July 22, 2016

ADDENDUM PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

REPLACEMENT OF GROUNDHOG RIVER BRIDGE, SITE 39W-093 AND STRUCTURAL CULVERT WEST OF BRIDGE
HIGHWAY 11 AT FAUQUIER
COCHRANE DISTRICT, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5049-07-00

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REPORT

Table of Contents

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

REFERENCES

LIST OF SYMBOLS AND ABBREVIATIONS

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

TABLES

APPENDICES

PART A

ADDENDUM PRELIMINARY FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement of the Groundhog River Bridge (Site 39W-093), located on Highway 11 in Fauquier, Cochrane District, Ontario. Golder completed a preliminary foundation investigation at the abutments for the proposed replacement of the Groundhog River Bridge, as reported in MTO GEOCRES No. 42 G-40 (Golder 2013), and this current addendum report should be read in conjunction with the Foundation Investigation Report referenced above. This current report presents the results of the addendum preliminary foundation investigation carried out at the pier locations for the proposed bridge replacement, as well as for the replacement of the structural culvert west of the bridge at Station 10+926.

The Terms of Reference and the Scope of Work for the foundation engineering services are outlined in MTO's Request for Proposal dated June 2011. Golder's Change Request 2 is associated with the additional work relating to the proposed piers for the Groundhog River Bridge replacement and the structural culvert at STA10+926. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundations engineering services for this project, dated December 2011. The Base Plan (General Arrangement Drawing) showing the alignment of Groundhog River Bridge was provided to Golder by URS in April 2013.

2.0 SITE DESCRIPTION

The Groundhog River Bridge carrying Highway 11 is situated immediately to the west of Fauquier, Ontario in Cochrane District. The surrounding land is generally flat, but slopes down towards the river, with a boat launch located on the east shore and residential development to the east of the boat launch (beyond 100 m east of the bridge). On the west side of the river, the topography is also generally flat-lying, with moderate tree cover and a bedrock outcrop west of the tree cover (beyond 100 m west of the bridge). The Ontario Northland Railway (ONR) Bridge is located to the south and parallels the Groundhog River Bridge. The Groundhog River flows in a northerly direction and is approximately 150 m wide and up to about 4 m deep at the bridge location at the time of the addendum foundation investigation. The river water level was surveyed at Elevation 212.0 m by Golder during the field investigation in August 2015 and was surveyed at Elevation 213.1 m on April 15, 2012, by Callon Dietz Inc. (Callon Dietz) under subcontract to URS. The existing Highway 11 Bridge was constructed in 1939 and consists of a 10 m wide, 180 m long four-span structure, comprised of two 76 m long steel trusses and two 14 m long concrete approach slabs.

In general, the topography in the vicinity of the proposed structural culvert at STA 10+926, located about 400 m to the west of the Groundhog River Bridge, consists of a low lying swampy area with a creek flowing northerly below the existing Highway 11 embankment and through a swamp west of the proposed culvert. A bedrock outcrop is present to the east of the swamp. The creek water level was surveyed by Golder at the existing culvert at Elevation 215.9 m on November 18, 2014.

3.0 INVESTIGATION PROCEDURES

The fieldwork for this current subsurface investigation was carried out between November 12 and November 19, 2014, and between August 29 and September 2, 2015. During the current investigation, Borehole GHR-1B was advanced at the proposed east abutment to core bedrock immediately adjacent to Boreholes GHR-1 and GHR1-A advanced during the original investigation, Boreholes 15-1 to 15-3 were advance at the proposed pier locations, and Boreholes GHR-7, GHR-7a, GHR-8, GHR-9, GHR-10 and GHR-10a were advanced at the proposed structural culvert, at the approximate locations shown on Drawings A1 and C1 in Appendix A and C.

The boreholes for this addendum foundation investigation were advanced using a D-25 tuck-mounted drill rig supplied and operated by Walker Drilling Ltd. of Barrie, Ontario, portable equipment supplied and operated by Landcore Drilling Inc. of Chelmsford, Ontario, and a CME 55 LC track-mounted drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville-Sur-La-Rouge, Quebec. The boreholes were advanced through the overburden using HW casing, NW casing and/or 108 mm inner diameter continuous flight hollow stem augers to refusal or competent stratum. In general, soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler driven by an automatic or cathead hammers, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Samples of the cobbles and boulders encountered in places and the bedrock in selected boreholes were cored using a NQ size core barrel. The boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The groundwater conditions were observed in the open boreholes immediately following the drilling operations and are described on the Record of Borehole sheets of the current and previous investigations at the bridge site, and at the culvert site are provided in Appendix A and B, and C, respectively.

The fieldwork was supervised on a full-time basis by a member of Golder's staff who located the boreholes in the field, directed the drilling, sampling and in situ testing operations and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder's Sudbury Laboratory for further examination and laboratory testing. Index and classification tests consisting of water content, Atterberg limits and grain size distribution were carried out on selected soil samples. Uniaxial Compression Strength (UCS) tests were carried out on select samples of the bedrock core. The geotechnical laboratory testing was completed according to applicable MTO LS standards.

The borehole locations and elevations were measured in the field by Golder personnel relative to the stakes installed by Callon Dietz. The borehole locations (referenced to the MTM NAD83 co-ordinate system), ground surface elevations (referenced to Geodetic datum) and borehole depths are presented on the Record of Borehole sheets in Appendices A, B and C, and are shown on Drawing A1 for the current and previous investigation and on Drawing C1 for the culvert site in Appendices A and C, respectively, and are summarized below.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground/Water Surface Elevation (m)	Borehole Depth (m)
GHR-1B	5464439.1	229162.0	216.9	13.4
15-1	5464491.5	229023.4	212.0*	19.8
15-2	5464470.0	229073.2	212.0*	15.6
15-3	5464450.1	229124.0	211.9*	6.5
GHR-7	5464641.6	228612.9	223.8	11.3
GHR-7Aa	5464640.0	228614.8	223.8	15.8
GHR-8	5464641.4	228625.2	223.7	15.6
GHR-9	5464656.6	228638.3	216.1	3.7
GHR-10	5464672.8	228664.2	216.0	3.4
GHR-10A	5464672.8	228665.6	216.0	8.2

*Water surface; borehole depth includes between 2.2 m and 4.0 m water column

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on NOEGTS[1] mapping, the subsoils in the vicinity of the Groundhog River Bridge site are characterized as an alluvial plain deposit consisting of silty soils; whereas, the subsoils in the vicinity of the structural culvert consist of clayey till soils.

In both areas, based on geological mapping by the Ministry of Natural Resources[2], the sites are underlain by bedrock of the Early Precambrian era consisting of granitic, metasedimentary or minor metavolcanic migmatite.

4.2 General Overview of Local Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during these investigations, together with the results of the laboratory tests carried out on selected soil and rock core samples, are presented on the Record of Borehole and Drillhole sheets and the laboratory test sheets in the respective appendices. The stratigraphic boundaries shown on the Record of Borehole and Drillhole sheets are inferred from non-continuous sampling, observations of drilling progress and in situ testing and are approximate. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

Boreholes GHR-1, GHR-1A and GHR-2 were advanced during the original investigation and the soil stratigraphy and laboratory testing are presented in the foundation investigation report, GEOCRES No. 42G-40 (Golder, 2013). The Record of Borehole sheets for these three boreholes and the laboratory test result figures are included in Appendix B for reference but the stratigraphy and laboratory tests for these three boreholes are not discussed further in this addendum report.

From the riverbed at the three pier locations of the proposed piers, the soil stratigraphy consists of a deposit of silt to sand underlain by a cohesive deposit, which in turn is underlain by a sequence of silt and sand, silt and sand, till, silt and sand and gravel deposits or pockets in places. Bedrock was cored in two pier holes as well as at the east abutment borehole.

At the site of the structural culvert at STA 10+926, existing embankment fill or peat is underlain by a silt and sand till deposit.

Detailed descriptions of the subsurface conditions at the investigated piers and culvert areas are provided in the following sections of this report. Groundwater and river/creek water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

4.3 Groundhog River Bridge

A total of four boreholes were completed for the proposed bridge: one borehole at each bridge pier (Boreholes 15-1 to 15-3); and one borehole at the proposed east abutment to core bedrock (Borehole GHR-1b). The interpreted stratigraphy at the borehole locations from both the original and current investigation is shown in profile on Drawing A1.

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[1] Northern Ontario Engineering Geology Terrain Study, Digital Maps, Ontario Geological Society Map Reference Number 42GSE.

[2] Ministry of Natural Resources, Geological Highway Map, Ontario Geological Survey, Map 2440.

4.3.1 Water

The water surface in Groundhog River measured at the time of drilling Boreholes 15-1, 15-2 and 15-3 in August 2015 is Elevations 211.9 m and 212.0 m, and the depth of water at the boreholes is between 2.2 m and 4.0 m.

4.3.2 Sandy Silt to Sand

A 0.7 m and 0.2 m thick deposit of grey, wet sandy silt to silty sand to sand was encountered at the riverbed in Boreholes 15-1 and 15-2 at Elevations 208.0 m and 209.5 m, respectively.

Two SPT ‘N’-values measured within the deposit are 6 blows and 26 blows per 0.3 m of penetration, indicating a loose to compact relative density.

4.3.3 Clayey Silt to Gravelly Silty Clay

A deposit of grey, wet, clayey silt, sandy clayey silt, silty clay and gravelly silty clay was encountered below the sandy silt to sand deposit in Boreholes 15-1 and 15-2. The top of the deposit was encountered at Elevations 207.3 m and 209.3 m, and the thicknesses of the deposit is 4.4 m and 3.5 m in Boreholes 15-1 and 15-2, respectively.

SPT ‘N’-values measured in the cohesive deposit range from 4 blows to 57 blows per 0.3 m of penetration. In situ field vane testing carried out in the cohesive material measured undrained shear strengths ranging between 72 kPa and greater than 100 kPa, with sensitivities of between 3 and 4. The in situ vane test result, together with the SPT ‘N’-values, suggest that the cohesive deposit has a generally firm to very stiff consistency.

The natural moisture content measured on three samples of the deposit range between about 11 per cent and 36 per cent.

Atterberg limits testing was carried out on three selected samples of the cohesive deposit and measured liquid limits between 21 per cent and 43 per cent, plastic limits between 13 per cent and 20 per cent, and plasticity indices between 9 per cent and 24 per cent. These results, which are plotted on a plasticity chart on Figure A1 in Appendix A, indicate that the deposit consists of clayey silt to silty clay of low plasticity to intermediate plasticity.

One grain size distribution test carried out a sample of the deposit is shown on Figure A2 in Appendix A.

4.3.4 Silt and Sand

A deposit of grey, wet, silt and sand was encountered below the cohesive deposit in Boreholes 15-1 and 15-2. The top of the deposit was encountered at Elevations 202.9 m and 205.8 m, and the thickness of the deposit is 6.3 m and 5.0 m in Boreholes 15-1 and 15-2, respectively. Cobbles, ranging in thickness from about 0.1 m to 0.2 m, were encountered within the silt and sand deposit below depths of 14.5 m and 7.7 m in Boreholes 15-1 and 15-2, respectively.

