explosives
OPSS.PROV 202	Construction Specifications for Rock Removal by Manual Scaling, Machine Scaling, Trim Blasting, or Controlled Blasting
OPSS.PROV 206	Construction Specifications for Grading.
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting
OPSS 802	Construction Specification for Topsoil

OPSS.MUNI 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS Standard Special Provisions

SSP101F02	Amendment to OPSS 120
SSP101S04	Amendment to OPSS 120
SSP105S22	Amendment to OPSS 501
SSP 110S06	Amendment to OPSS 1010
SSP 110S16	Amendment to OPSS 1004
SSP112S10	Amendment to OPSS 120
SSP 202S01	Amendment to OPSS 202
SSP 206F04	Amendment to OPSS 206
SSP 206F06	Amendment to OPSS 206
SSP 804F02	Amendment to OPSS 804

Non-Standard Special Provisions

NSSP FOUN0002	Dowels into Rock
NSSP FOUN0003	Dewatering Structure Excavations (Amendment to OPSS 902)

Notice to Contractor

Subsurface Conditions

Ontario Water Resource Act

Regulation 903 Wells (as amended)

MINISTRY OF TRANSPORTATION, ONTARIO

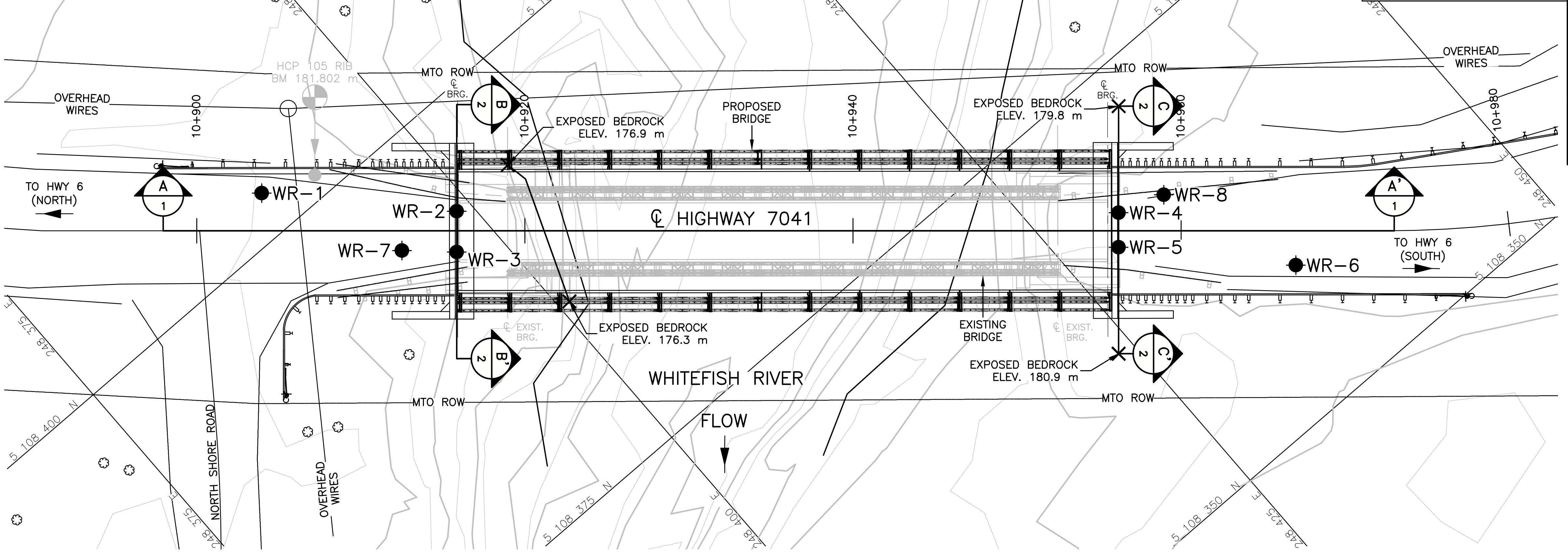
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5182-17-01

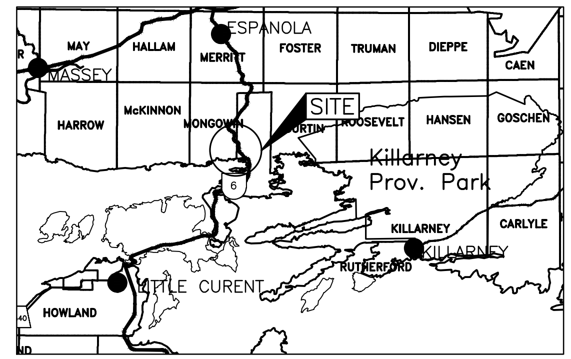
SHEET

HIGHWAY 7041
WHITEFISH RIVER BRIDE
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER
MEMBER OF WSP



PLAN
SCALE
3 0 3 6 m



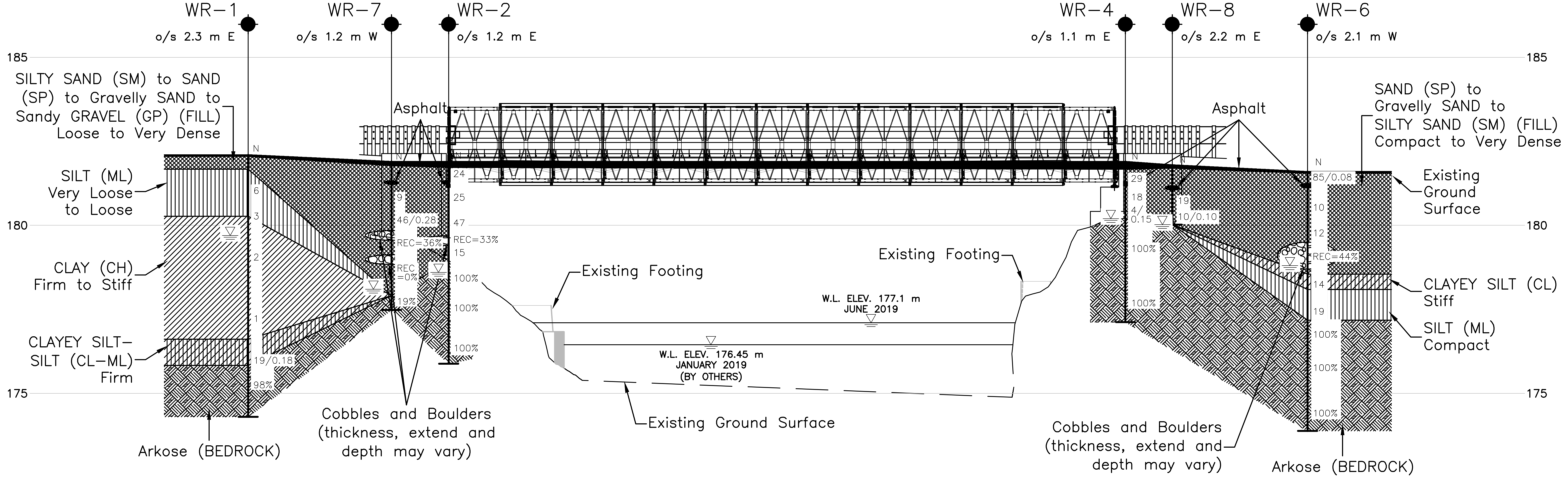
KEY PLAN
SCALE
10 0 10 20 km

LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▽	WL upon completion of drilling
✕	Exposed bedrock survey location

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
WR-1	182.1	5108402.6	248390.8
WR-2	181.9	5108394.0	248399.1
WR-3	181.9	5108392.1	248397.5
WR-4	181.9	5108367.6	248429.6
WR-5	181.9	5108366.0	248428.2
WR-6	181.6	5108358.1	248435.7
WR-7	181.9	5108394.4	248395.0
WR-8	181.8	5108366.6	248432.4



A-A' 1 HIGHWAY CENTERLINE PROFILE

HORIZONTAL SCALE
3 0 3 6 m
1.5 0 1.5 3 m
VERTICAL SCALE

170

J.P. DITTRICH
SEPT 3, 2021

D.A.MULDOWNEY
100164716
SEPT 3, 2021

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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 in digital format by D.M. WILLS LTD. drawing file no. 4678 – Whitefish River Bridge – 01 – General Arrangement, received MAY 16, 2019 and BC70411.dwg, received September 11, 2019.

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 41I-365

HWY. 7041		PROJECT NO. 18103241	DIST. .
SUBM'D. .	CHKD. AC	DATE: 9/3/2021	SITE: 46X-0201/B0
DRAWN: TR	CHKD. DAM	APPD. JMAC/JPD	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

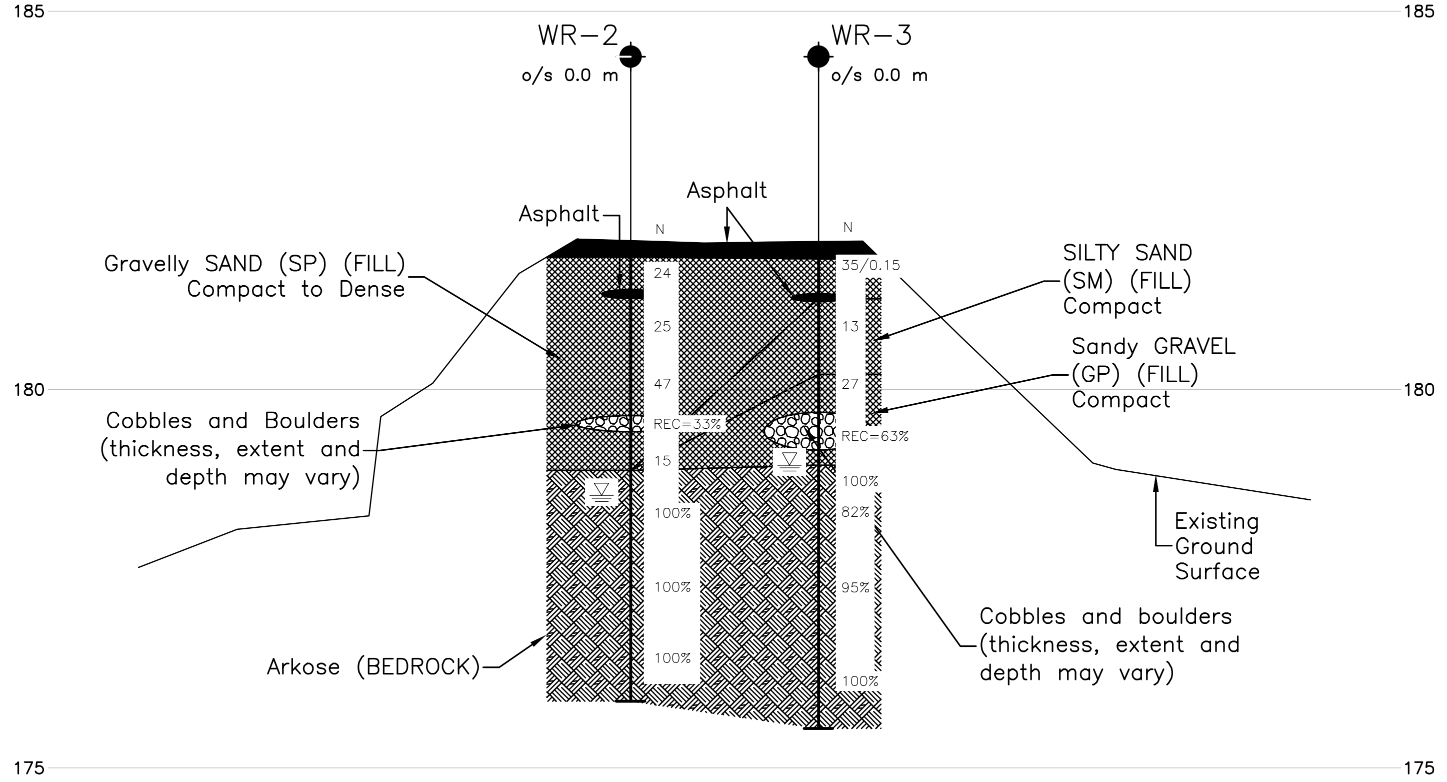
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5182–17–01