SPT ‘N’-values measured in the non-cohesive deposit range from 6 blows to 26 blows per 0.3 m of penetration, indicating a loose to compact relative density; and SPT ‘N’-value of 8 blows per 0.05 m of penetration was recorded at the top of the gravel seam or from an inferred cobble.

The natural moisture content measured on two samples of the deposit are 10 per cent and 11 per cent.

Grain size distribution tests carried out on two samples of the silt and sand deposit are shown on Figure A3 in Appendix A.

4.3.5 Silt and Sand (Till)

A deposit of grey, wet, silt and sand till was encountered below the silt and sand deposit in Borehole 15-2. The top of the deposit was encountered at Elevation 200.8 m, and the thickness of the deposit is 4.4 m. An approximately 0.1 m thick silt seams/layer was noted below a depth of 14.3 m in the borehole.

SPT ‘N’-values of 99 blows per 0.3 m of penetration and 134 blows per 0.25 m of penetration were measured within the till deposit, indicating a very dense relative density.

4.3.6 Silty Sand and Gravel to Sand and Gravel

A 0.9 m thick deposit of grey, wet, silty sand and gravel to sand and gravel was encountered below the silt and sand deposit in Borehole 15-1 and at the riverbed at Borehole 15-3. The top of the deposit was encountered at Elevations 196.6 m and 209.7 m in Boreholes 15-1 and 15-3, respectively.

Two SPT ‘N’-values of 13 and 30 blows per 0.3 m of penetration were measured within the deposit, indicating a compact relative density. One SPT ‘N’-value of 140 blows per 0.08 m of penetration was measured at the bottom of the deposit at the contact with the underlying bedrock.

4.3.7 Bedrock

In Boreholes 15-1, 15-3 and GHR-1B, bedrock was encountered at depths between 3.1 m and 16.3 m below ground/water surface, with the surface of the bedrock ranging from Elevation 208.8 m to Elevation 195.7 m. The bedrock was cored for lengths between 3.2 m and 3.5 m and the retrieved bedrock core is described as medium to very coarse grained, slightly weathered to fresh, pink to grey, granitic gneiss to gneiss. Photographs of the retrieved bedrock core are presented on Figures A4 and A5 in Appendix A.

The Total Core Recovery (TCR) from the recovered bedrock core is between 94 per cent and 100 per cent. The Rock Quality Designation (RQD) measured on the recovered core typically ranges between 45 per cent and 100 per cent, indicating a rock mass of poor to excellent quality as per Table 3.10 in the Canadian Foundation Engineering Manual (CFEM, 2006)[3]; RQD values of 0 per cent and 27 per cent were recorded in Borehole GHR-1B below 12.0 m depth, indicating that the recovered rock core at this depth is of very poor quality.

Laboratory Unconfined Compressive Strength (UCS) tests were carried out on two samples of the bedrock core from Boreholes 15-1 and 15-3 and yielded a uniaxial compressive strengths of 88 MPa and 103 MPa. The UCS values presented on the Record of Drillhole sheets in Appendix A indicate that the bedrock is strong to very strong (R4 50<USC<100MPa to R5, 100<USC< 250 MPa) as per Table 3.5 in CFEM (2006)[3].

4.3.8 Groundwater Conditions

The groundwater level in Borehole GHR-1B upon completion of bedrock coring was measured at a depth of 1.3 m below ground surface corresponding to Elevation 215.6 m.

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

4.4 Culvert at STA 10+926

A total of six boreholes (Boreholes GHR-7 to GHR-10, GHR-7A and GHR-10A) were completed for the structural culvert at STA 10+926: two boreholes were advanced at the proposed inlet and outlet of the culvert, and two boreholes were advanced along the proposed culvert alignment. Due to equipment limitations during an initial phase of the investigation in 2014, three boreholes were advanced to greater depths during the subsequent investigation in 2015 to aid in interpreting the soil stratigraphy along the proposed culvert alignment, as shown in plan and profile on Drawing C1.

4.4.1 Embankment Fill

In Boreholes GHR-7 and GRH-8, which were advanced through the southbound lane and shoulder of Highway 11, a 100 mm and 65 mm layer of asphalt was encountered from pavement surface at Elevation 223.8 m and 223.7 m, respectively.

Borehole GHR-7 penetrated embankment fill 8.6 m thick, consisting of a 1.3 m thick layer of sand and gravel, a 4.2 m thick layer of clayey silt and a 3.1 m thick layer of sand to sand and gravel. In Borehole GHR-8, the embankment fill is 9.1 m thick and consists of a 5.4 m thick layer of silty sand and gravel to sand and a 3.6 m thick layer of cobbles and boulders intermixed within a sand and gravel matrix.

SPT ‘N’-values measured within the non-cohesive fill range from 2 blows to 35 blows per 0.3 m of penetration, indicating a very loose to dense relative density. In Borehole GHR-8, the cobbles and boulders layer was cored, indicating boulder sites up to about 1 m; and an SPT ‘N’-value of 12 blows per 0.3 m of penetration was measured in the sand and gravel matrix indicating a compact relative density. SPT ‘N’-values measured within the cohesive fill layer range from 8 blows to 29 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

The natural moisture content measured on three samples of the non-cohesive fill ranges between 3 per cent and 9 per cent, whereas, results of a the moisture content measured on one sample of the cohesive fill is 17 per cent.

The results of a grain size distribution test carried out on one sample of the non-cohesive fill and on one sample of the cohesive fill are shown on Figures C1 and C2, respectively, in Appendix C.

An Atterberg limits test was carried out on one selected sample of the cohesive fill and measured a liquid limit of about 26 per cent, a plastic limit of about 15 per cent and a plasticity index of about 11 per cent. The result is plotted on a plasticity chart on Figure C3 in Appendix C, and indicates that the cohesive fill consists of clayey silt of low plasticity.

4.4.2 Peat

A deposit of dark brown, wet, amorphous, sandy peat to peat was encountered below the embankment fill in Boreholes GHR-7 and GHR-8, as well as from surface in Boreholes GHR-9, GHR-10 and GHR-10A. The top of the deposit was encountered between Elevations 216.1 m and 214.6 m, and the thicknesses of the deposit ranges between 0.1 m and 1.2 m.

SPT ‘N’-values measured in the deposit range from 0 blows (i.e. weight of hammer) to 14 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency.

The natural moisture content measured on two samples of the deposit are about 174 per cent and 320 per cent.

4.4.3 Sand and Gravel

A 1 m thick deposit/pocket of wet, brown to grey sand and gravel was encountered below the peat deposit in Borehole GHR-9 at Elevation 214.9 m.

One SPT 'N'-value measured in the deposit is 9 blows per 0.3 m of penetration, indicating a loose relative density.

4.4.4 Sandy Silt to Silt and Sand (Till)

A till deposit comprised of grey, wet sandy silt to silt and sand to gravelly silty sand was encountered in Boreholes GHR-7 to GHR-10 between Elevations 215.0 m and 213.9 m. The boreholes were terminated within the till deposit after exploring the deposit for a thickness ranging from 1.5 m to 6.4 m. Boreholes GHR-7A, GHR-8 and GHR-10A were advanced to further explore the deposit at these locations to depths between 8.2 m and 15.8 m below ground surface, respectively, to Elevations between 208.1 m and 207.8 m.

SPT 'N'-values measured in the till deposit range from 18 blows to 141 blows per 0.3 m of penetration, and up to 50 blows per 0.08 m of penetration, indicating a compact to very dense relative density. Core samples of the till deposit were recovered in Borehole GHR-10 to check for the presence of cobbles and boulders given the very dense relative density of the deposit; however, such materials were not noted in the core samples.

The natural moisture content measured on six samples of the deposit range between 10 per cent and 13 per cent.

The results of grain size distribution tests carried out on six samples of the till deposit are shown on Figure C4 in Appendix C.

Atterberg limits testing was carried out on four selected samples of the sandy silt to sand and gravel till and measured liquid limits between about 16 per cent and 18 per cent, plastic limits between about 11 per cent and 12 per cent and plasticity indices between about 4 per cent and 6 per cent. These results, which are plotted on a plasticity chart on Figure C5 in Appendix C, indicate that the fines of the till deposit consists of silt of slight plasticity. One Atterberg Limits test indicates the material to be non-plastic.

4.4.5 Groundwater Conditions

The groundwater level was measured in the open boreholes upon completion of drilling at depths between 0.1 m and 7.8 m below ground surface, between Elevations 216.5 m and 212.6 m.

The groundwater levels as encountered in the boreholes may not be representative of static levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.

5.0 CLOSURE

The field drilling program was supervised by Mr. Matt Thibeault, P.Eng. and this Addendum Preliminary Foundation Investigation Report was prepared by Mr. Matt Thibeault, P.Eng. and reviewed by Mr. Andre Bom, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact and a Senior Consultant with Golder, carried out an independent quality control review of this report.

Report Signature Page

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PART B

ADDENDUM PRELIMINARY FOUNDATION INVESTIGATION REPORT
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

Golder completed a preliminary foundation investigation at and provided foundation design recommendations for the abutments for the proposed replacement of the Groundhog River Bridge as reported in MTO GEOCRES No. 42G-40 (Golder 2013). This current (addendum) report should be read in conjunction with the Foundation Design Report referenced above. This section of the addendum report provides recommendations for the preliminary foundation design of the proposed piers for the replacement bridge (Section 6.2) and for the proposed replacement of the structural culvert at STA 10+926 (Section 6.3). The preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the proposed piers and culvert. Further investigation and analysis will be required during the detail design phase of the project.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project. Those requiring information on construction aspects should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Groundhog River Bridge Piers

6.2.1 Foundation Options

The existing four-span Groundhog River Bridge was constructed in 1939 and the abutments and piers are likely supported on shallow foundations as shown on the General Arrangement drawing dated May 1987, for the rehabilitation of the existing bridge. Due to the age and poor condition of the existing bridge, replacement will be required. We understand that the new structure will be skewed to the north of the existing bridge, with the center of the new east and west abutments located about 16 m and 30 m from the center of existing east and west abutments, respectively.

The proposed finished grade for the new Highway 11 alignment as provided by URS is Elevation 224.9 m at the east abutment and Elevation 224.0 m at the west abutment. The new east and west approach embankments will be up to approximately 8 m and 9 m high, respectively, relative to the existing natural ground surface at the abutments, and up to about 12 m above the Groundhog River water level (approximately Elevation 213.1 m) surveyed in April 2012 by Callon Dietz. The General Arrangement drawing provided by URS in April 2013 indicates that the new 190 m long four-span structure will consist of two 55 m long middle spans and two 40 m long outer spans, with the three piers located in the Groundhog River.