HIGHWAY 7041
WHITEFISH RIVER BRIDGE
SOIL STRATA

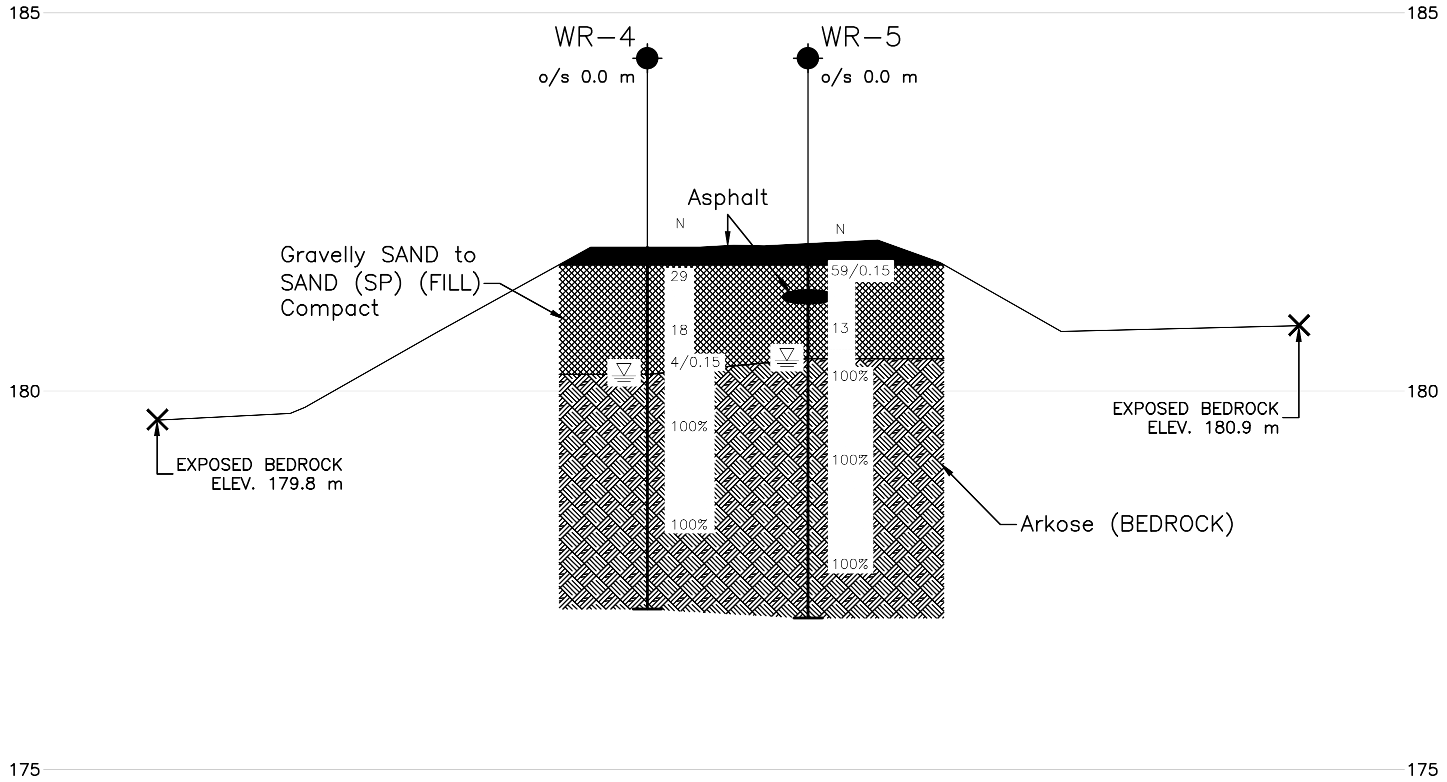
SHEET

GOLDER
MEMBER OF WSP



B–B'
1
CROSS–SECTION

HORIZONTAL SCALE
1 0 1 2 m
1 0 1 2 m
VERTICAL SCALE



C–C'
1
CROSS–SECTION

HORIZONTAL SCALE
1 0 1 2 m
1 0 1 2 m
VERTICAL SCALE

LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
∇	WL upon completion of drilling
✕	Exposed bedrock survey location

BOREHOLE CO–ORDINATES (NAD 83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
WR–2	181.9	5108394.0	248399.1
WR–3	181.9	5108392.1	248397.5
WR–4	181.9	5108367.6	248429.6
WR–5	181.9	5108366.0	248428.2

J.P. DITTRICH
SEPT 3, 2021

D.A.MULDOWNEY
100164716
SEPT 3, 2021

NOTES

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NO.	DATE	BY	REVISION

Geocres No. 41I–365

HWY. 7041		PROJECT NO. 18103241	DIST.
SUBM'D.	CHKD. AC	DATE: 9/3/2021	SITE: 46X–0201/B0
DRAWN: TR	CHKD. DAM	APPD. JMAC/JPD	DWG. 2

PLOT DATE: September 3, 2021
FILENAME: R:\Sudbury\SIM\Clients\MTO\Hwy7041&6\99_PROJ\18103241\40_PROD\0001_Whitefish_River_Bridge_Replacement\18103241-0001-BG-0002.dwg

Photograph 1: South Approach – Facing North (June 2019)

Photograph 2: North Approach – Facing South (June 2019)


Photograph 3: Northeast Quadrant of North Abutment – Facing Southwest (June 2019)


Photograph 4: West Elevation from North side of River – Facing Southeast (upstream) - (June 2019)

Photograph 5: South Abutment on Exposed Bedrock –North Side of Bridge Facing South (June 2019)

Photograph 6: Bedrock Outcrop at west side of the South Approach – Facing East (June 2019)

APPENDIX A

Record of Boreholes

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS
MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
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)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS[1,2]

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (*i.e.,* SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (*i.e.,* some sand)
≤ 10	trace (*i.e.,* trace fines)

1. Only applicable to components not described by Primary Group Name.
2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve friction (f_s) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness[1]

Term	SPT 'N' (blows/0.3m)[2]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	➢ 50

3. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
4. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS
MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
U	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
E	void ratio
N	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

Fresh (W1): no visible sign of rock material weathering.

Slightly Weathered (W2): discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

Moderately Weathered (W3): less than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Highly Weathered (W4): more than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Completely Weathered (W5): 100% of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

Residual Soil (W6): 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.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR 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

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90º angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

AXJ	Axial Joint
BD	Bedding
BC	Broken Core
CC	Continuous Core
CL	Closed
CO	Contact
CU	Curved
CT	Coated
FLT	Fault
FOL	Foliation
FR	Fracture
GO	Gouge
IN	Infilled
IR	Irregular
JN	Joint
KV	Karstic Void
K	Slickensided
LC	Lost Core
MB	Mechanical Break
PL	Planar
PO	Polished
RO	Rough
SA	Slightly Altered
SH	Shear
SM	Smooth
SR	Slightly Rough
SY	Stylolite
UN	Undulating
VN	Vein
VR	Very Rough

ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250

RECORD OF BOREHOLE No. WR-1

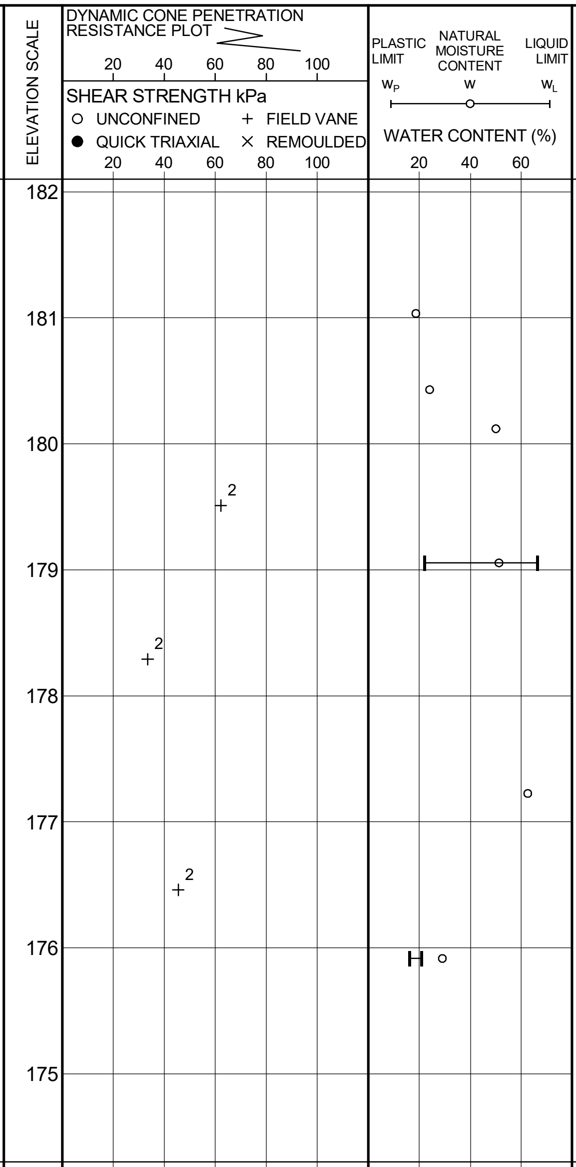
1 OF 1 **METRIC**

PROJECT 18103241			
W.P. 5182-17-01	LOCATION N 5108402.6; E 248390.8 NAD83 MTM ZONE 12 (LAT. 46.113084; LONG. -81.729771)		ORIGINATED BY SA
DIST	HWY 7041	BOREHOLE TYPE 150 mm O.D. Solid Stem Augers, NW Casing, Wash Boring and NQ Coring	COMPILED BY TR
DATUM GEODETIC	DATE June 26, 2019		CHECKED BY AC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.1	GROUND SURFACE						
0.0	ASPHALT (50 mm)						
	RAP (150 mm)						
181.7 0.4	SAND (SP), some gravel, trace asphalt (FILL) Brown Moist						
	SILT (ML), trace sand, trace clay Very loose to loose Grey / brown mottled Moist to wet	1	SS	6			
		2A 2B	SS	3			0 9 85 6
180.3 1.8	CLAY (CH) Firm to stiff Red to grey (below 4.6 m) Wet				∇		
		3	SS	2			0 0 28 72
		4	SS	1			
176.6 5.5	CLAYEY SILT - SILT (CL-ML) Firm Grey Wet (w>PL)	5	SS	19/0.18			
175.8 6.3	ARKOSE (BEDROCK) For coring details see Record of Drillhole WR-1. Bedrock cored from 6.3 m to 7.8 m depth.	1	RC	REC 98%			RQD = 98%
174.3 7.8	END OF BOREHOLE						

NOTE:

1. Water level at a depth of 2.4 m below ground surface (Elev. 179.7 m) upon completion of bedrock coring.



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

PROJECT: 18103241

RECORD OF DRILLHOLE: WR-1

SHEET 1 OF 1

LOCATION: N 5108402.6; E 248390.8
NAD83 MTM ZONE 12 (LAT. 46.113084; LONG. -81.729771)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 26, 2019
DRILL RIG: CME75
DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX METRES (5 10 15 20)	DISCONTINUITY DATA B Angle (0 90 180 270)	DIP w.r.t. CORE AXIS (0 30 60 90)	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.
		BEDROCK SURFACE		175.8															
7	NW NQ Coring June 26, 2019	ARKOSE Fine grained Fresh Grey	▨	6.3 174.3	1	Grey 100							MB MB JNIRSM JNIRSM						
8		END OF DRILLHOLE		7.8															
9																			
10																			
11																			
12																			
13																			
14																			
15																			
16																			
17																			
18																			

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DEPTH SCALE
1 : 60

GOLDER

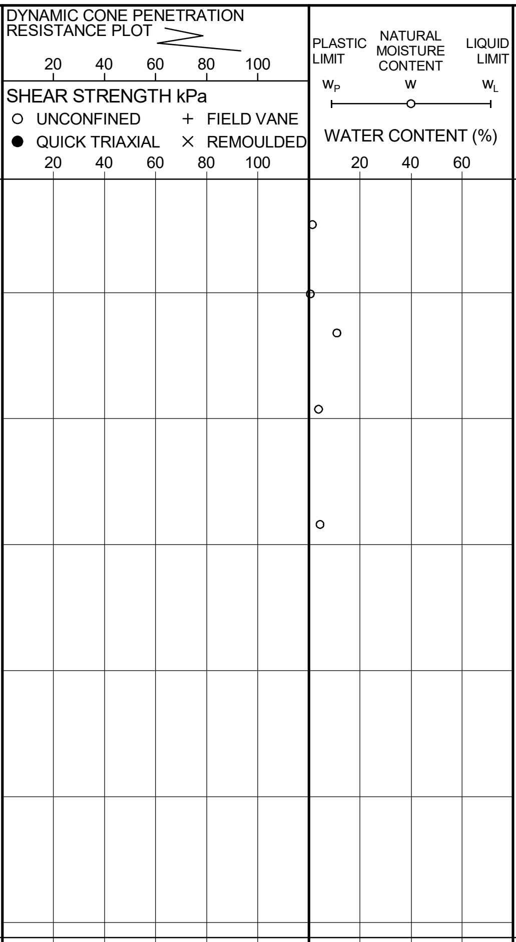
LOGGED: SA
CHECKED: AC

RECORD OF BOREHOLE No. WR-2

1 OF 1 **METRIC**

PROJECT 18103241
W.P. 5182-17-01
DIST
HWY 7041
DATUM GEODETIC
LOCATION N 5108394.0; E 248399.1 NAD83 MTM ZONE 12 (LAT. 46.113007; LONG. -81.729663)
BOREHOLE TYPE 150 mm O.D. Solid Stem Augers, NW Casing, Wash Boring and NQ Coring
DATE June 26, 2019
ORIGINATED BY SA
COMPILED BY TR
CHECKED BY AC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.9	GROUND SURFACE						
0.0	ASPHALT (150 mm)						
0.2	Gravelly SAND (SP), trace silt (FILL) Compact Red / brown Moist	1	SS	24			
181.3 0.7	ASPHALT (100 mm)					181	
	Gravelly SAND (SP) to sandy GRAVEL (GP), trace silt (FILL) Compact to dense Brown Moist - 100 mm cobble encountered at 2.3 m depth	2A 2B	SS	25			
		3	SS	47		180	
		-	RC	REC 33%			
		4	SS	15			72 23 (5)
178.9 3.0	ARKOSE (BEDROCK) For coring details see Record of Drillhole WR-2. Bedrock cored from 3.0 m to 6.0 m depth.	1	RC	REC 100%	∇	179	RQD = 100%
		2	RC	REC 100%		178 177	RQD = 100%
		3	RC	REC 100%		176	RQD = 100%
175.9 6.0	END OF BOREHOLE NOTE: 1. Water level at a depth of 3.3 m below ground surface (Elev. 178.6 m) upon completion of bedrock coring.						



UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

PROJECT: 18103241
LOCATION: N 5108394.0; E 248399.1
NAD83 MTM ZONE 12 (LAT. 46.113007; LONG. -81.729663)
INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: WR-2

DRILLING DATE: June 26, 2019
DRILL RIG: CME75
DRILLING CONTRACTOR: Landcore Drilling

SHEET 1 OF 1
DATUM: GEODETIC

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX METRES 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		BEDROCK SURFACE		178.9																
3	NW	ARKOSE Fine grained Fresh Very strong Grey		3.0																
	NQ Coring June 26, 2019				1	Grey 100														UCS = 170.1 MPa
4													MB							
					2	Grey 100							MB							
5													MB							
					3	Grey 100														
6				175.9									MB							
		END OF DRILLHOLE		6.0																
7																				
8																				
9																				
10																				
11																				
12																				
13																				
14																				

SUD-MTO-RCK S:\CLIENTS\MTO\HWY7041&6\02_DATA\GINT\18103241.GPJ GAL-MISS.GDT 9-26-19 TR

DEPTH SCALE
1 : 60

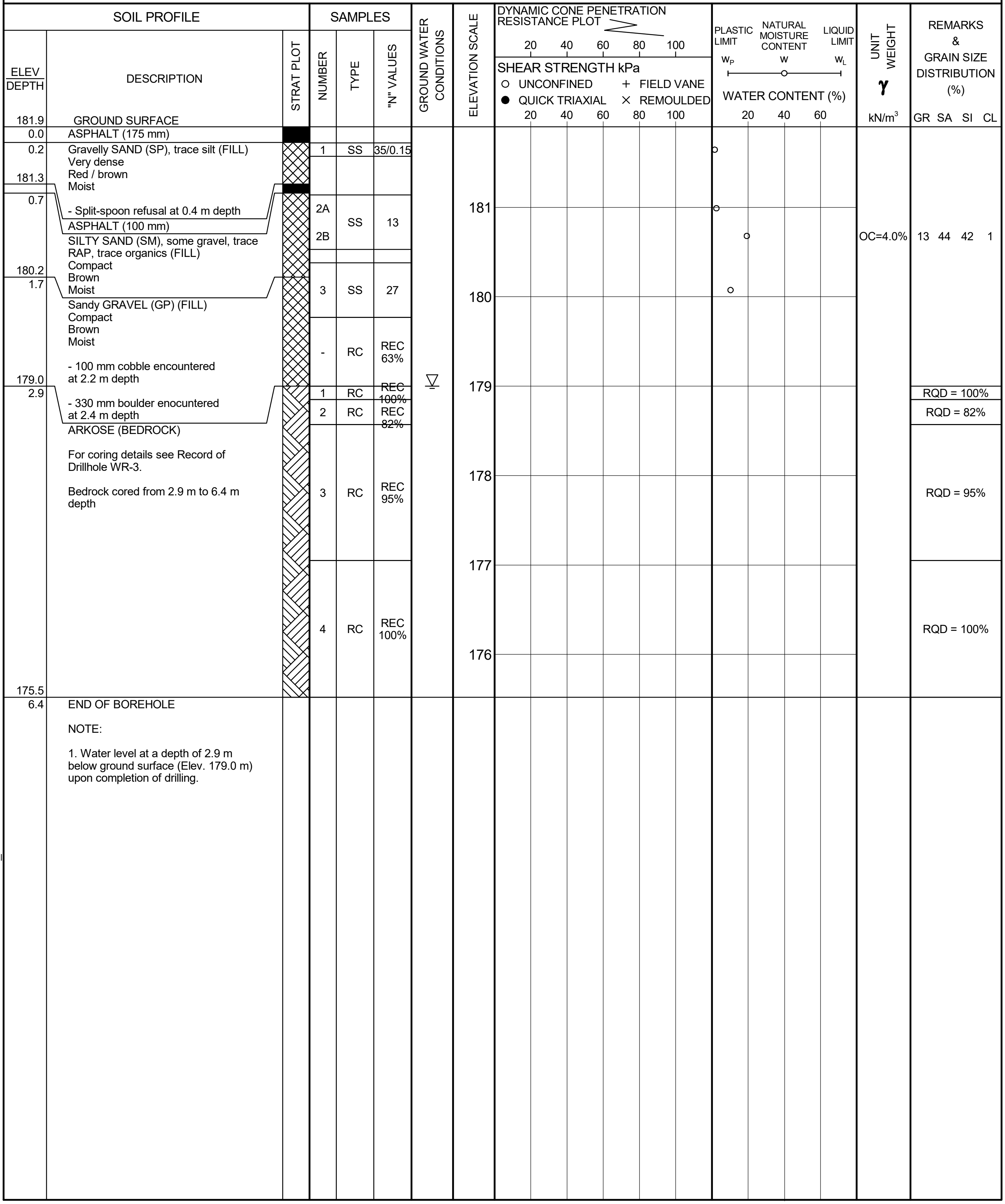
GOLDER

LOGGED: SA
CHECKED: AC

RECORD OF BOREHOLE No. WR-3

1 OF 1 **METRIC**

PROJECT 18103241
W.P. 5182-17-01
DIST
HWY 7041
DATUM GEODETIC
LOCATION N 5108392.1; E 248397.5 NAD83 MTM ZONE 12 (LAT. 46.112991; LONG. -81.729683)
BOREHOLE TYPE 150 mm O.D. Solid Stem Augers, NW Casing, Wash Boring and NQ Coring
DATE June 24, 2019
ORIGINATED BY SA
COMPILED BY TR
CHECKED BY AC



ELEV DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.9	GROUND SURFACE							
0.0	ASPHALT (175 mm)							
0.2	Gravelly SAND (SP), trace silt (FILL) Very dense Red / brown Moist - Split-spoon refusal at 0.4 m depth	1	SS	35/0.15				
181.3 0.7	ASPHALT (100 mm)					181		
	SILTY SAND (SM), some gravel, trace RAP, trace organics (FILL) Compact Brown Moist	2A 2B	SS	13			OC=4.0%	13 44 42 1
180.2 1.7	Sandy GRAVEL (GP) (FILL) Compact Brown Moist - 100 mm cobble encountered at 2.2 m depth - 330 mm boulder enocuntered at 2.4 m depth	3	SS	27		180		
		-	RC	REC 63%	∇	179		
179.0 2.9	ARKOSE (BEDROCK) For coring details see Record of Drillhole WR-3. Bedrock cored from 2.9 m to 6.4 m depth	1	RC	REC 100%				RQD = 100%
		2	RC	REC 82%				RQD = 82%
		3	RC	REC 95%		178		RQD = 95%
		4	RC	REC 100%		177 176		RQD = 100%
175.5 6.4	END OF BOREHOLE NOTE: 1. Water level at a depth of 2.9 m below ground surface (Elev. 179.0 m) upon completion of drilling.							

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

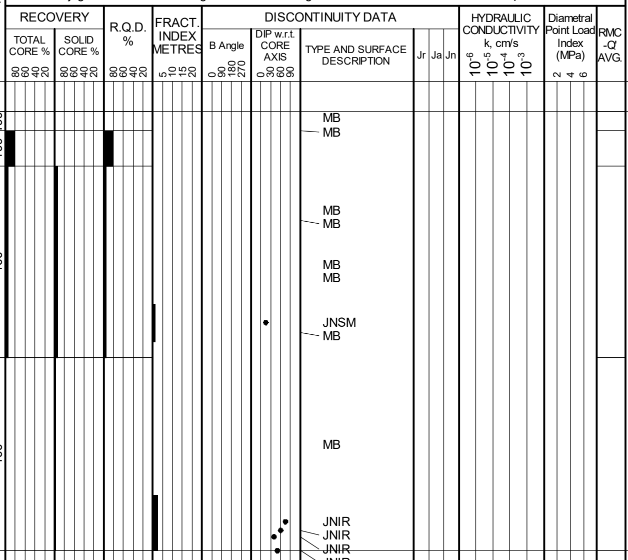
PROJECT: 18103241
LOCATION: N 5108392.1; E 248397.5
NAD83 MTM ZONE 12 (LAT. 46.112991; LONG. -81.729683)
INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: WR-3

DRILLING DATE: June 24, 2019
DRILL RIG: CME75
DRILLING CONTRACTOR: Landcore Drilling

SHEET 1 OF 1
DATUM: GEODETIC

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	COLOUR / % RETURN	FLUSH	TYPE AND SURFACE DESCRIPTION	
		BEDROCK SURFACE		179.0					
3	NW	ARKOSE Fine grained Fresh Medium strong Grey to pink		2.9	1	Grey 100		MB	
	NQ Coring June 24, 2019				2	Grey 100		MB	
4					3	Grey 100		MB MB MB MB JNSM MB	UCS = 41.9 MPa
5					4	Grey 100		MB JNIR JNIR JNIR JNIR	
6				175.5					
		END OF DRILLHOLE		6.4					
7									
8									
9									
10									
11									
12									
13									
14									

DEPTH SCALE
1 : 60

GOLDER

LOGGED: SA
CHECKED: AC

RECORD OF BOREHOLE No. WR-4

1 OF 1 **METRIC**

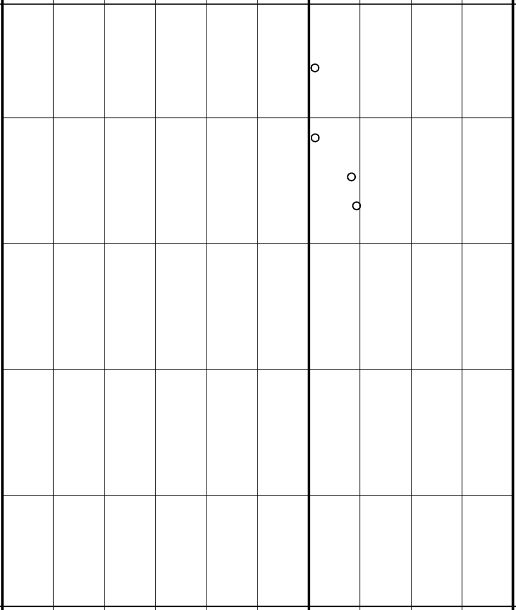
PROJECT 18103241
W.P. 5182-17-01
LOCATION N 5108367.6; E 248429.6 NAD83 MTM ZONE 12 (LAT. 46.112772; LONG. -81.729265)
ORIGINATED BY SA
DIST ____ HWY 7041
BOREHOLE TYPE 150 mm O.D. Solid Stem Augers, NW Casing, Wash Boring and NQ Coring
COMPILED BY TR
DATUM GEODETIC
DATE June 24, 2019
CHECKED BY AC