In Golder’s 2013 Foundation Design Report, it is recommended that the proposed abutments be supported on deep foundations consisting of steel H-piles. The piles would be driven to refusal within the cobbles and boulders deposit at the east abutment and to bedrock at the west abutment. However, based on the results of Borehole GHR-1B advanced during the current investigation, the steel H-piles at the east abutment will be founded either within the cobbles and boulders deposit overlying bedrock or on bedrock.

Based on the subsurface conditions encountered at the piers during the current investigation, we recommend that the east pier be supported by shallow foundations constructed on bedrock, the west pier be supported on deep foundations driven to bedrock and the middle pier be supported by deep foundations driven to within the sand and silt till deposit. Table 1 following the text of this report presents a comparison of the following deep foundation options based on advantages, disadvantages, risks and relative costs:

- Driven and socketted steel H-piles;
- Driven and socketted steel pipe (tube) piles; and
- Caissons, which are not generally constructed in Northern Ontario due to constructability issues associated with socketing the large diameter caissons within the strong bedrock. Tremie concrete construction methods would be required.

The following sections provide recommendations for shallow foundations at the east pier and deep foundation options for the west and middle piers. From a foundations perspective, driven steel H-piles at the west and middle pier locations are recommended.

As discussed in Section 6.4, additional boreholes should be advanced during the detail design phase of the project at the location of the proposed foundation elements to delineate the bedrock surface across the footprint of the foundations.

6.2.2 Shallow Foundations (East Pier)

6.2.2.1 Founding Elevation

The east pier could be supported on strip or spread footings constructed on the bedrock surface. The bedrock surface at the east pier in Borehole 15-3 was encountered at Elevation 208.8 m, below 2.2 m of water and a 0.9 m thick silty sand and gravel deposit. The bedrock surface elevation and bedrock quality will likely be variable across the footprint of the pier foundation element.

For the footing founded directly on the bedrock, frost protection is not required.

Dewatering would be required to allow for cleaning of the overburden overlying bedrock and for placement of the concrete for footing construction in-the-dry as discussed in Section 6.2.4.2.

The footing subgrade should be inspected by a Quality Verification Engineer following excavation, in accordance with OPSS 902 *(Excavating and Backfilling - Structures)* to check that the founding elevation is reached and that all unsuitable material, including loose soil materials and fractured rock, have been removed. Due to the potential for the surface of the bedrock to be uneven/sloping, dowelling and/or levelling of the bedrock may be required and should be considered/included in the design.

6.2.2.2 Geotechnical Axial Resistance

For strip/spread footings placed directly on the surface of the properly prepared and inspected bedrock subgrade at the east pier, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 10,000 kPa may be used for design. The geotechnical reaction at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS and as such, ULS conditions will govern for this foundation type.

The Preliminary Design geotechnical axial resistance values provided above will have to be re-evaluated and modified as necessary during Detail Design, based on future additional subsurface investigation at the proposed west abutment.

6.2.3 Driven/Socketed Steel H-Pile or Steel Pipe (Tube) Foundations (West and Middle Piers)

6.2.3.1 Founding Elevations

The west and middle piers may be supported on steel H-piles or steel pipe (tube) piles driven to bedrock at the west pier and into the sand and silt till deposit at the middle pier. The following pile tip elevations may be used for preliminary design:

Foundation Element (Borehole Number)	Approximate Estimated Design Elevation of Pile Tip (m)
West Pier (15-1)	195.7
Middle Pier (15-2)	197.5

The elevation of the underside of the west pier and middle pier pile caps should be below the lowest depth of ice surface and frost penetration. The soil frost penetration depth at this site is 2.6 m below ground surface, as per OPSD 3090.100 (*Foundation Frost Penetration Depths for Northern Ontario)* and we recommend that the underside of the west pier and middle pier pile caps be 2.6 m below the design low water elevation.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the presence of cobbles and boulders within the sand and silt till deposit. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are more likely to “hang up” or deflect away from their vertical or battered orientation during installation, due to their larger end area. The piles should be fitted with driving shoes or flange plates (reinforced tips) in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe) to minimize damage to the pile tip during driving. A heavier pile section, such as HP 310X125 could be used in conjunction with reinforced tips, to reduce the potential of damaging the pile during more difficult driving.

6.2.3.2 Geotechnical Axial Resistance

At the west pier, for HP 310X110 piles driven to bedrock, a factored geotechnical axial resistance at ULS of 2,000 kN may be used for the design. This value represents a structural limitation for the piles rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored geotechnical axial resistances at ULS. Since the bedrock is considered to be an unyielding material, ULS conditions will govern for this foundation type.

At the middle pier, for HP 310X110 piles driven into the sand and silt till deposit, a factored geotechnical axial resistance at ULS of 1,600 kN and a geotechnical reaction at SLS (for 25 mm of settlement) of 1,100 kN may be used for preliminary design. If a greater geotechnical axial resistance is required, consideration should be given to drilling through the boulder deposit and founding the H-Piles on bedrock. However, additional boreholes will be required to confirm the bedrock elevation at the middle pier.

Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm ($^3/_8$ in). Pipe (tube) pile tip reinforcement should be consistent with OPSD 3001.100 (Steel Tube Pile Driving Shoe).

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to allow seating on the bedrock surface, if applicable, and to also avoid overdriving and possibly damaging the piles.

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during Detail Design, and appropriate pile driving notes referenced, in consideration of the additional subsurface investigation at the foundation elements.

6.2.4 Construction Considerations

6.2.4.1 Obstructions

The soils at this site are glacially derived and include cobbles and boulders, which could affect the installation of deep foundations.

6.2.4.2 Cofferdam Construction

Construction of the pile caps for the in-water piers will require some form of cofferdam. Conventional cofferdam construction (i.e., the use of interlocking steel sheet piles driven through the overburden to form a water tight box structure) should be feasible for the west and middle pier but likely impractical at the east pier because of the presence of bedrock at shallow depth and potential for sloping bedrock surface downwards to the west. Consideration could be given to using a prefabricated cofferdam at the east pier (i.e., box), floated and then anchored into place. Due to the potential for sloping/uneven bedrock across the footprint of the cofferdam at the east pier, the need for sealing along the base of the sidewalls of the cofferdam should be anticipated to restrict water from entering the base of the cofferdam. The Contractor should be alerted that excavations for the east pier foundation will be advanced through cohesionless soils, which will likely be unstable below the water level.

6.3 Structural Culvert at STA 10+926

6.3.1 Foundation Options

The proposed 52 m long replacement culvert is to be located on Highway 11 at STA 10+926, about 400 m to the west of the Groundhog River Bridge and about 10 m east of the existing culvert. The creek will be realigned to the new culvert location and the highway will be realigned such that the new centreline will be about 20 m north of the existing centreline. The existing/proposed highway embankment is approximately 8 m high relative to the proposed invert of the culvert outlet (north end). The existing embankment is generally constructed of earth fill with a zone of cobbles and boulders encountered in one of the two boreholes advanced through the roadway. Due to the realignment of the highway, the new culvert will need to be constructed in stages using roadway protection. Based on the proposed culvert dimensions (3660 mm wide by 2740 mm high), a pre-cast concrete box culvert or open footing culvert are feasible at this site.

6.3.1.1 Embankment Stability and Settlement

For the subsurface conditions encountered in the boreholes and the proposed embankment height up to about 8 m, a granular fill embankments at this site will be stable at side slopes inclined at 2 Horizontal to 1 Vertical (2H:1V), or flatter, provided existing fill and organic soils are removed from below the proposed embankment footprint. Rock fill embankments inclined at 1.25H:1V will also be stable at this site.

The culvert will be constructed within the proposed highway realignment. Along the proposed culvert alignment the native soils will experience varying amounts of additional loading/unloading, as the southern portion of the founding soils have already experienced loading from the existing embankment. Due to the presence of compact to very dense till foundation soils, the total settlement along the north half of the culvert (under the new embankment loading/widening) is anticipated to be less than 25 mm and will occur after completion of embankment construction. For the south section of the culvert, within the footprint of the existing embankment, the settlement is anticipated to be negligible. The resulting differential settlement is up to about 25 mm and will occur after embankment construction.

6.3.1.2 Design Recommendations for Concrete Culverts

6.3.1.2.1 Box Culvert

It is not necessary to found a box culvert at the standard depth for frost protection purposes, as a box structure is tolerant of small magnitudes of movement related to freeze-thaw cycles, should these occur.

The factored geotechnical axial resistance at Ultimate Limit States (ULS) and the geotechnical reaction at Serviceability Limit States (SLS) for 25 mm of settlement for a 3 m to 4 m wide box culvert founded on a granular bedding placed on a properly prepared subgrade comprised of the native till may be taken as 350 kPa and 250 kPa, respectively.

The geotechnical resistance/reaction provided above are based on loading applied perpendicular to the base of the culvert; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 and Section C6.7.4 of the Canadian Highway Bridge Design Code (CHBDC 2006) and its Commentary.

It is recommended that the structural engineer exercise caution when utilizing the values of the geotechnical resistance at SLS provided above in the design of the box culvert and that consideration be given to the sequence and staging of construction, particularly of the proposed new embankment construction, as the settlement under the culvert resulting from the embankment loading (not culvert loading) will govern.

6.3.1.2.2 Open Footing Culvert

Strip footings for an open footing culvert should be founded at a minimum depth of 2.6 m below the lowest surrounding grade to provide adequate protection against frost penetration, as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario). In addition, the footings should extend below any existing fill and organic deposits, where present.

The factored geotechnical axial resistance at ULS and geotechnical reaction at SLS for 25 mm of settlement for an assumed 1 m wide strip footing placed on the properly prepared native till subgrade may be taken as 210 kPa and 150 kPa, respectively.

The factored geotechnical axial resistance at ULS and geotechnical reaction at SLS are dependent on the foundation size, configuration and applied loads; the geotechnical axial resistance/reaction should, therefore, be reviewed if the culvert footing width is different than given above.

The geotechnical resistance/reaction provided above are based on loading applied perpendicular to the base of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 and Section 6.7.4 of the CHBDC and its Commentary.

It is recommended that the structural engineer exercise caution when utilizing the values of the geotechnical resistance at SLS provided above in the design of the open footing culvert and that consideration be given to the sequence and staging of construction, particularly if the proposed new embankment construction, as the settlement under the culvert resulting from the embankment loading (not culvert loading) will govern.

6.3.1.3 Resistance to Lateral Loads / Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of the concrete box culvert and the new granular fill/bedding, or the cast-in-place open footings on the bedrock or native subsoils, placed following sub-excavation of unsuitable materials should be calculated in accordance with Section 6.7.5 of the CHBDC. The following summarizes the coefficient of friction for the interface materials for a precast and cast–in-place culvert.