SOIL PROFILE			SAMPLES					REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR SA SI CL
181.9	GROUND SURFACE							
181.7 / 0.0	ASPHALT (230 mm)	■						
0.2	Gravelly SAND (SP) to SAND (SP), some silt (FILL) Compact Red to brown Moist	▩	1	SS	29		181	
			2A 2B	SS	18			
180.4	SILTY SAND (SM), trace gravel (FILL) Compact Brown Moist	▩	3	SS	4/0.15	∇	180	2 80 18 0
1.7	ARKOSE (BEDROCK) For coring details see Record of Drillhole WR-4. Bedrock cored from 1.7 m to 4.8 m depth	▨	1	RC	REC 100%		179	RQD = 100%
		▨	2	RC	REC 100%		178	RQD = 100%
177.1 / 4.8	END OF BOREHOLE NOTE: 1. Water level at a depth of 1.7 m below ground surface (Elev. 180.2 m) upon completion of bedrock coring.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

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

UNIT WEIGHT γ kN/m³



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

PROJECT: 18103241
LOCATION: N 5108367.6; E 248429.6
NAD83 MTM ZONE 12 (LAT. 46.112772; LONG. -81.729265)
INCLINATION: -90° AZIMUTH: ---

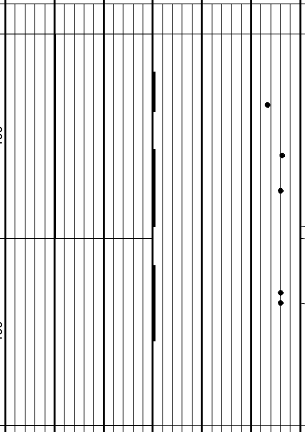
RECORD OF DRILLHOLE: WR-4

DRILLING DATE: June 24, 2019
DRILL RIG: CME75
DRILLING CONTRACTOR: Landcore Drilling

SHEET 1 OF 1
DATUM: GEODETIC

JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate	BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage	PL - Planar CU- Curved UN - Undulating ST - Stepped IR - Irregular	PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break	BR - Broken Rock **NOTE:** For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX METRES 5 10 15 20	B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		BEDROCK SURFACE		180.2																	
2 3 4	NW NQ Coring June 24, 2019	ARKOSE Fine grained Fresh Very strong Grey to grey / pink		1.7	1	Grey 100								MB JNIR MB JNIR JNIR MB MB MB							UCS = 200.3 MPa
				177.1	2	Grey 100								MB JNIR JNIR MB MB							
5 6 7 8 9 10 11 12 13		END OF DRILLHOLE		4.8										MB							



DEPTH SCALE
1 : 60

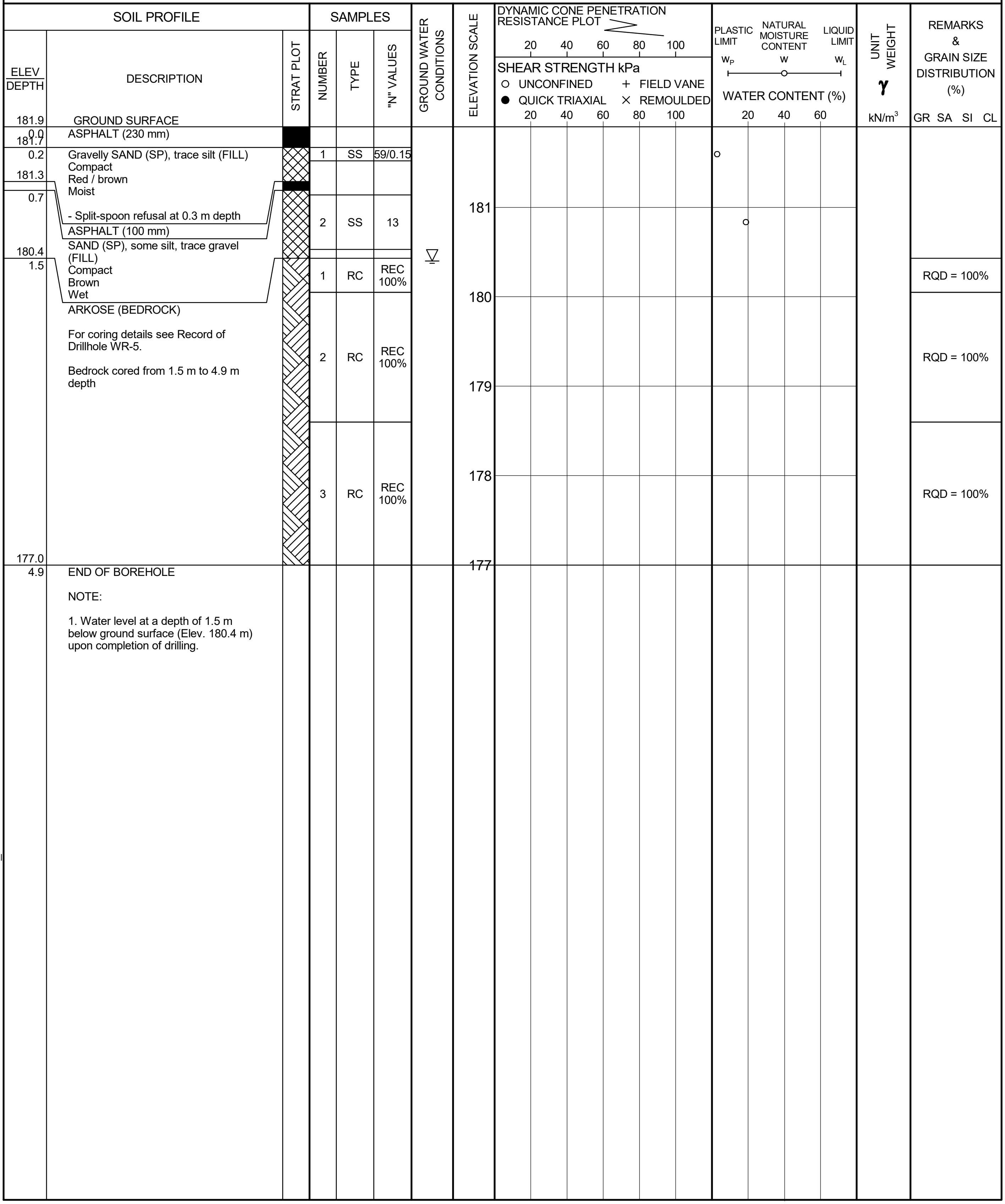
GOLDER

LOGGED: SA
CHECKED: AC

RECORD OF BOREHOLE No. WR-5

1 OF 1 **METRIC**

PROJECT 18103241
W.P. 5182-17-01
DIST
HWY 7041
DATUM GEODETIC
LOCATION N 5108366.0; E 248428.2 NAD83 MTM ZONE 12 (LAT. 46.112758; LONG. -81.729283)
BOREHOLE TYPE 150 mm O.D. Solid Stem Augers, NW Casing, Wash Boring and NQ Coring
DATE June 25, 2019
ORIGINATED BY SA
COMPILED BY TR
CHECKED BY AC



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.9	GROUND SURFACE						
0.0	ASPHALT (230 mm)						
181.7 0.2	Gravelly SAND (SP), trace silt (FILL) Compact Red / brown Moist - Split-spoon refusal at 0.3 m depth	1	SS	59/0.15			
181.3	ASPHALT (100 mm)						
0.7	SAND (SP), some silt, trace gravel (FILL) Compact Brown Wet	2	SS	13	∇		
180.4 1.5	ARKOSE (BEDROCK) For coring details see Record of Drillhole WR-5. Bedrock cored from 1.5 m to 4.9 m depth	1	RC	REC 100%			RQD = 100%
		2	RC	REC 100%			RQD = 100%
		3	RC	REC 100%			RQD = 100%
177.0 4.9	END OF BOREHOLE						

NOTE:

1. Water level at a depth of 1.5 m below ground surface (Elev. 180.4 m) upon completion of drilling.

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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT: 18103241

RECORD OF DRILLHOLE: WR-5

SHEET 1 OF 1

LOCATION: N 5108366.0; E 248428.2
NAD83 MTM ZONE 12 (LAT. 46.112758; LONG. -81.729283)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 25, 2019
DRILL RIG: CME75
DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN / FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX METRES 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		BEDROCK SURFACE		180.4																
	NW	ARKOSE Fine grained Fresh Very strong Light grey		1.5	1	Grey 100							MB MB							
2	NQ Coring June 25, 2019				2	Grey 100							JNIR MB JNIR MB							
3													MB JNIR							UCS = 145.7 MPa
					3	Grey 100							MB							
4													JNIR MB MB MB MB							
		- Lost bottom 75 mm of rock core		177.0																
5		END OF DRILLHOLE		4.9																
6																				
7																				
8																				
9																				
10																				
11																				
12																				
13																				

SUD-MTO-RCK S:\CLIENTS\MTO\HWY7041&6\02_DATA\GINT\18103241.GPJ GAL-MISS.GDT 9-26-19 TR

DEPTH SCALE
1 : 60

GOLDER

LOGGED: SA
CHECKED: AC

RECORD OF BOREHOLE No. WR-6

1 OF 1 **METRIC**

PROJECT 18103241
W.P. 5182-17-01
LOCATION N 5108358.1; E 248435.7 NAD83 MTM ZONE 12 (LAT. 46.112687; LONG. -81.729185)
ORIGINATED BY SA
DIST ____ HWY 7041
BOREHOLE TYPE 150 mm O.D. Solid Stem Augers, NW Casing, Wash Boring and NQ Coring
COMPILED BY TR
DATUM GEODETIC
DATE June 25, 2019
CHECKED BY AC

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
181.6	GROUND SURFACE
0.0	SURFACE TREATMENT (50 mm) Gravelly SAND (SP), trace silt (FILL) Very dense Brown Moist
181.2 0.5	ASPHALT (100 mm) SILTY SAND (SM), some gravel, trace RAP (FILL) Compact Grey / brown mottled Moist to wet
180.2 1.4	Gravelly SAND (SP), some silt (FILL) Compact Brown Moist Cobbles encountered as follows: - 150 mm cobble at 2.4 m depth - 200 mm cobble at 2.7 m depth
178.5 3.1	CLAYEY SILT (CL), with silt laminations, trace gravel, trace sand Stiff Red / brown with grey laminations Wet (w>PL)
178.1 3.5	SILT (ML), trace gravel, trace sand, trace clay Compact Grey / red Wet
177.2 4.4	ARKOSE (BEDROCK) For coring details see Record of Drillhole WR-6. Bedrock cored from 4.4 m to 7.7 m depth
173.9 7.7	END OF BOREHOLE NOTE: 1. Water level at a depth of 2.8 m below ground surface (Elev. 178.8 m) upon completion of bedrock coring.

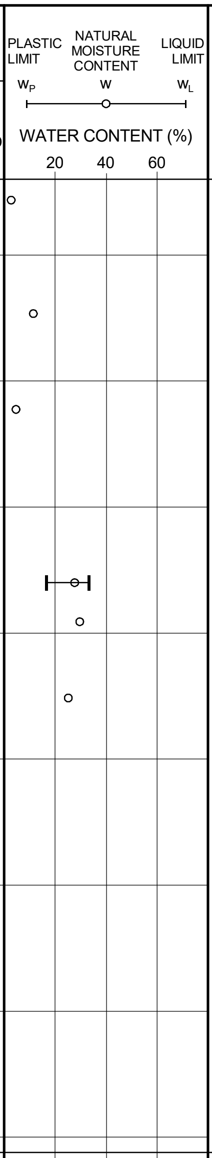
SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	85/0.08						
2	SS	10						
3	SS	12						
-	RC	REC 44%	∇					
4A 4B	SS	14			4	5	51	40
5	SS	19		NP	1	7	86	6
1	RC	REC 100%			RQD = 100%			
2	RC	REC 100%			RQD = 100%			
3	RC	REC 100%			RQD = 100%			

ELEVATION SCALE: 181, 180, 179, 178, 177, 176, 175, 174

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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



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

PROJECT: 18103241

RECORD OF DRILLHOLE: WR-6

SHEET 1 OF 1

LOCATION: N 5108358.1; E 248435.7
NAD83 MTM ZONE 12 (LAT. 46.112687; LONG. -81.729185)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 25, 2019
DRILL RIG: CME75
DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
MB - Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX METRES 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.
		BEDROCK SURFACE		177.2																
	NW	ARKOSE, some quartz Fine grained Fresh Grey	▨	4.4	1		Grey 100							MB MB MB MB						
5	NQ Coring June 25, 2019		▨		2		Grey 100							MB MB MB MB						
6 7			▨		3		Grey 100							MB MB MB						
				173.9 7.7																
8 9 10 11 12 13 14 15 16		END OF DRILLHOLE																		

DEPTH SCALE
1 : 60

GOLDER

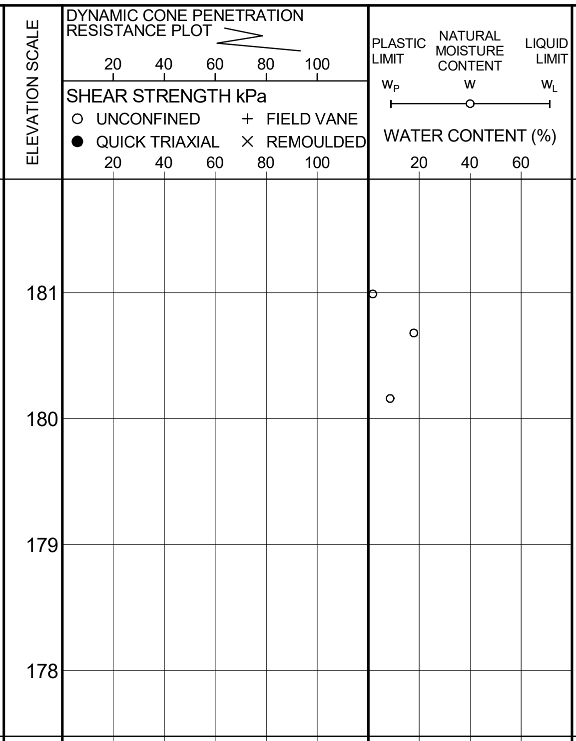
LOGGED: SA
CHECKED: AC

RECORD OF BOREHOLE No. WR-7

1 OF 1 **METRIC**

PROJECT 18103241
W.P. 5182-17-01
LOCATION N 5108394.4; E 248395.0 NAD83 MTM ZONE 12 (LAT. 46.113011; LONG. -81.729716)
ORIGINATED BY SA
DIST
HWY 7041
BOREHOLE TYPE 150 mm O.D. Solid Stem Augers, NW Casing, Wash Boring and NQ Coring
COMPILED BY TR
DATUM GEODETIC
DATE June 27, 2019
CHECKED BY AC

ELEV DEPTH	SOIL PROFILE — DESCRIPTION	STRAT PLOT	SAMPLES — NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.9	GROUND SURFACE							
0.0	ASPHALT (125 mm)	■						
0.1 181.3	SAND (SP) and gravel (FILL) Brown Moist	▨						
	ASPHALT (100 mm)	■						
0.7	SILTY SAND (SM) and gravel to sandy GRAVEL (GP), trace to some silt (FILL) Loose to dense Brown Moist Cobbles encountered as follows: - 100 mm cobble at 2.1 m depth - 230 mm cobble at 2.8 m depth - Core barrel tip partially plugged from 3.0 m to 4.0 m depth, no recovery	▨	1A 1B	SS	9			
			2	SS	46/0.28			38 45 (17)
			-	RC	REC 36%			
			-	RC	REC 0%	▽		
177.9 4.0	Inferred ARKOSE (BEDROCK)	▧	-	RC	REC 19%			RQD = 0%
177.5 4.4	END OF BOREHOLE NOTES: 1. Cored 0.4 m into the bedrock and was only able to recover 75 mm. 2. Water level at a depth of 3.8 m below ground surface (Elev. 178.1 m) upon completion of soil drilling.							



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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

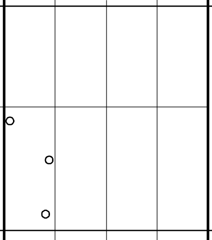
GOLDER
MEMBER OF WSP

Foundation Design

RECORD OF BOREHOLE No. WR-8

1 OF 1 **METRIC**

PROJECT 18103241
W.P. 5182-17-01
LOCATION N 5108366.6; E 248432.4 NAD83 MTM ZONE 12 (LAT. 46.112764; LONG. -81.729229)
ORIGINATED BY SA
DIST HWY 7041
BOREHOLE TYPE 150 mm O.D. Solid Stem Augers
COMPILED BY TR
DATUM GEODETIC
DATE June 27, 2019
CHECKED BY AC



SOIL PROFILE			SAMPLES								REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
181.8	GROUND SURFACE													
0.0	ASPHALT (100 mm)	■												
0.1 181.2	SAND (SP) and gravel (FILL) Red to brown Moist	☒												
	ASPHALT (130 mm)	■					181							
180.7 1.1	Gravelly SAND (SP), trace silt (FILL) Compact Brown Moist	☒	1A 1B	SS	19				○ ○					
180.0 1.8	SILTY SAND (SM), some gravel, trace clay (FILL) Compact Brown Moist to wet	☒	2	SS	10/0.10	∇			○		13	60	25	2

END OF BOREHOLE
AUGER AND SPLIT-SPOON
REFUSAL
(I.E. HAMMER BOUNCING)

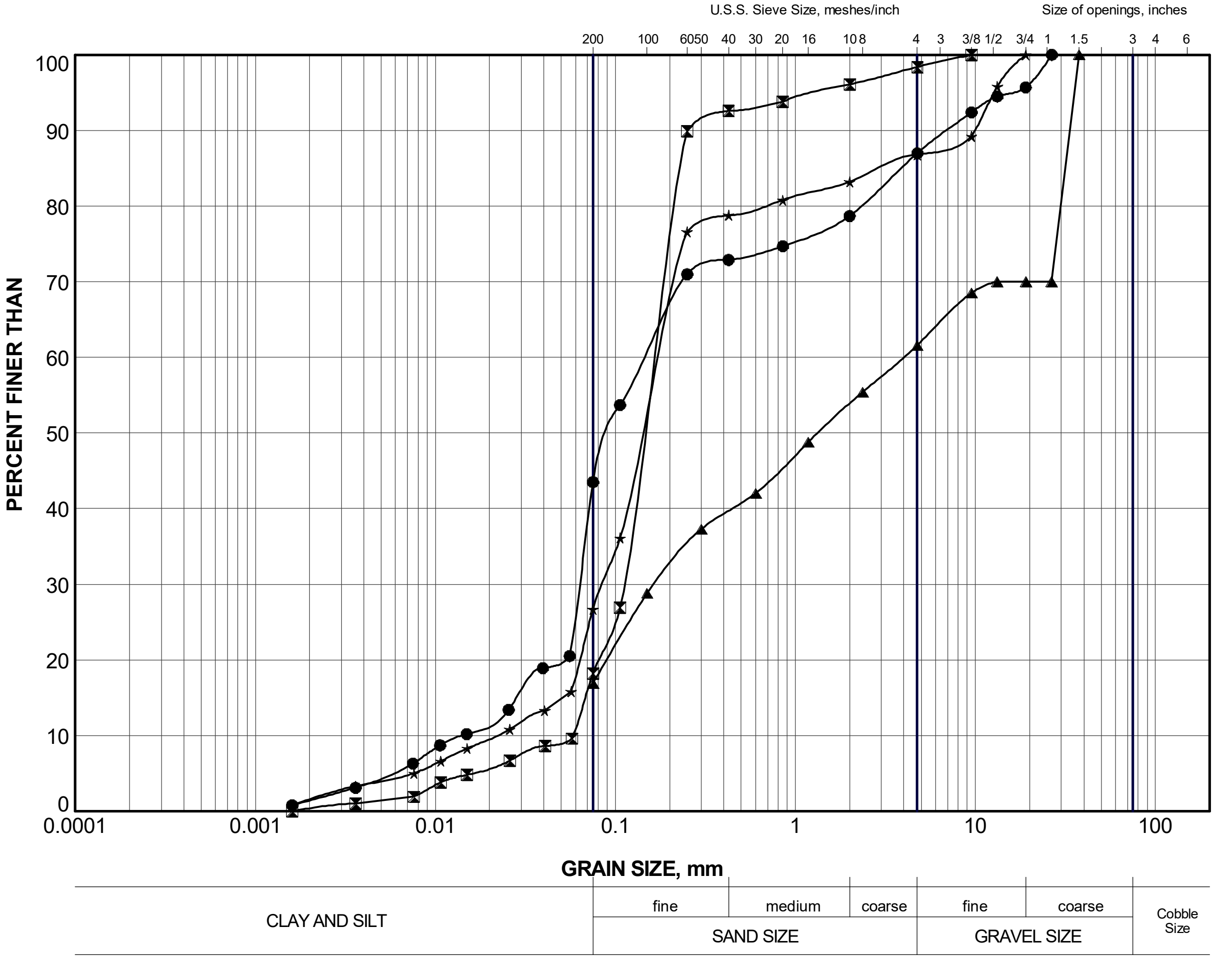
NOTE:

1. Water level at a depth of 1.8 m below ground surface (Elev. 180.0 m) upon completion of drilling.

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

APPENDIX B

Geotechnical Laboratory Test Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WR-3	2B	180.7
⧗	WR-4	3	180.3
▲	WR-7	2	180.2
★	WR-8	2	180.0

PROJECT

HIGHWAY 7041
WHITEFISH FALLS RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

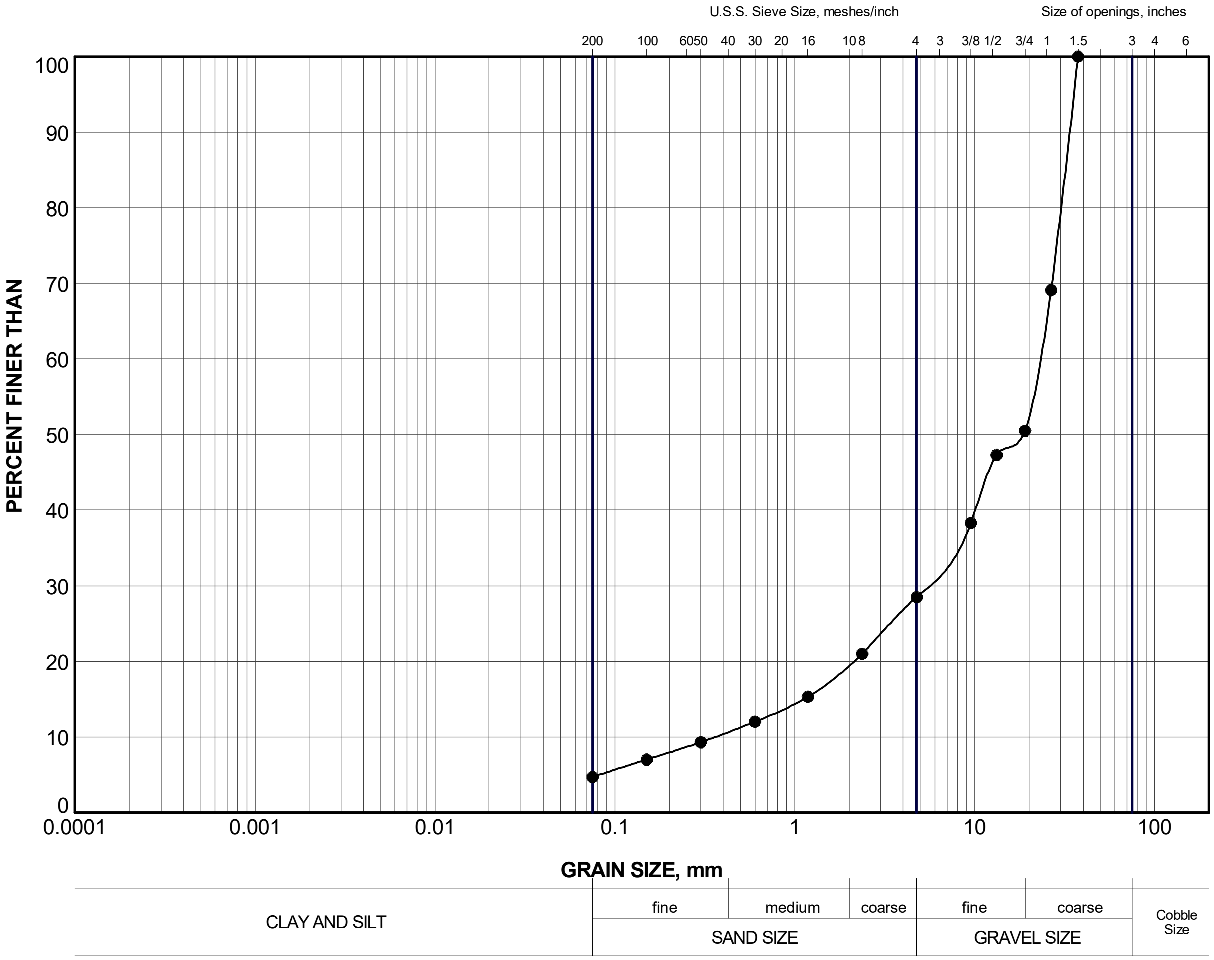
SILTY SAND (SM) to SILTY SAND (SM) and gravel (FILL)

GOLDER

SUDBURY, ONTARIO

PROJECT No.		18103241	FILE No.	18103241.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AC	Sep 2019	**FIGURE B-1**	
APPR	JMAC	Sep 2019		