Interface Materials	Coefficient of Friction
Precast Concrete on Compacted Granular ‘B’ Type II material	tan δ = 0.45
Cast-In-Place Concrete on Compacted Granular ‘B’ Type II or Native Till	tan δ = 0.58

6.3.2 Lateral Earth Pressures

The lateral earth pressures acting on the walls of the culvert and culvert and wing walls will depend on the type and method of placement of backfill materials, the nature of the soils/embankment fill behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structures, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the culvert walls and wing walls. It should be noted that these design recommendations and parameters are for level backfill and ground surface behind the walls, and 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 Granular 'A' or Granular 'B' Type II should be used as backfill behind the culvert walls, and on top of the culvert for a thickness of not less than 300 mm. Backfill should be placed in a maximum of 200 mm loose lift thickness and nominally compacted. 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).
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary* to the *CHBDC*). For unrestrained walls, fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H: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*). The pressures are based on the proposed embankment replacement backfill material and the following parameters (unfactored) may be used:

Fill Type	**Soil Unit Weight**	**Coefficients of Static Lateral Earth Pressure**	
		At-Rest, K_o	**Active, K_a**
Granular 'A'	22 kN/m^3	0.43	0.27
Granular 'B' Type II	21 kN/m^3	0.43	0.27

If the wall structure allows for lateral yielding, active earth pressures may be used in the foundation design. If the wall structure does not allow for lateral yielding, at-rest earth pressures should be assumed for culvert design. The movement to allow active pressures to develop within the backfill, and thereby assume a restrained structure, may be taken as per Table C6.6 of the *Commentary to the CHBDC*.

6.3.3 Construction Considerations

6.3.3.1 Temporary Roadway Protection

The temporary excavation for the new culvert will extend through the existing earth fill with zones of cobbles and boulders and into native soils which are comprised of very loose to compact sandy peat, loose sand and gravel and compact to very dense gravelly silty sand to sandy silt till. All excavations must be carried out in accordance with Ontario Regulation 213, Ontario Occupational Health and Safety Act for Construction Projects (as amended). The existing fill and non-organic native soils are considered to be Type 3 soil above the groundwater table and Type 4 soil below the groundwater level. Temporary open-cut excavations in Type 3 soils should remain stable if side slopes are formed no steeper than 1.5H:1V. In Type 4 soils, the side slopes should be formed no steeper than 3H:1V.

Temporary protection support systems may be required along the highway to facilitate construction staging and maintain traffic during culvert replacement work. The temporary support systems could consist of either driven sheet-piling extending to a suitable depth, or soldier piles and lagging where H-piles are driven to a suitable depth and horizontal lagging is installed as the excavation proceeds. Support to the system could be in the form of struts and walers and rakers or anchors. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). Temporary excavation support systems should be designed to Performance Level 2 for any excavation adjacent to the existing roadway.

The installation of the sheet-piles for culvert construction and/or temporary shoring may be impeded by the presence of boulders (greater than 300 mm in size) within the lower portion of existing embankment fill material. It may be necessary to excavate and replace the existing fill material in the areas of sheet-pile installation in a series of limited length and narrow trenches. In general, the narrowest suitable excavator bucket should be used. The replacement fill could consist of excavated fill material or imported granular material provided that 100 per cent of the material passes the 75 mm sieve size. Sieving, sorting or picking of large particles from the excavated spoil pile may be required if the excavated material is re-used. Alternatively, imported Granular ‘A’ or Granular ‘B’ Type I or II may be used as backfill for the excavated trench. Excavation and replacement should be carried out on the same day to avoid leaving any trench open overnight.

As an alternative to excavation/trenching of the cobble/boulder layer, pre-drilling for the soldier (or tube) pile and lagging system could be completed in advance to allow for pile installation. Between the piles, the cobble and boulder fill may have to be line-drilled to break up the cobbles and boulders into smaller pieces to facilitate lagging installation and to minimize loosening of the embankment cobble and boulder matrix.

The support systems may be designed based on the following soil parameters:

	COEFFICIENT OF EARTH PRESSURE			**INTERNAL ANGLE OF FRICTION**	**UNIT WEIGHT**	**UNDRAINED SHEAR STRENGTH**
SOIL TYPE	**Active, K_a**	**At Rest, K_o**	**Passive, K_p**	**(ϕ, degrees)**	**(γ, kN/m^3)**	**(S_u, kPa)**
Existing Sand to Sand and Gravel Fill	0.33	0.50	3.0	30	20	-
Existing Clayey Silt Fill	0.36	0.53	2.8	28	18	-
Existing Cobbles and Boulders and Sand and Gravel Fill	0.27	0.43	3.7	35	19	-
Sandy Silt to Silt and Sand Till	0.31	0.47	3.2	32	20	-

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly. Further, hydrostatic pressures must be added to the earth pressure where groundwater in not fully covered to below the excavation level.

6.3.3.2 Excavation and Replacement Below Culvert

Prior to placement of any bedding material, engineered fill or concrete, all organics (including peat, topsoil, organic clay or mixed organic materials) and any softened/loosened or disturbed soils, should be sub-excavated from below the plan limits of the proposed works to the founding level.

The culvert subgrade should be inspected by a Quality Verification Engineer following sub-excavation to ensure that all organics and other unsuitable materials have been removed as noted above, in accordance with OPSS 422 (Precast Reinforced Concrete Box Culverts) for a pre-cast box culvert and OPSS 902 (Excavating and Backfilling Structures) for an open footing culvert. Following inspection, the sub-excavated area should be backfilled with granular material meeting the requirements of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II that is placed and compacted in accordance with OPSS.PROV 501 (Compacting). The use of Granular ‘B’ Type II fill is recommended in peat sub-excavation areas with wet ground conditions or below water and placement should be in accordance with OPSS.PROV 209 (Embankments over Swamps).

6.3.3.3 Culvert Bedding and Backfill

6.3.3.3.1 Box Culvert

The bedding and levelling pad requirements for a pre-cast box culvert should be accordance with OPSS 422 (Precast Reinforced Concrete Box Culverts). Given the potential for surface water flow and some groundwater seepage through the native soils during excavation to the invert and bedding level, it is recommended that a minimum 300 mm thick layer of OPSS.PROV 1010 (Aggregates) Granular ‘B’ Type II material be used for bedding purposes. As the native soil below the bedding is generally fine grained, it is recommended that a non-woven geotextile be placed between the native soil and the bottom of the bedding. The geotextile should meet the specifications for OPSS 1860 (Geotextiles) Class II, and have a fabric opening size (FOS) not greater than 212 µm. The bedding should be placed in maximum 300 mm thick loose lifts and compacted to at least 98 per cent of the Standard Proctor Maximum Dry Density (SPMDD) of the materials as specified in OPSS.PROV 501 (Compacting). In addition, a 75 mm thick uncompacted levelling pad consisting of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or fine concrete aggregate meeting the grading requirements specified in OPSS.PROV 1002 (Aggregates – Concrete) should be provided with geometry similar to that provided on OPSD 803.010 (Backfill and Cover for Concrete Culverts) for culvert construction in dry conditions.

Although the box culvert may be of a slightly greater span than 3 m, a frost taper should be constructed with geometry similar to that provided on OPSD 803.010 (Backfill and Cover for Concrete Culverts).

6.3.3.3.2 Open Footing Culvert

The excavation and backfilling requirements for the open footing culvert replacement should be in accordance with OPSS 902 (Excavating and Backfilling – Structures) and also in similar configuration to that shown on OPSD 803.010 as noted in Section 6.4.3.1.

Should pre-cast strip footings be selected for the open footing culvert replacement option, a bedding layer and levelling pad will be required above the native soil. The bedding layer and levelling pad for the pre-cast open footings should follow the recommendations as discussed above in Section 6.3.3.3.1 for the box culvert replacement option.

A frost taper should be constructed with geometry similar to that provided on OPSD 803.010 (Backfill and Cover for Concrete Culverts), even though the culvert span width may be slightly greater than 3 m.

6.3.3.3.3 Backfill

Backfill behind the culvert walls should consist of granular fill meeting the specifications for OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type I or II. The backfill should be placed in maximum 200 mm thick loose lifts and be compacted to at least 98 per cent of the SPMDD of the materials in accordance with OPSS.PROV 501 (Compacting). The fill should also be placed concurrently on both sides of the culvert, ensuring that the backfill depth on one side does not exceed the other side by more than 400 mm.

Backfill placement for reconstruction of existing roadway embankments over the culvert should be carried out as per OPSD 208.010 (Benching of Earth Slopes) to integrate the existing embankment fill and new fill along the cut faces.

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.

6.3.3.4 Subgrade Protection

The native till soils at this site will be susceptible to disturbance from construction traffic and/or ponded water. To limit the effect of this disturbance, and as an alternative to the 300 mm compacted bedding layer, a concrete working slab should be placed on the subgrade if the concrete footings, or the box culvert, is not placed within four hours after preparation, inspection, and approval of the foundation subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28 day compressive strength of 20 MPa.

6.3.3.5 Erosion Protection

Provision should be made for scour and erosion protection at the culvert location. In order to prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or concrete cut-off wall should be provided at the upstream end of the culvert. If a clay seal is adopted, the clay material should meet the requirements of OPSS.PROV 1205 (Clay Seal), and the seal should be a minimum thickness of 1 m, if constructed of natural clay or soil bentonite mix. The clay seal should extend from a depth of 1 m below the scour level to a minimum vertical height equivalent to the high water level. The seal should also extend a minimum horizontal distance of 2 m on either side of the culvert inlet opening. Alternatively, a 0.6 m thick clay blanket may be constructed, extending upstream three times the culvert height and along the adjacent slopes to a height of two times the culvert height or the high water level, whichever is greater.

The requirements for and design of erosion protection measures for the inlet and outlet of the culvert should be assessed by the hydraulics design engineer. As a minimum, rip rap treatment for the outlet of the culvert should be consistent with the standard presented in OPSD 810.010 (Rip Rap Treatment). Erosion protection for the inlet of the culvert should also follow the standard presented in OPSD 810.010 (Rip Rap Treatment) similar to the outlet but with the rip rap placed up to the toe of slope level, in combination with the cut off measures noted above. Similarly, rip rap should be provided over the full extent of the clay blanket, including the creek side slopes and fill slope over the culvert if a clay seal is adopted.

6.3.4 Control of Groundwater and Surface Water

Excavation along the culvert alignment will be required to remove embankment fills, organic and overburden soils prior to placement of backfill, bedding material, engineered fill and the actual culvert structure. Groundwater flow into the excavation extending below the adjacent ground surface can be expected due to the depth of the excavations and the presence of relatively permeable fill. Therefore, control of groundwater will be necessary to allow for construction to be carried out in dry conditions, where required. Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade.

Depending on the creek flow, local surface water flow conditions and groundwater level at the time of construction, water flow could be passed through the area by means of a temporary culvert or by pumping from behind temporary cofferdams or by constructing a creek channel diversion.