SUD-MTO GSD GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WR-2	4	179.2

PROJECT

HIGHWAY 7041
WHITEFISH FALLS RIVER BRIDGE

TITLE

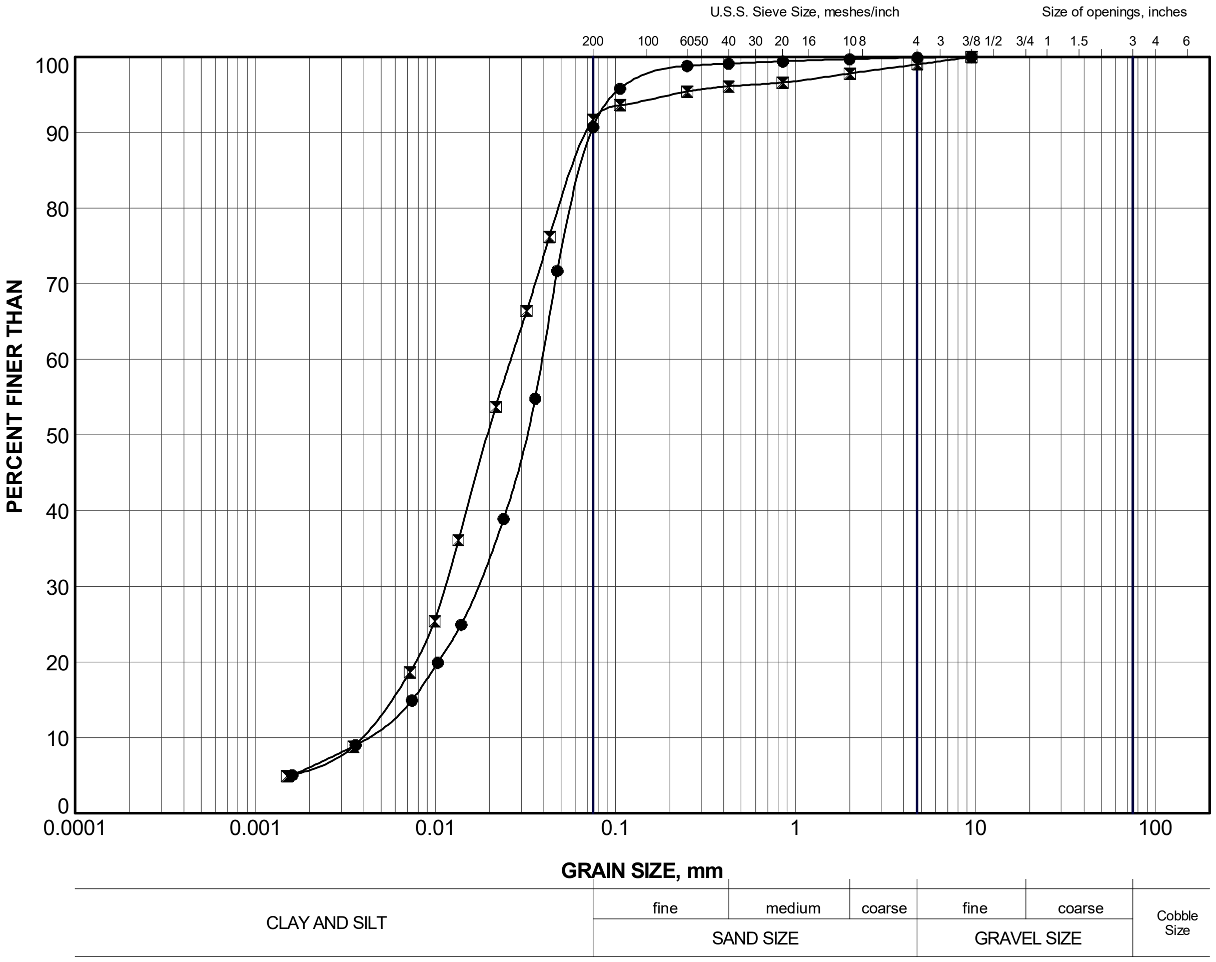
GRAIN SIZE DISTRIBUTION

Sandy GRAVEL (GP) (FILL)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		18103241	FILE No.	18103241.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AC	Sep 2019		
APPR	JMAC	Sep 2019	**FIGURE B-2**	

SUD-MTO GSD GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WR-1	2A	180.4
⧗	WR-6	5	177.5

PROJECT

HIGHWAY 7041
WHITEFISH FALLS RIVER BRIDGE

TITLE

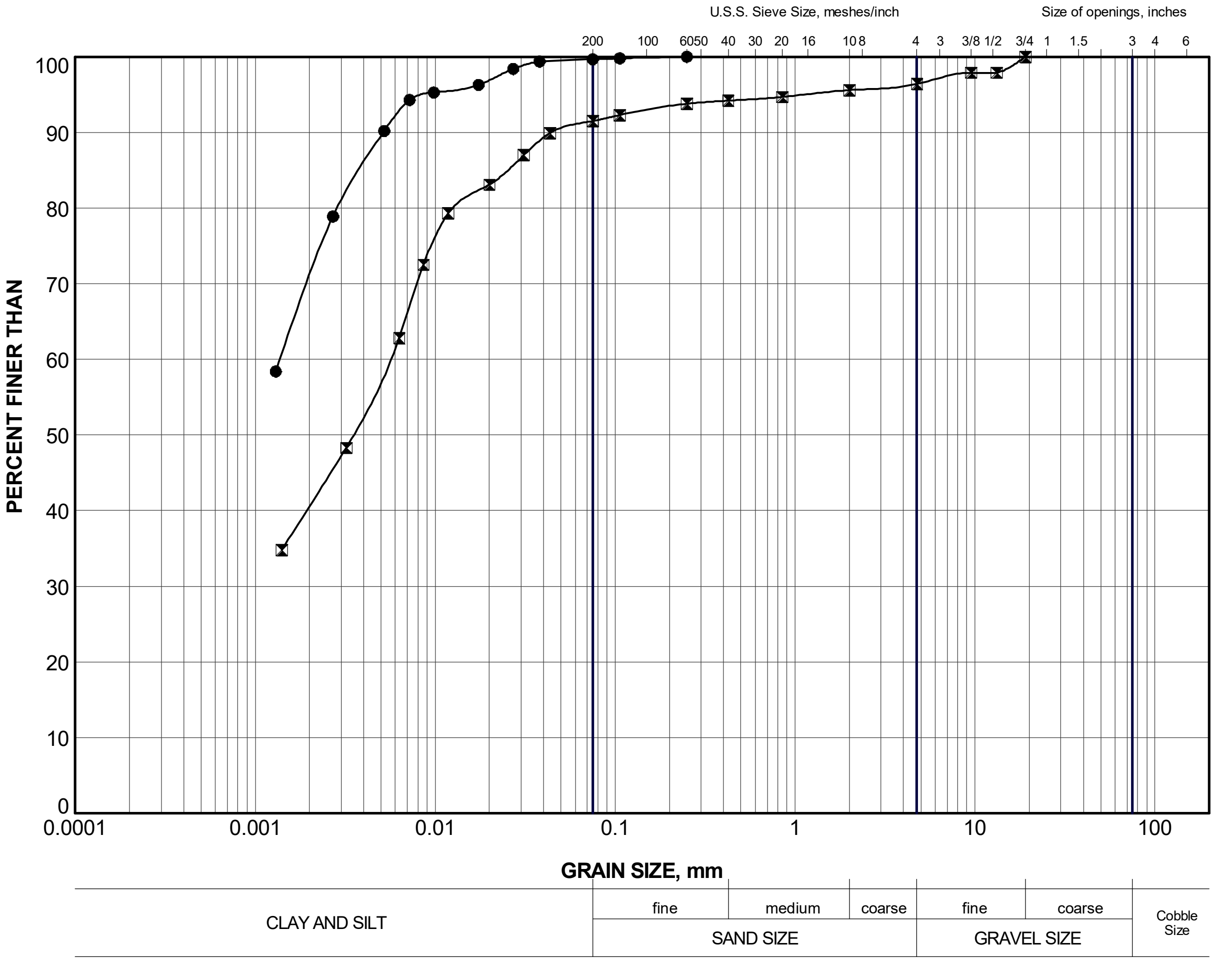
GRAIN SIZE DISTRIBUTION

SILT (ML)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		18103241	FILE No.	18103241.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AC	Sep 2019		
APPR	JMAC	Sep 2019	**FIGURE B-3**	

SUD-MTO GSD GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WR-1	3	179.1
⧗	WR-6	4A	178.3

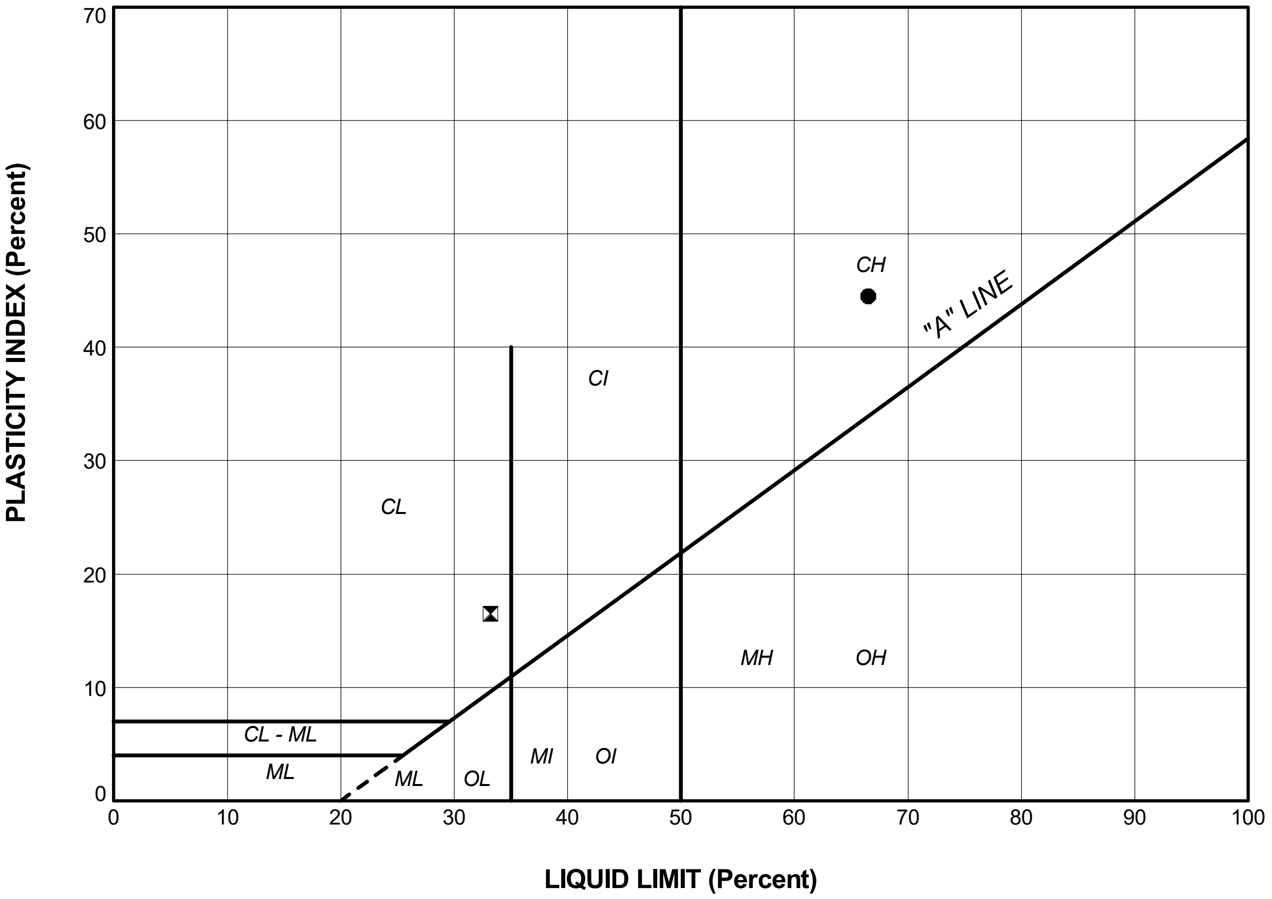
PROJECT
HIGHWAY 7041
WHITEFISH FALLS RIVER BRIDGE

TITLE
GRAIN SIZE DISTRIBUTION
CLAYEY SILT (CL) to CLAY (CH)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		18103241	FILE No. 18103241.GPJ
DRAWN	TR	Sep 2019	SCALE N/A REV.
CHECK	AC	Sep 2019	
APPR	JMAC	Sep 2019	**FIGURE B-4**