For both the box and open footing culvert options, excavations will extend below the creek water level and likely below the groundwater level and will therefore require temporary shoring with dewatering to allow for construction/placement of the footings and/or placement of engineered fill in dry conditions. Temporary shoring and dewatering could be in the form of a sheet-pile cut-off wall or cofferdam advanced to an appropriate depth to control groundwater inflow from the creek. As discussed in Section 6.3.3.3, engineered fill or organic sub-excavation replacement backfill can be placed sub-aqueously, however, dewatering may still be required for footing/box culvert placement as the culvert invert is at or below the creek water level.

Dewatering of all excavations should be carried out in accordance with OPSS 517 (Dewatering).

At this preliminary stage, an accurate prediction of the groundwater pumping volumes cannot be made, as the flow rate would be dependent on construction methods adopted by the contractor. However, it is considered that groundwater pumping volumes could exceed 50,000 L/day but likely less than 400,000 L/day during initial drawdown stages and/or during periods of heavy precipitation. Under recently introduced changes to the Environmental Protection Act by the Ontario Ministry of the Environment and Climate Change (MOECC), water taking for construction site dewatering for volumes greater than 50,000 L/day but less than 400,000 L/day qualify for the Environmental Activity Section Registry (EASR). Under the EASR, a Permit to Take Water is not required for water taking for construction site dewatering for volumes less than 400,000 L/day.

6.3.5 Obstructions

The contractor should be alerted to the presence of cobble and boulder site material within the embankment fill material as encountered in Borehole GHR-8.

6.4 Recommendations for Further Work during Detail Design

Additional boreholes will be required within each of the bridge foundation elements and within the approach embankment areas for detail design investigation at the bridge, as applicable, to further assess and/or confirm the subsurface conditions and the Preliminary Design recommendations provided herein, as follows:

- Further assessment of the elevation of the bedrock surface across each of the foundation elements, including at the abutments for the assessment of socketing the piles if the bedrock surface is sloped, and including the east pier for shallow foundations on bedrock;
- Confirmation of the tip elevation for driven steel H-piles including assessment of “refusal” condition for end bearing piles; and
- Observation of the presence of cobbles and/or boulders within the native non-cohesive deposits to assess the need to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of deep foundations.

7.0 CLOSURE

This report was prepared by Mr. Matt Thibeault, P.Eng. and the technical aspects were reviewed by Mr. André Bom, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., Golder’s Designated MTO Contact for this project and a Senior Consultant with Golder, conducted an independent quality control review of the report.

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REFERENCES

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6-06, 2006. CSA Special Publication, S6.1-06. Canadian Standard Association.

Golder Associates Ltd., Preliminary Foundation Investigation and Design Report, Replacement of Groundhog River Bridge, Site 39W-093, Highway 11 at Fauquier, Cochrane District, Ontario, GWP 5049-07-00, May 2013. GEOCRES NO. 42G-40

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Ontario Provincial Standard Drawings

OPSD 208.010	Benching of Earth Slopes
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0 m
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Pile Driving Shoe
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario

Ontario Provincial Standard Specifications

OPSS.PROV 209	Construction Specification for Embankments Over Swamps and Compressible Soils
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1002	Material Specification for Aggregates – Concrete
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS.PROV 1205	Material Specification for Clay Seal
OPSS 1860	Material Specification for Geotextiles

Ontario Water Resources Act

Ontario Regulation 903/90	Wells; O.Reg 468/10 Amendment to Ontario Regulation 903

LIST OF SYMBOLS

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 = $(w_l - w_p)$
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 = $\tan\delta$
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

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. 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.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; 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, 60º 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

Piezo-Cone Penetration Test (CPT)

A 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 (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index Relative Density	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency

	c_u, s_u	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
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, G_s)
DS	direct shear test
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	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

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, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

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 90^{o} 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

JN	Joint
FLT	Fault
SH	Shear
VN	Vein
FR	Fracture
SY	Stylolite
BD	Bedding
FO	Foliation
CO	Contact
AXJ	Axial Joint
KV	Karstic Void
MB	Mechanical Break
PL	Planar
CU	Curved
UN	Undulating
IR	Irregular
K	Slickensided
PO	Polished
SM	Smooth
SR	Slightly Rough
RO	Rough
VR	Very Rough

Table 1: Comparison of Foundation Alternatives – West and Middle Piers

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles Driven to bedrock at the west pier, driven into the till at the middle pier and socketted into bedrock at the east pier.	1	■ Straightforward construction. ■ Consistent foundations with the abutments.	■ Socketing of piles into strong bedrock will be required at the east pier to achieve the minimum pile length for structural design and to provide for lateral stability/fixity for the tip. ■ Potential for “hanging up” on cobbles and boulders within non-cohesive deposits. ■ Will require cofferdams in river for excavation of overburden and construction of pile cape.	■ Relative costs lower than caisson. ■ Additional foundations cost for socketing piles into strong bedrock at east pier.	■ Need to achieve minimum required pile length by socketing into strong bedrock at east pier.
Driven Steel Tube Piles	2	■ Straightforward construction.	■ Depending on the elevation of the underside of pile cap at the west abutment, socketing of piles into strong bedrock will be required to achieve minimum pile length. ■ Higher potential for deflecting off alignment due to the presence of cobbles and boulders deposit or within non-cohesive deposits. ■ Will require cofferdams in river for excavation of overburden and construction of pile cape.	■ Relative costs lower than caissons. ■ Additional cost for socketing of piles into strong bedrock at east pier.	■ Need to achieve minimum required pile length by socketing into strong bedrock.

Table 1: Comparison of Foundation Alternatives – West and Middle Piers

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons	3	■ Higher axial resistance compared to steel H-piles or tube piles. ■ Possible elimination of pile cap and associated excavation within cofferdams in rivers.	■ Requires rock drilling/large socket for seating caissons into bedrock. ■ Potential for difficulty associated with seating a larger diameter caisson into strong bedrock. ■ Require temporary or permanent liners to advance caissons at east abutment. ■ Different foundation types of abutments compared to piers, requiring different construction equipment and possibly different contractors.	■ Relative costs much higher than for steel H-piles, although fewer foundation units are required.	■ Likely able to reach the required termination depth into bedrock. ■ Potential for construction problems associated with river water and/or groundwater inflow into caisson during installation will likely have to use tremie concrete construction methods. ■ May also need liner to advance caissons.

Prepared by: AB
Reviewed by: JMAC

APPENDIX A

Groundhog River Bridge – Current Investigation

MINISTRY OF TRANSPORTATION, ONTARIO

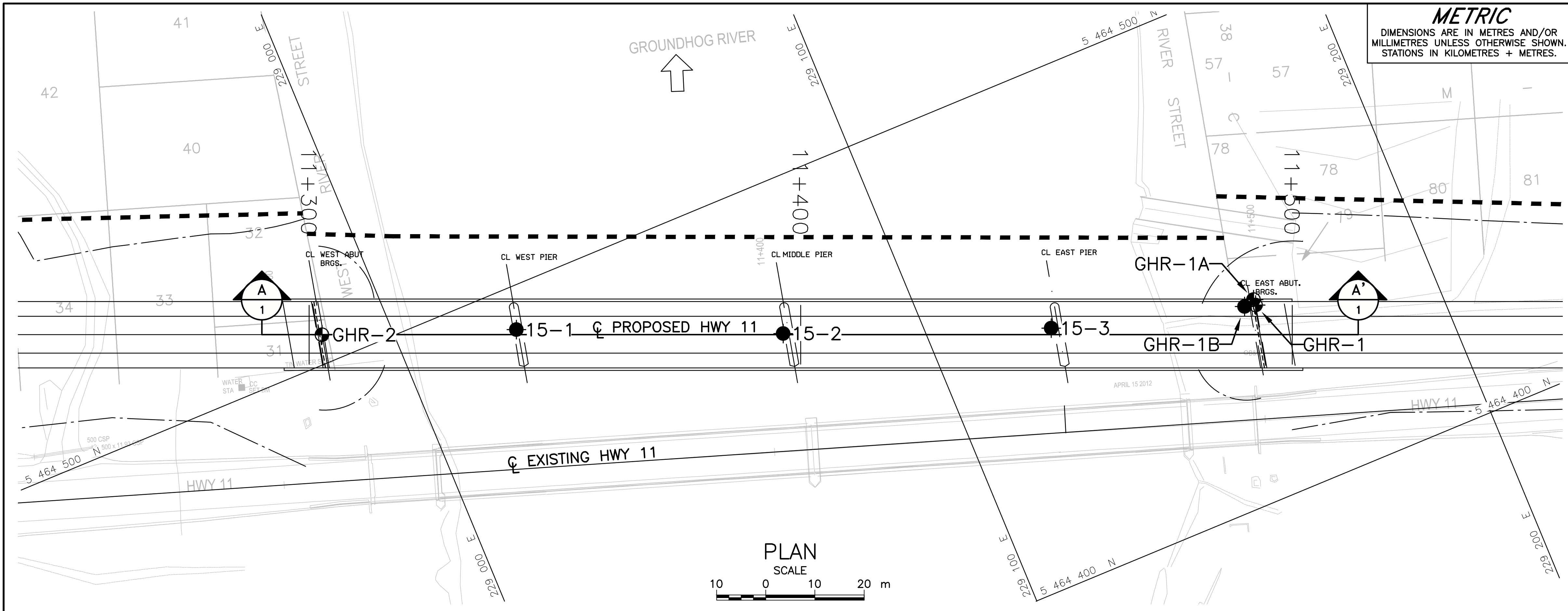
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.5049-07-00

HIGHWAY 11
GROUNDHOG RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

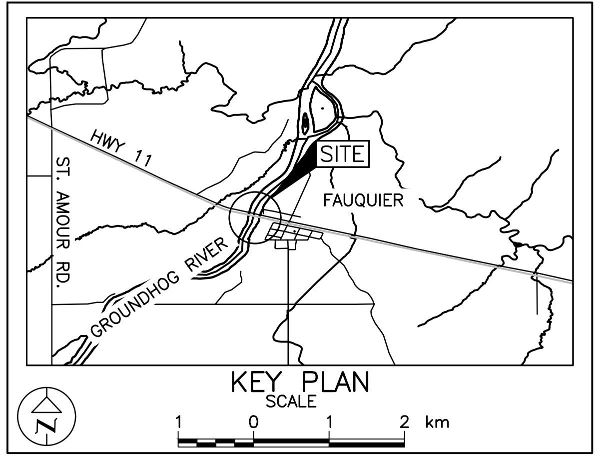
SHEET
.

Golder Associates



PLAN

SCALE
10 0 10 20 m



KEY PLAN

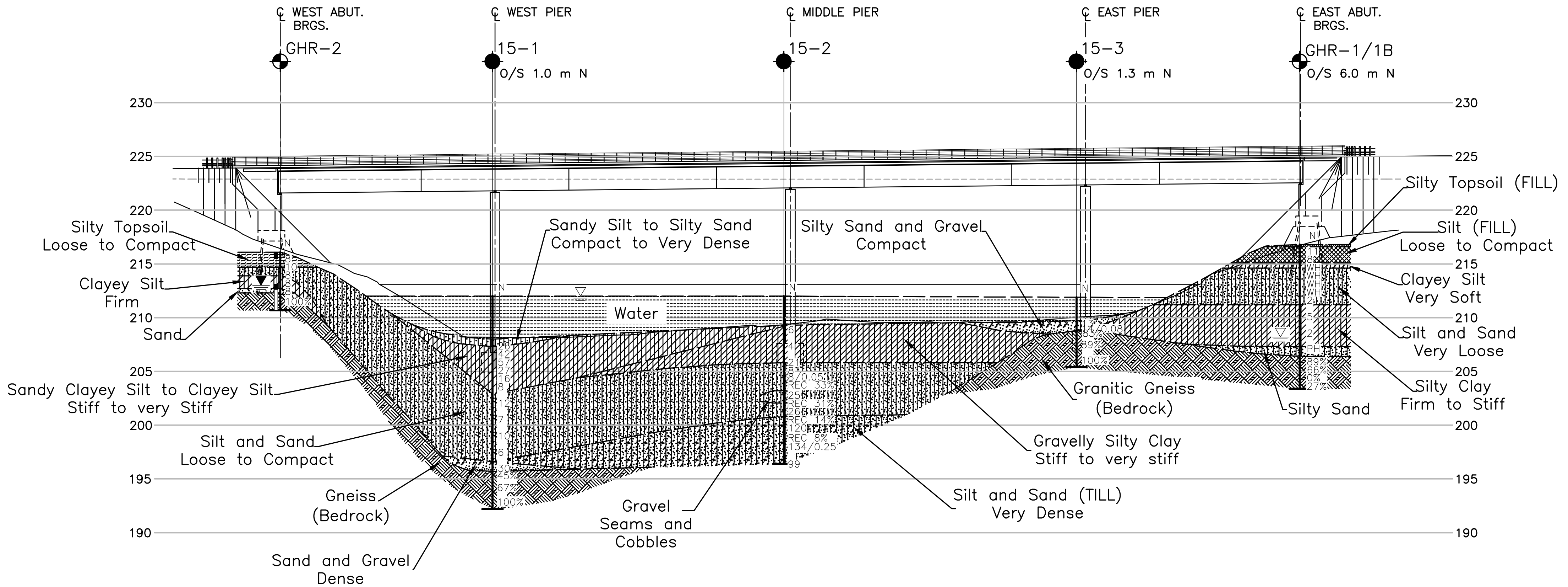
SCALE
1 0 1 2 km

LEGEND

●	Borehole – Current Investigation
⊕	Borehole – 2012
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
REC %	Recovery
▼	WL in piezometer, measured on AUG 1, 2012
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
15-1	212.0	5464491.5	229023.4
15-2	212.0	5464470.0	229073.2
15-3	211.9	5464450.1	229124.0
GHR-1	216.8	5464438.5	229164.2
GHR-1A	216.8	5464439.7	229164.2
GHR-1B	216.9	5464439.1	229162.0
GHR-2	216.1	5464505.7	228986.5



A-A 1 CENTRELINE PROFILE

HORIZONTAL SCALE
10 0 10 20 m
5 0 5 10 m
VERTICAL SCALE

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. GA_Groundhog River_3B.dwg, received APR 12, 2013.

A.J.K. BOM
100075715
JUL 19, 2016

J.M.A.COSTA
JUL 19, 2016

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 42G-62

HWY. 11		PROJECT NO. 11-1191-0025	DIST. .
SUBM'D. MT	CHKD. .	DATE: 7/19/2016	SITE: 39W-093
DRAWN: JJL	CHKD. AB	APPD. JMAC	DWG. A1

PLOT DATE: July 20, 2016
FILENAME: H:\Projects\2011\11-1191-0025 hwy 11 onr and ghr\2-GHR\1111910025AA004.dwg

PROJECT 11-1191-0025	**RECORD OF BOREHOLE No GHR-1B**	1 OF 2	**METRIC**
G.W.P. 5049-07-00	LOCATION N 5464439.1; E 229162.0		ORIGINATED BY TM
DIST ___ HWY 11	BOREHOLE TYPE HW Casing, NQ Coring		COMPILED BY MT
DATUM GEODETIC	DATE November 12, 2014		CHECKED BY AB

SOIL PROFILE			SAMPLES								
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/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.9	GROUND SURFACE										
0.0	For stratigraphy details refer to Record of Borehole No GHR-1										
							216				
						∇					
							215				
							214				
							213				
							212				
							211				
							210				
							209				
							208				
							207				
206.7 10.2	GRANITIC GNEISS (BEDROCK) Bedrock cored from 10.2 m depth to 13.4 m depth. For coring details see Record of Drillhole GHR-1B.	▨	1	RC	REC 100%						RQD = 89%
		▨					206				
		▨	2	RC	REC 100%						RQD = 66%
		▨					205				
		▨	3	RC	REC 100%						RQD = 0%
		▨	4	RC	REC 94%		204				RQD = 27%
203.5 13.4	END OF BOREHOLE Note: 1. Water level at a depth of 1.3 m below ground surface (Elev. 215.6 m) upon completion of drilling.										

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

PROJECT: 11-1191-0025

RECORD OF DRILLHOLE: GHR-1B

SHEET 2 OF 2

LOCATION: N 5464439.1 ;E 229162.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 12, 2014

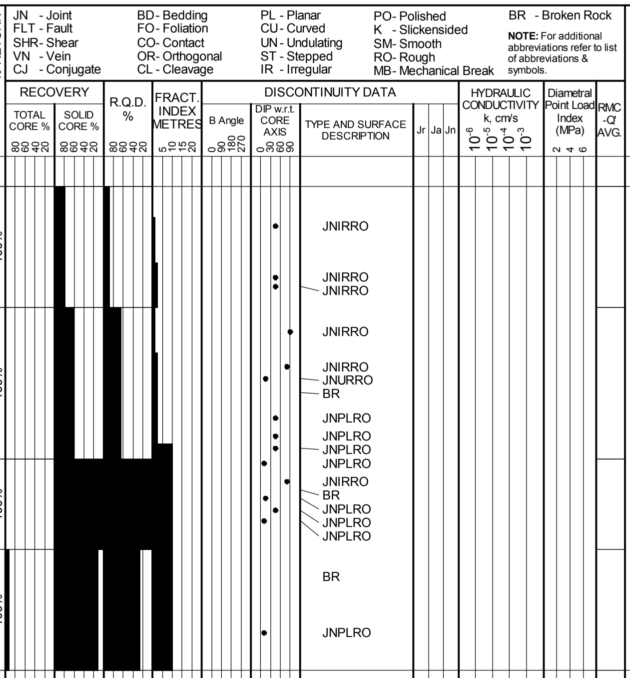
DRILL RIG: D25

DRILLING CONTRACTOR: Walker 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.	FLUSH COLOUR % RETURN	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		TOP OF BEDROCK						
	HW	GRANITIC GNEISS Medium to coarse grained Slightly weathered Strong Pink Broken core encountered from 11.6 m to 11.7 m, 12.1 m to 12.2 m and 12.4 m to 13.1 m depth.		206.7 10.2	1	GREY 100%	JNIRRO JNIRRO JNIRRO	
11	November 12, 2014 NQ Coring				2	GREY 100%	JNIRRO JNIRRO JNURRO BR JNPLRO JNPLRO JNPLRO JNPLRO	
12					3	GREY 100%	JNIRRO BR JNPLRO JNPLRO JNPLRO	
13				203.5	4	GREY 100%	BR JNPLRO	
		END OF DRILLHOLE		13.4				
14								
15								
16								
17								
18								
19								
20								



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: TM

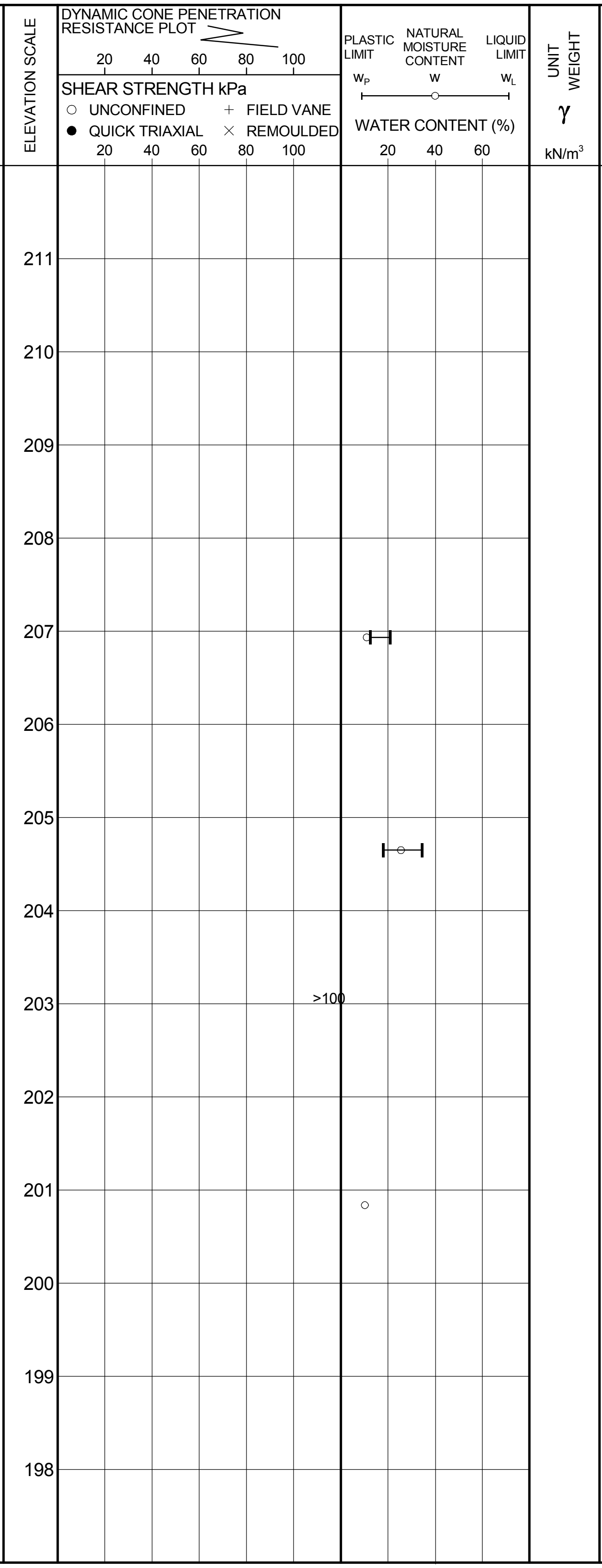
CHECKED: AB

RECORD OF BOREHOLE No 15-1

1 OF 3 **METRIC**

PROJECT 11-1191-0025
G.W.P. 5049-07-00
LOCATION N 5464491.5; E 229023.4
ORIGINATED BY MT
DIST
HWY 11
BOREHOLE TYPE HW Casing, NW Casing, NQ Coring
COMPILED BY MT
DATUM GEODETIC
DATE August 30 and 31, 2015
CHECKED BY AB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
212.0	WATER SURFACE					▽				
0.0	WATER									
208.0 4.0	Sandy SILT to Silty SAND, some clay Compact Grey Wet		1	SS	26					
207.3 4.7	Sandy CLAYEY SILT to CLAYEY SILT Stiff to very stiff Grey Wet		2	SS	42		0	23	53	24
			3	SS	57					
			4	SS	27					
			5	SS	16					
			6	SS	8					
202.9 9.1	SILT and SAND, some gravel, trace to some clay Loose to compact Grey Wet		7	SS	12					
			8	SS	7		14	47	33	6
			9	SS	10					
			10	SS	6					
	0.2 m cobble encoutered at approximately 15.1 m depth.		-	RC	REC 50%					