SUD-MTO GSD GLDR_LDN.GDT



SOIL TYPE	**PLASTICITY**
C = Clay	L = Low
M = Silt	I = Intermediate
O = Organic	H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WR-1	3	66.5	22.0	44.5
⧗	WR-6	4A	33.2	16.7	16.5

PROJECT

HIGHWAY 7041
WHITEFISH FALLS RIVER BRIDGE

TITLE

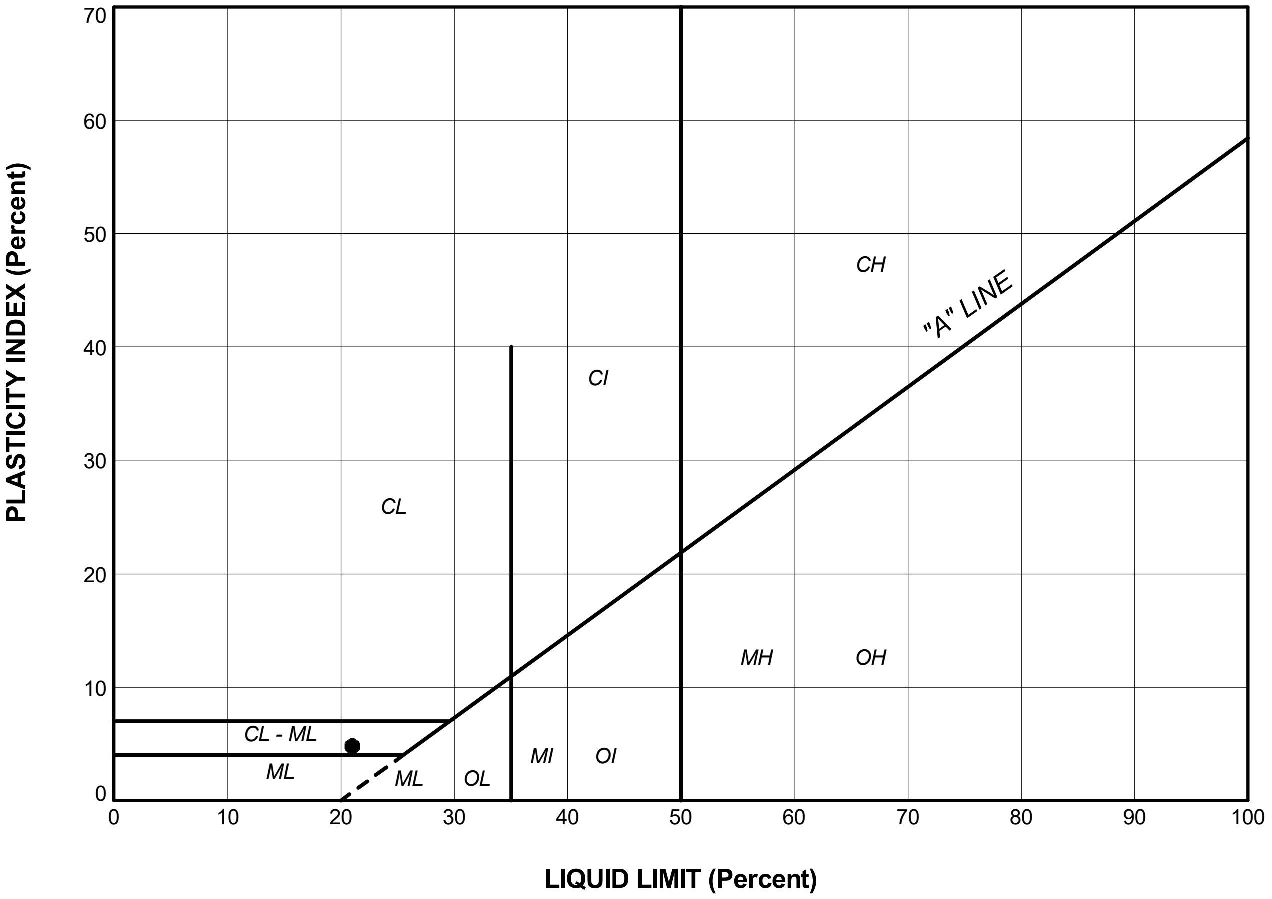
PLASTICITY CHART

CLAYEY SILT (CL) to CLAY (CH)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		18103241	FILE No.	18103241.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AC	Sep 2019	**FIGURE B-5**	
APPR	JMAC	Sep 2019		

SUD-MTO PL GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WR-1	5	21.0	16.2	4.8

PROJECT

HIGHWAY 7041
WHITEFISH FALLS RIVER BRIDGE

TITLE

PLASTICITY CHART

CLAYEY SILT - SILT (CL-ML)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		18103241	FILE No.	18103241.GPJ	
DRAWN	TR	Sep 2019	SCALE	N/A	REV.
CHECK	AC	Sep 2019	**FIGURE B-6**		
APPR	JMAC	Sep 2019			

Borehole WR-1

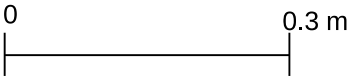
Elevation 175.8 m to 174.3 m

Borehole WR-2

Elevation 178.9 m to 175.9 m

Borehole WR-3

Elevation 179.0 m to 175.5 m



Approximate Scale

PROJECT

Highway 7041
Whitefish River Bridge

TITLE

Bedrock Core Photographs

GOLDER

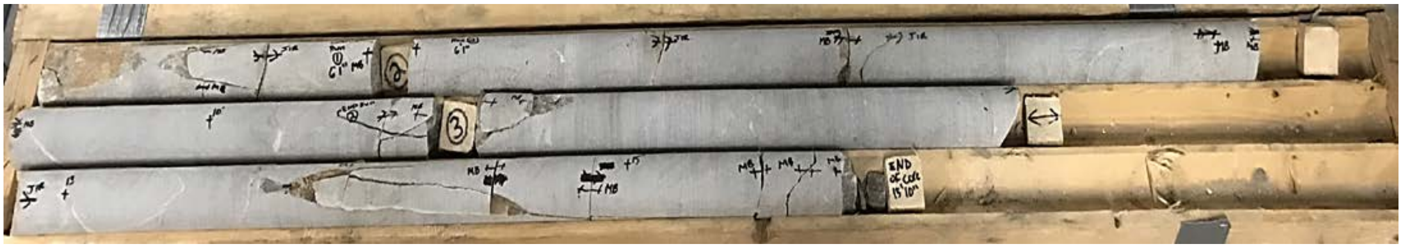
PROJECT No.	18103241		FILE No.	----
DESIGN	JGH	Sep 19	SCALE	NTS
CADD	-- --		REV.	
CHECK	AC	Sep 19		
REVIEW	JMAC	Sep 19		

FIGURE B-7A

Borehole WR-4

Elevation 180.2 m to 177.1 m

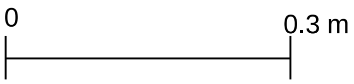
Borehole WR-5



Elevation 180.4 m to 177.0 m

Borehole WR-6

Elevation 177.2 m to 173.9 m



Approximate Scale

PROJECT	
Highway 7041 **Whitefish River Bridge**	

TITLE

Bedrock Core Photographs

GOLDER

PROJECT No.	18103241		FILE No.	----	
DESIGN	JGH	Sep 19	SCALE	NTS	REV.
CADD	-- --		**FIGURE B-7B**		
CHECK	AC	Sep 19			
REVIEW	JMAC	Sep 19			

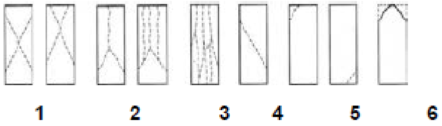
Golder Associates Ltd.
33 Mackenzie Street
Sudbury, Ontario, Canada P3C 4Y1
Telephone: (705) 524-6861
Fax: (705) 524-1984

SUMMARY OF ROCK CORE TEST DATA

PROJECT NO.: **18103241 phase2300**
PROJECT NAME: **MTO/5018-E-0009/Hwy 6 & 7041**
TYPE OF UNIT: **Rock Core**
TESTED BY: **EW**
DATE TESTED: **August 1, 2019**

GOLDER LAB NUMBER	**T1353**	**T1354**	**T1355**	**T1356**	~~**T1357**~~
BOREHOLE NUMBER:	**WR-2**	**WR-3**	**WR-4**	**WR-5**	~~**D-2**~~
SAMPLE NUMBER:	**N/A**	**N/A**	**N/A**	**N/A**	~~**N/A**~~
DEPTH OF TESTED CORE	11'	14'4"	8'4"	9'6"	~~34'8"~~
LENGTH AS CUT (mm)	104.0	103.0	103.0	101.0	~~101.0~~
DIAMETER (mm)	48.0	47.0	47.0	47.0	~~48.0~~
DENSITY (kg/m3)	2551	2686	2686	2682	~~2626~~
COMPRESSIVE STRENGTH (KN)	307.9	72.7	347.5	252.8	~~207.0~~
CORRECTED STRENGTH (MPa)	**170.1**	**41.9**	**200.3**	**145.7**	~~**114.4**~~
TYPE OF FRACTURE	**3**	**3**	**3**	**3**	~~**3**~~

Type of Fracture



COMMENTS:

Input by: JM
Reviewed by: TG

PROJECT	
Highway 7041 **Whitefish River Bridge**	

TITLE

Summary of Rock Core Test Data

PROJECT No.	18103241		FILE No.	----
DESIGN	JGH	Sep 19	SCALE	NTS
CADD	-- --		REV.	
CHECK	AC	Sep 19		
REVIEW	JMAC	Sep 19	**FIGURE B-8**	

APPENDIX C

Analytical Laboratory Test Results

Your Project #: 18103241
Site Location: HWY 7041 WHITEFISH FALLS BRIDGE
Your C.O.C. #: n/a

Attention: David Muldowney

Golder Associates Ltd
33 Mackenzie Street
Suite 100
Sudbury, ON
Canada P3C 4Y1

Report Date: 2019/07/16
Report #: R5799008
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: B9I1367
Received: 2019/07/02, 12:30

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	2	2019/07/08	2019/07/08	CAM SOP-00463	SM 4500-Cl E m
Conductivity	2	2019/07/09	2019/07/09	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 3)	2	2019/07/10	2019/07/11	BBY8SOP-00017	BCMOE BCLM Dec2000 m
Sulphide in Soil (1)	2	2019/07/10	2019/07/11	BBY6SOP-00052 BBY6SOP-00006	EPA-821-R-91-100 m
pH CaCl2 EXTRACT	2	2019/07/04	2019/07/04	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2019/07/03	2019/07/09	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2019/07/08	2019/07/09	CAM SOP-00464	EPA 375.4 m
Redox Potential (2, 4)	2	N/A	N/A		

Remarks:

Bureau Veritas Laboratories are accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by BV Labs are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in BV Labs profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and BV Labs in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

BV Labs liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. BV Labs has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by BV Labs, unless otherwise agreed in writing. BV Labs is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by BV Labs, results relate to the supplied samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
(1) This test was performed by Campo to Burnaby - Offsite
(2) This test was performed by Sub from Campo to Env. Testing Canada (Eurofins)
(3) Offsite analysis requires that subcontracted moisture be reported.
(4) Oxidation-Reduction Potential (ORP) values are determined using a Ag/AgCl reference electrode.