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-1

2 OF 3 **METRIC**

PROJECT 11-1191-0025
G.W.P. 5049-07-00
LOCATION N 5464491.5; E 229023.4
ORIGINATED BY MT
DIST
HWY 11
BOREHOLE TYPE HW Casing, NW Casing, NQ Coring
COMPILED BY MT
DATUM GEODETIC
DATE August 30 and 31, 2015
CHECKED BY AB

SOIL PROFILE			SAMPLES					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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	--- CONTINUED FROM PREVIOUS PAGE ---										
196.6 15.4	SAND and GRAVEL, trace to some silt Dense Grey Wet		11	SS	30		196				
195.7 16.3	GNEISS (BEDROCK) Bedrock cored from 16.3 m depth to 19.8 m depth. For coring detetic see Record of Drillhole 15-1.		1	RC	REC 100%		195				RQD = 45%
			2	RC	REC 100%		194				RQD = 67%
			3	RC	REC 100%		193				RQD = 100%
192.2 19.8	END OF BOREHOLE										

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

SUD-MTO 001 11-1191-0025.GPJ GAL-MISS.GDT 12/11/15 DATA INPUT:

PROJECT: 11-1191-0025

RECORD OF DRILLHOLE: 15-1

SHEET 3 OF 3

LOCATION: N 5464491.5 ;E 229023.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 30 and 31, 2015

DRILL RIG: CME-55

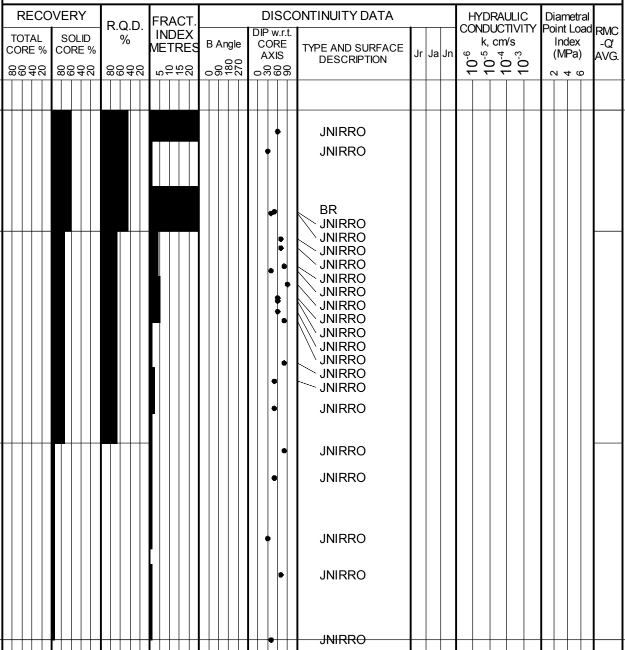
DRILLING CONTRACTOR: Downing Drilling

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	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		TOP OF BEDROCK	195.7				
17	NW August 31, 2015 NQ Coring	GNEISS Coarse grained Strong Grey Fresh Broken core encountred between 16.8 m to 16.9 m depth.	16.3	1	GREY 100%	JNIRRO JNIRRO BR JNIRRO	
18				2	GREY 100%	JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO	UCS = 88 MPa
19			192.2 19.8	3	GREY 100%	JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO	
20		END OF DRILLHOLE					
21							
22							
23							
24							
25							
26							



DEPTH SCALE
1 : 50

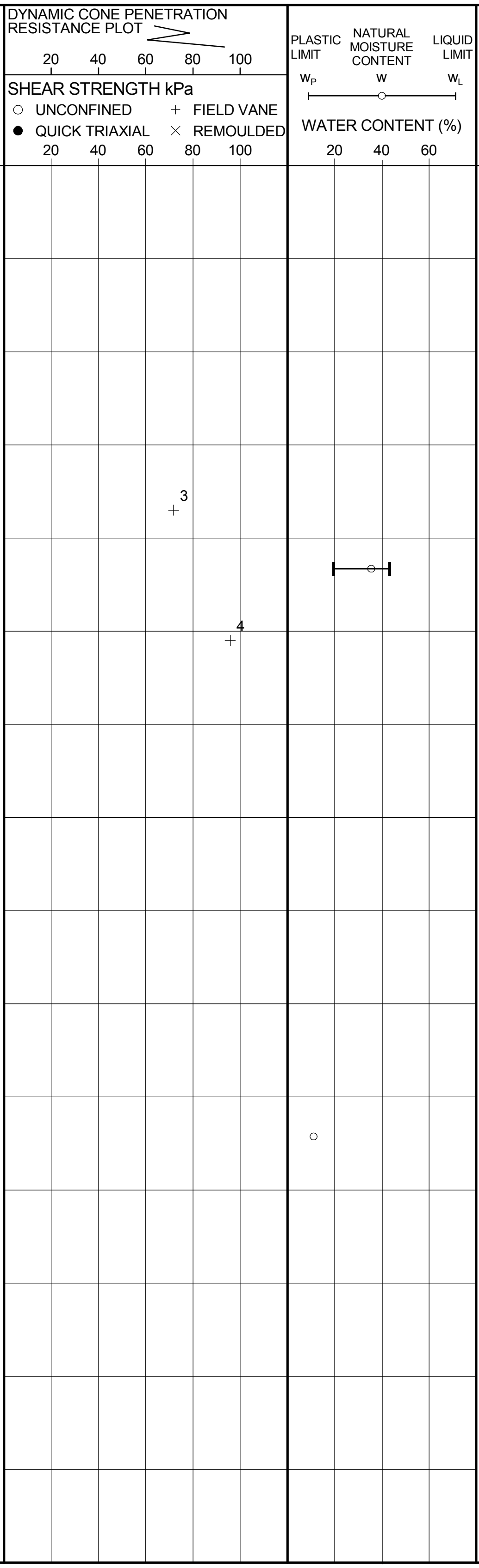
LOGGED: MT
CHECKED: AB

RECORD OF BOREHOLE No 15-2

1 OF 2 **METRIC**

PROJECT 11-1191-0025
G.W.P. 5049-07-00
LOCATION N 5464470.0; E 229073.2
ORIGINATED BY MT
DIST HWY 11
BOREHOLE TYPE HW Casing, NW Casing, NQ Coring
COMPILED BY MT
DATUM GEODETIC
DATE August 29 and 30, 2015
CHECKED BY AB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
212.0	WATER SURFACE							
0.0	WATER							
209.5 2.7	SAND Loose Brown Wet	1	SS	6				
	SILTY CLAY to Gravelly SILTY CLAY Stiff to very stiff Grey Wet	2	SS	4				
		3	SS	21				
205.8 6.2	SILT and SAND, some gravel, trace clay Loose to compact Grey Wet Gravel seams noted between 7.1 m and 11.2 m depth.	4	SS	8				
		5	SS	8/0.05				
	0.2 m cobble encoutered at approximately 8.1 m depth.	-	RC	REC 33%				
		6	SS	25				
	0.1 m and 0.2 m cobbles encountered at approximately 9.7 m depth.	-	RC	REC 31%				
		7	SS	26	13	52	31	4
	0.1 m cobble encoutered at approximately 11.0 m depth.	-	RC	REC 14%				
200.8 11.2	SILT and SAND, some gravel (TILL) Very dense Grey Wet	8	SS	120				
		-	RC	REC 8%				
		9	SS	134/0.25				
	Silt seams/layers below 14.3 m depth.							



UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No 15-2

2 OF 2 **METRIC**

PROJECT 11-1191-0025

G.W.P. 5049-07-00

DIST

HWY 11

DATUM GEODETIC

LOCATION N 5464470.0; E 229073.2

BOREHOLE TYPE HW Casing, NW Casing, NQ Coring

DATE August 29 and 30, 2015

ORIGINATED BY MT

COMPILED BY MT

CHECKED BY AB

SOIL PROFILE			SAMPLES					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/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
			10	SS	99						
196.4 15.6	END OF BOREHOLE										

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

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

RECORD OF BOREHOLE No 15-3

1 OF 2 **METRIC**

PROJECT 11-1191-0025
G.W.P. 5049-07-00
DIST ____ HWY 11
DATUM GEODETIC

LOCATION N 5464450.1; E 229124.0
BOREHOLE TYPE HW Casing, NW Casing, NQ Coring
DATE August 29, 2015

ORIGINATED BY MT
COMPILED BY MT
CHECKED BY AB

SOIL PROFILE			SAMPLES								
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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
211.9	WATER SURFACE					▽					
0.0	WATER						211 210				
209.7 2.2	Silty SAND and GRAVEL Compact Grey Wet		1	SS	13						
			2	SS	14/0.08		209				
208.8 3.1	GRANITIC GNEISS (BEDROCK) Bedrock cored from 3.1 m depth to 6.5 m depth. For coring details see Record of Drillhole 15-3.		1	RC	REC 100%						RQD = 83%
			2	RC	REC 100%		208 207				RQD = 89%
			3	RC	REC 100%		206				RQD = 100%
205.4 6.5	END OF BOREHOLE										