Your Project #: 18103241
Site Location: HWY 7041 WHITEFISH FALLS BRIDGE
Your C.O.C. #: n/a

Attention: David Muldowney

Golder Associates Ltd
33 Mackenzie Street
Suite 100
Sudbury, ON
Canada P3C 4Y1

Report Date: 2019/07/16
Report #: R5799008
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: B9I1367
Received: 2019/07/02, 12:30

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Alisha Williamson, Project Manager
Email: Alisha.Williamson@bvlabs.com
Phone# (613)274-0573
==
BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

RESULTS OF ANALYSES OF SOIL

BV Labs ID		KDY922	KDY923			KDY923		
Sampling Date		2019/06/26 08:45	2019/06/24 10:30			2019/06/24 10:30		
COC Number		n/a	n/a			n/a		
	UNITS	**WR-2**	**WR-4**	**RDL**	**QC Batch**	**WR-4 Lab-Dup**	**RDL**	**QC Batch**
CONVENTIONALS								
Sulphide	ug/g	<0.30	<0.30	0.30	6228145			
Calculated Parameters								
Resistivity	ohm-cm	2900	1600		6209285			
Inorganics								
Soluble (20:1) Chloride (Cl-)	ug/g	150	340	20	6216266	330	20	6216266
Conductivity	umho/cm	342	622	2	6218306	646	2	6218306
Available (CaCl2) pH	pH	7.28	6.78		6210747			
Soluble (20:1) Sulphate (SO4)	ug/g	<20	<20	20	6216267			
Physical Testing								
Moisture-Subcontracted	%	4.9	15	0.30	6228144			

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

TEST SUMMARY

BV Labs ID: KDY922
Sample ID: WR-2
Matrix: Soil

Collected: 2019/06/26
Shipped:
Received: 2019/07/02

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6216266	2019/07/08	2019/07/08	Deonarine Ramnarine
Conductivity	AT	6218306	2019/07/09	2019/07/09	Kazzandra Adeva
Moisture (Subcontracted)	BAL	6228144	2019/07/10	2019/07/11	Ailene Pagdonsolan
Sulphide in Soil	SPEC/UVVS	6228145	2019/07/10	2019/07/11	Ife Olotu
pH CaCl2 EXTRACT	AT	6210747	2019/07/04	2019/07/04	Neil Dassanayake
Resistivity of Soil		6209285	2019/07/09	2019/07/09	Anastassia Hamanov
Sulphate (20:1 Extract)	KONE/EC	6216267	2019/07/08	2019/07/09	Deonarine Ramnarine
Redox Potential	COND	6222461	2019/07/11		Katherine Szozda

BV Labs ID: KDY923
Sample ID: WR-4
Matrix: Soil

Collected: 2019/06/24
Shipped:
Received: 2019/07/02

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6216266	2019/07/08	2019/07/08	Deonarine Ramnarine
Conductivity	AT	6218306	2019/07/09	2019/07/09	Kazzandra Adeva
Moisture (Subcontracted)	BAL	6228144	2019/07/10	2019/07/11	Ailene Pagdonsolan
Sulphide in Soil	SPEC/UVVS	6228145	2019/07/10	2019/07/11	Ife Olotu
pH CaCl2 EXTRACT	AT	6210747	2019/07/04	2019/07/04	Neil Dassanayake
Resistivity of Soil		6209285	2019/07/09	2019/07/09	Anastassia Hamanov
Sulphate (20:1 Extract)	KONE/EC	6216267	2019/07/08	2019/07/09	Deonarine Ramnarine
Redox Potential	COND	6222461	2019/07/11		Katherine Szozda

BV Labs ID: KDY923 Dup
Sample ID: WR-4
Matrix: Soil

Collected: 2019/06/24
Shipped:
Received: 2019/07/02

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6216266	2019/07/08	2019/07/08	Deonarine Ramnarine
Conductivity	AT	6218306	2019/07/09	2019/07/09	Kazzandra Adeva

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	23.0°C

Sample KDY922 [WR-2] : Sample analyzed past method specified hold time for Sulphide in Soil. Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised. Sample was analyzed past method specified hold time for Total Sulphide.

Sample KDY923 [WR-4] : Sample analyzed past method specified hold time for Moisture. Sample analyzed past method specified hold time for Sulphide in Soil. Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised. Sample received past method specified hold time for Sulphide in Soil. Sample was analyzed past method specified hold time for Total Sulphide.

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**UNITS**	**Value (%)**	**QC Limits**
6210747	Available (CaCl2) pH	2019/07/04			100	97 - 103			0.92	N/A
6216266	Soluble (20:1) Chloride (Cl-)	2019/07/08	NC	70 - 130	101	70 - 130	<20	ug/g	3.4	35
6216267	Soluble (20:1) Sulphate (SO4)	2019/07/09	NC	70 - 130	98	70 - 130	<20	ug/g	8.4	35
6218306	Conductivity	2019/07/09			104	90 - 110	<2	umho/cm	3.8	10
6228144	Moisture-Subcontracted	2019/07/11					<0.30	%	4.8	20
6228145	Sulphide	2019/07/11	16 (1)	75 - 125	95	75 - 125	<0.50	ug/g	NC	30

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

(1) Recovery or RPD for this parameter is outside control limits. The overall quality control for this analysis meets acceptability criteria.

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Anastassia Hamanov, Scientific Specialist

Andy Lu, Ph.D., P.Chem., Scientific Specialist

David Huang, BBY Scientific Specialist

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam
A Bureau Veritas Group Company

6740 Campobello Road, Mississauga, Ontario L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266
CAM FCD-01191/4

CHAIN OF CUSTODY RECORD 127632

Page ____ of ____

Invoice Information		Report Information (if differs from invoice)	
Company Name:	Golder Associates	Company Name:	
Contact Name:	David Muldowney	Contact Name:	
Address:	33 Mackenzie St. Suite 100, Sudbury ON, P3C 4Y1	Address:	
Phone:	705-524-6861 Fax:	Phone:	Fax:
Email:	D_Muldowney@Golder.com	Email:	

Project Information (where applicable)	
Quotation #:	
P.O. #/ AFE#:	
Project #:	18103241
Site Location:	HWY 7041 Whitefish Falls
Site #:	Bridge
Site Location Province:	ON
Sampled By:	S. Albert

Turnaround Time (TAT) Required

☒ Regular TAT (5-7 days) Most analyses

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Surcharges will be applied)

☐ 1 Day ☐ 2 Days ☐ 3-4 Days

Date Required:

Rush Confirmation #:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Regulation 153

☐ Table 1 ☐ Res/Park ☐ Med/ Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/ Other
☐ Table ____

FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations

☐ CCME ☐ Sanitary Sewer Bylaw
☐ MISA ☐ Storm Sewer Bylaw
☐ PWQO Region ____
☐ Other (Specify) ____
☐ REG 558 (MIN. 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

LABORATORY USE ONLY

CUSTODY SEAL Y [✓ N] Present	Intact	COOLER TEMPERATURES
N		23, 23, 23°C
		3|3|3

COOLING MEDIA PRESENT: Y / [✓ N]

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	Corrosivity								HOLD- DO NOT ANALYZE	COMMENTS
1	WR-2	2019/06/26	8:45	Soil									X									
2	WR-4	2019/06/24	10:30	Soil									X									
3																						
4																						
5																						
6																						
7																						
8																						
9																						
10																						

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)
	2019/07/02	12:30	Bradley Frazier	2019/07/02	12:30
			Cole Gallagher T drive	2019/07/03	08:30

Received in Sudbury

02-Jul-19 12:30
Alisha Williamson
B9I1367
MRK ENV-1135

Unless otherwise agreed to in writing, work submitted on this Chain of Custody is subject to Maxxam's standard Terms and Conditions. Signing of this Chain of Custody document is acknowledgment and acceptance of our terms which are available for viewing at www.maxxam.ca/terms. Sample container, preservation, hold time and packages information can be viewed at http://maxxam.ca/wp-content/uploads/Ontario-COC.pdf.

COC-1004 (01/19)

White: Maxxam ~ Yellow: Client

APPENDIX D

Non-Standard Special Provisions and Notice to Contractor

DOWELS INTO ROCK - Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR THE SUPPLY, INSTALLATION AND TESTING OF DOWELS INTO ROCK

1.0 SCOPE

The work for the above noted tender item shall be in accordance with OPSS 904, including all Special Provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications, or publications:

ASTM International D1143M Standard Test Methods for Deep Foundations Under Static Axial Compressive Load

3.0 DEFINITIONS

For the purpose of this Special Provision, the following definitions apply:

Dowels into Rock means reinforcing steel dowels and non-shrink grout.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Working Drawings

The Contractor shall submit Working Drawings two weeks prior to construction to the Contract Administrator as follows:

a) All Working Drawings shall be sealed and signed by the design Engineer and design check Engineer

b) A plan illustrating the layout of the dowels

c) Detail drawing of the dowel into bedrock.

d) The method for constructing of the holes, maintaining the holes, and placing reinforcing steel dowels, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.

e) The procedures to verify hole length. Records of measurements that verify the hole length.

f) Records of all drilling procedures, rock conditions encountered, and installation times.

g) Test procedures for Dowels into Rock. Test results verifying the 28 day strength of non-shrink grout.

h) Drawings and design calculations for a suitable reaction system for the applied test loads.

i) Drawings and details for reference system arrangement.

j) Calibration curves for all gauges.

5.0 MATERIALS

5.01 Non-Shrink Grout

The non-shrink grout shall be an approved product from the MTO's Pre-Qualified Products List.

5.02 Anti-Washout Agent

The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock.

5.03 Manufacturer Information

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

a) Data sheets for the non-shrink grout and anti-washout agent,

b) Technical information that proves that the non-shrink grout and anti-washout agent are compatible, and

c) Installation procedures.

5.04 Steel Dowels

Steel dowels shall conform to the requirements of OPSS 905.

6.0 EQUIPMENT

All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment shall not cause damage to the reinforcing steel dowels.

7.0 CONSTRUCTION

7.01 General

The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock. The Contractor shall conduct the specified acceptance tests under the direction of the Contractor's Engineer.

The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

7.02 Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

7.03 Construction of Holes

The sides and end of the hole shall not be disturbed. The Contractor shall construct the holes, maintain the holes, and place reinforcing steel dowels, grout and other materials in the holes.

The hole diameters and hole length for this project are as specified on the Contract Drawings.

At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit these records to the Contract Administrator upon completion of the work.

7.04 Installation of Reinforcing Steel Dowels

Reinforcing steel dowels shall be installed in strict accordance with the Contract Drawings and installation procedures.

Centering devices shall be provided to ensure that the reinforcing steel dowels are located centrally in the hole.

Dowels shall extend into fair quality bedrock (i.e., RQD > 50%).

7.05 Grout and Anti-Washout Agent

The non-shrink grout shall entirely fill the annular space between the reinforcing steel dowels and side for the dowel hole.

Anti-washout agent shall be used in accordance with the specifications of the manufacturer.

7.06 Installation of Dowels

Upon completion of installation for each group of dowels, the Contractor shall submit to the Contract Administrator a Request to Proceed. The testing shall be conducted by the Contractor and inspected by the Contract Administrator.

The next operation after the completion of testing shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

8.0 QUALITY ASSURANCE

In each group, 10% of the dowels rounded up to the next whole number, but no fewer than two dowels, shall be tested.

8.01 General Testing Requirements

The testing of Dowels into Rock shall be inspected by a Foundation Engineering Specialist retained by the Contract Administrator. The Contractor shall refer to the Contract Drawings for specific test details.

The Contractor shall supply materials and equipment to conduct the tests for the dowels into rock. The equipment and materials shall be capable of stressing the dowels into rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements.

The Contract Administrator shall supervise the testing of the Dowels into Rock. The Contractor shall notify the Contract Administrator of the testing schedule at least 10 working days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted as scheduled by the Contract Administrator.

8.01.01 Testing Equipment

The Dowels into Rock will be carried out generally in accordance with the prevailing requirements of ASTM International D1143M superseded where applicable by the procedures specified in the Contract Drawings.

Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel dowels or stressing system.

The Contractor shall construct suitable enclosures to provide complete protection for all equipment from variations in the weather conditions and disturbances.

8.01.02 Testing for Dowels into Rock, and Report

At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel dowels, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit the above noted records to the Contract Administrator.

The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. The precision of gauges and the minimum ram dimension of the jack shall be as per the Contract Drawings. The Contractor shall submit details for current calibration and curves for all gauges to the Contract Administrator.

8.01.03 Testing Loading

The testing procedures and loading shall be in accordance with the requirement and specifications in Contract Drawings.

8.02 Acceptance Criteria

The following acceptance criteria apply:

a) The inspection of dowels shall be carried out by the Contractor's Engineer in advance of the installation of Dowels into Rock.

b) The Contractor shall submit the Request to Proceed to the Contract Administrator for the acceptance of the Dowels into Rock.

9.0 MEASUREMENT FOR PAYMENT

For measurement purposes, a count shall be made of the number of dowels installed.

10.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work.

WARRANT: Use only in consultation with Regional Structural Section with the above non-standard tender item.

DEWATERING STRUCTURE EXCAVATIONS – Item No.

Special Provision No. FOUN0003

Amendment to OPSS 902, November 2010

902.02 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 517	Dewatering
OPSS 805	Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

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

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

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

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

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 5 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

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

902.04.02 Submission Requirements

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

902.04.02.01 Preconstruction Survey

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

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

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

902.04.02.02 Working Drawings

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

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Subsection 902.07.04 of OPSS 902 is amended by the addition of the following clauses:

902.07.04.01 General

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

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

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

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

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

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

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

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

902.07.04.05 Removal

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

NOTICE TO CONTRACTOR – Subsurface Conditions

Special Provision

The Contactor is hereby notified that the existing embankment fill at the Whitefish River Bridge site contains cobble and boulder obstructions as encountered and confirmed by coring and as inferred to be present due to instances of split-spoon refusal. The extent and depth of the cobble and boulder obstructions may vary beyond and between the borehole locations.

The Contractor is further notified the bedrock surface elevations at the Whitefish River Bridge site are variable. The magnitude and extent of the variations in the bedrock surface may vary beyond and between the borehole locations.

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