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

PROJECT: 11-1191-0025

RECORD OF DRILLHOLE: 15-3

SHEET 2 OF 2

LOCATION: N 5464450.1 ;E 229124.0

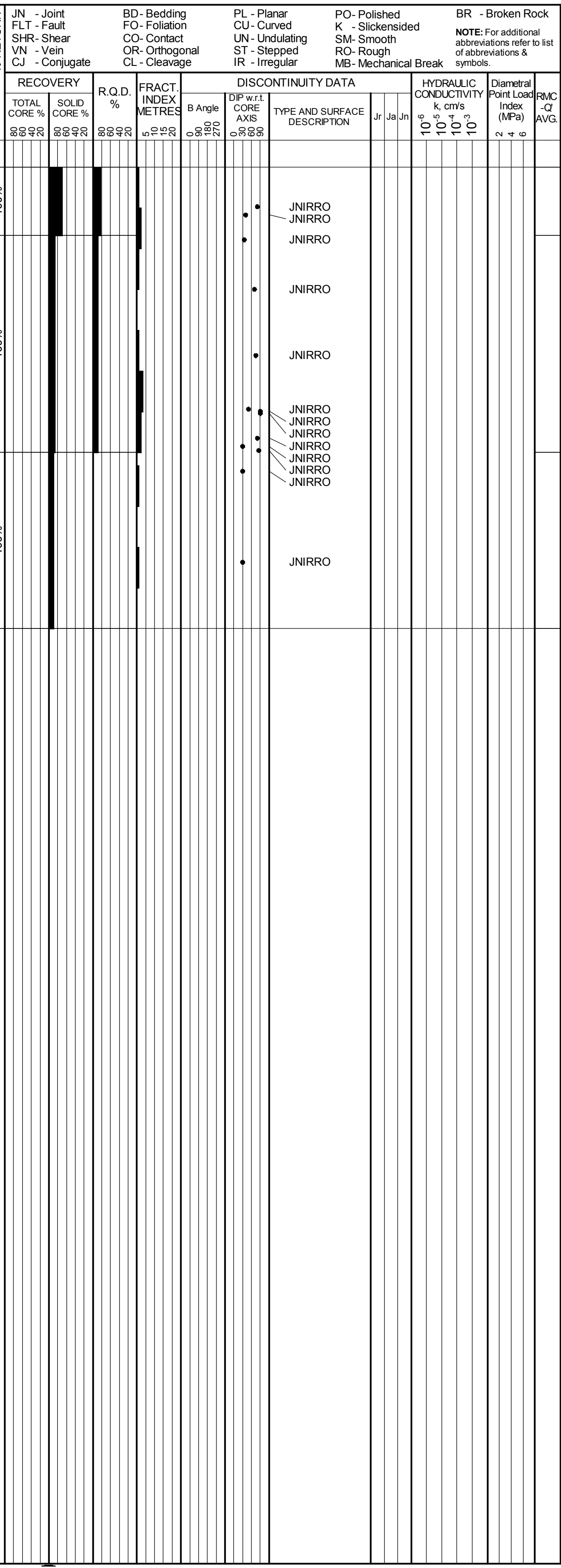
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 29, 2015
DRILL RIG: CME-55
DRILLING CONTRACTOR: Downing 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	

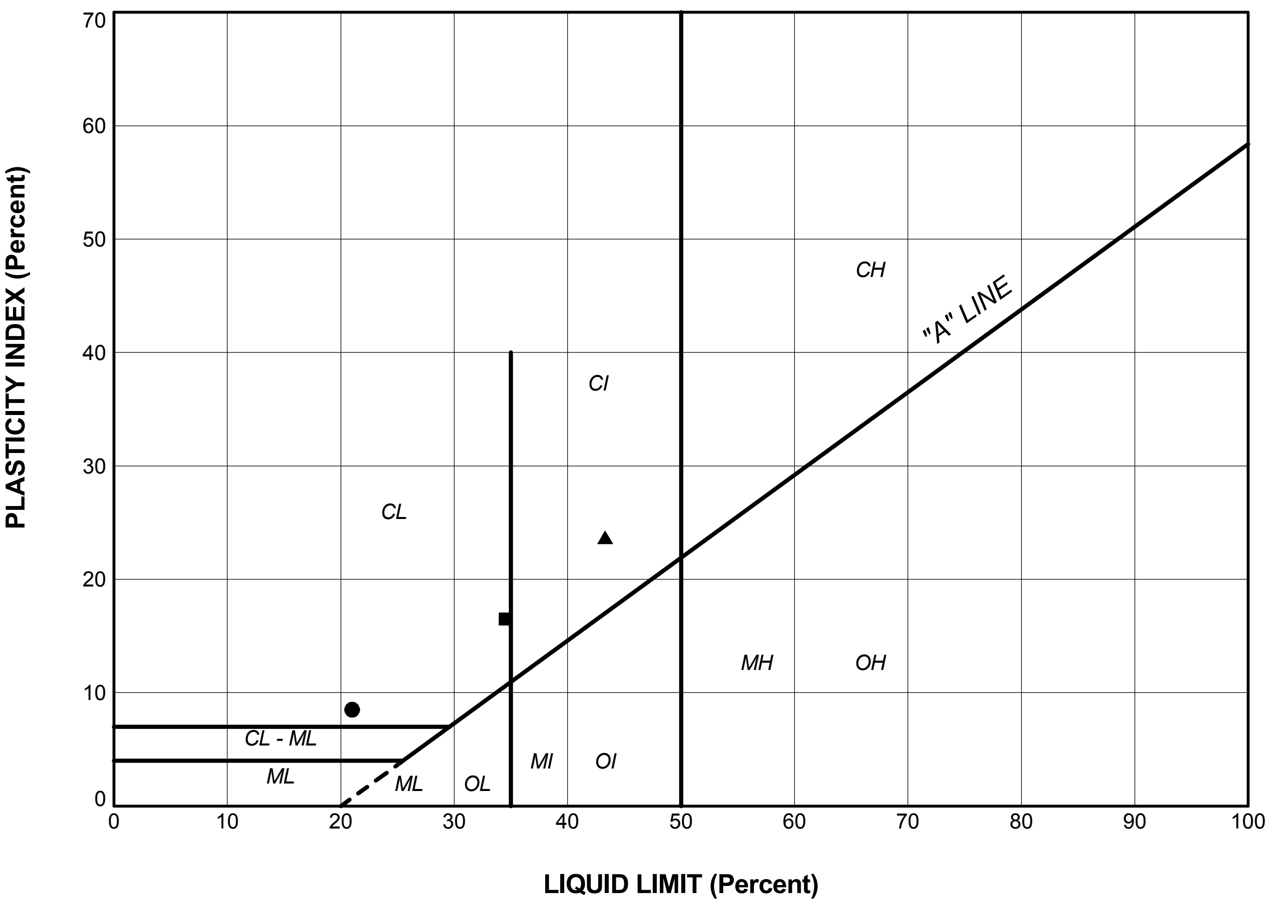
DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH: COLOUR / % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		TOP OF BEDROCK		208.8				
	NW August 29, 2015 NQ Coring	GRANITIC GNEISS Very coarse grained Very strong Pinkish grey Fresh		3.1	1	GREY 100%	JNIRRO JNIRRO JNIRRO	
4					2	GREY 100%	JNIRRO JNIRRO	UCS = 103 MPa
5							JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO JNIRRO	
6				205.4	3	GREY 100%	JNIRRO	
		END OF DRILLHOLE		6.5				
7								
8								
9								
10								
11								
12								
13								



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: MT
CHECKED: AB



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	15-1	2	21.0	12.5	8.5
■	15-1	5	34.5	18.0	16.5
▲	15-2	2	43.3	19.6	23.7

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

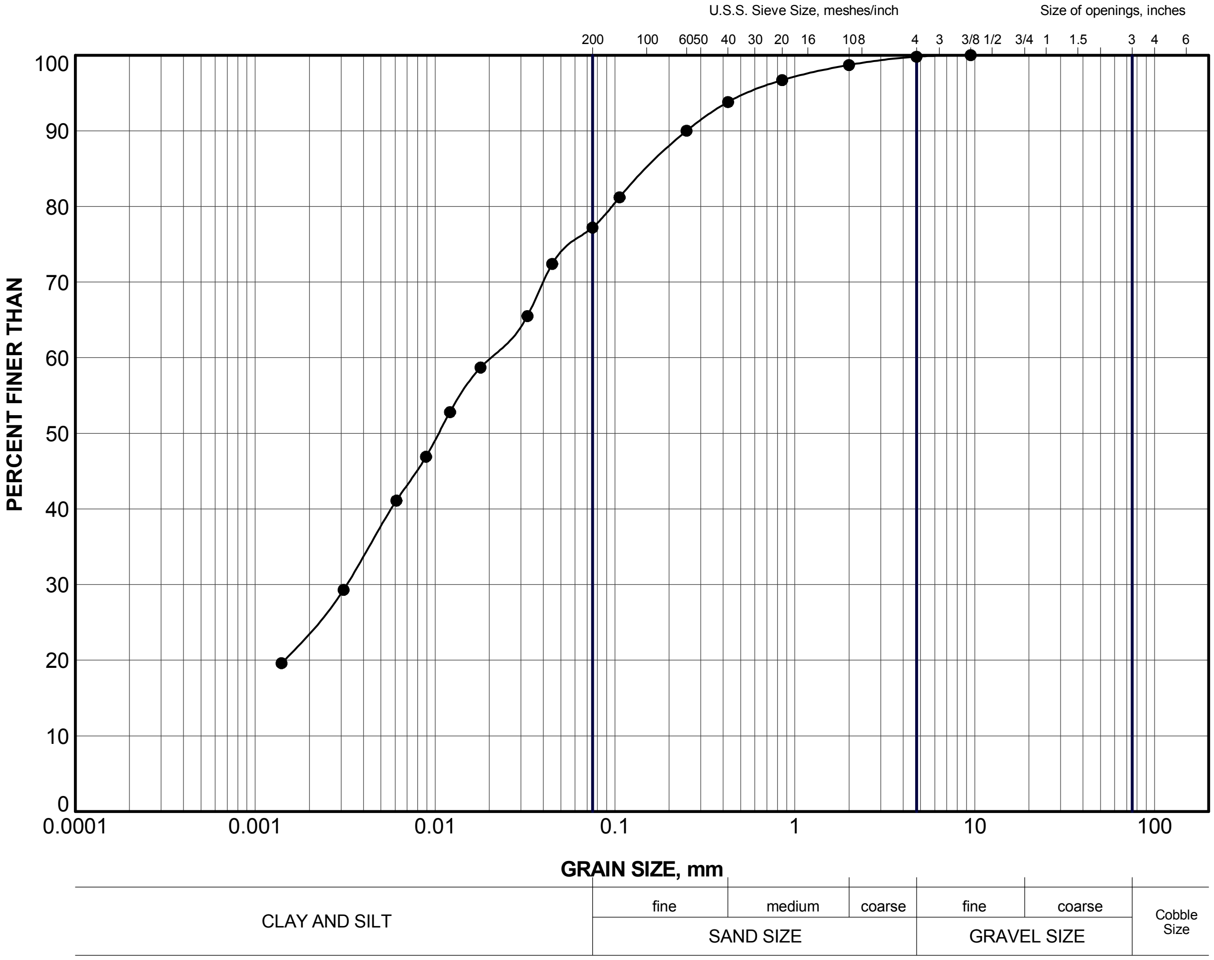
PLASTICITY CHART

SANDY CLAYEY SILT TO SILTY CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ	
DRAWN	JJL	Nov 2015	SCALE	N/A	REV.
CHECK	AB	Nov 2015			
APPR	JMAC	Nov 2015	**FIGURE A1**		

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	15-1	2	206.9

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

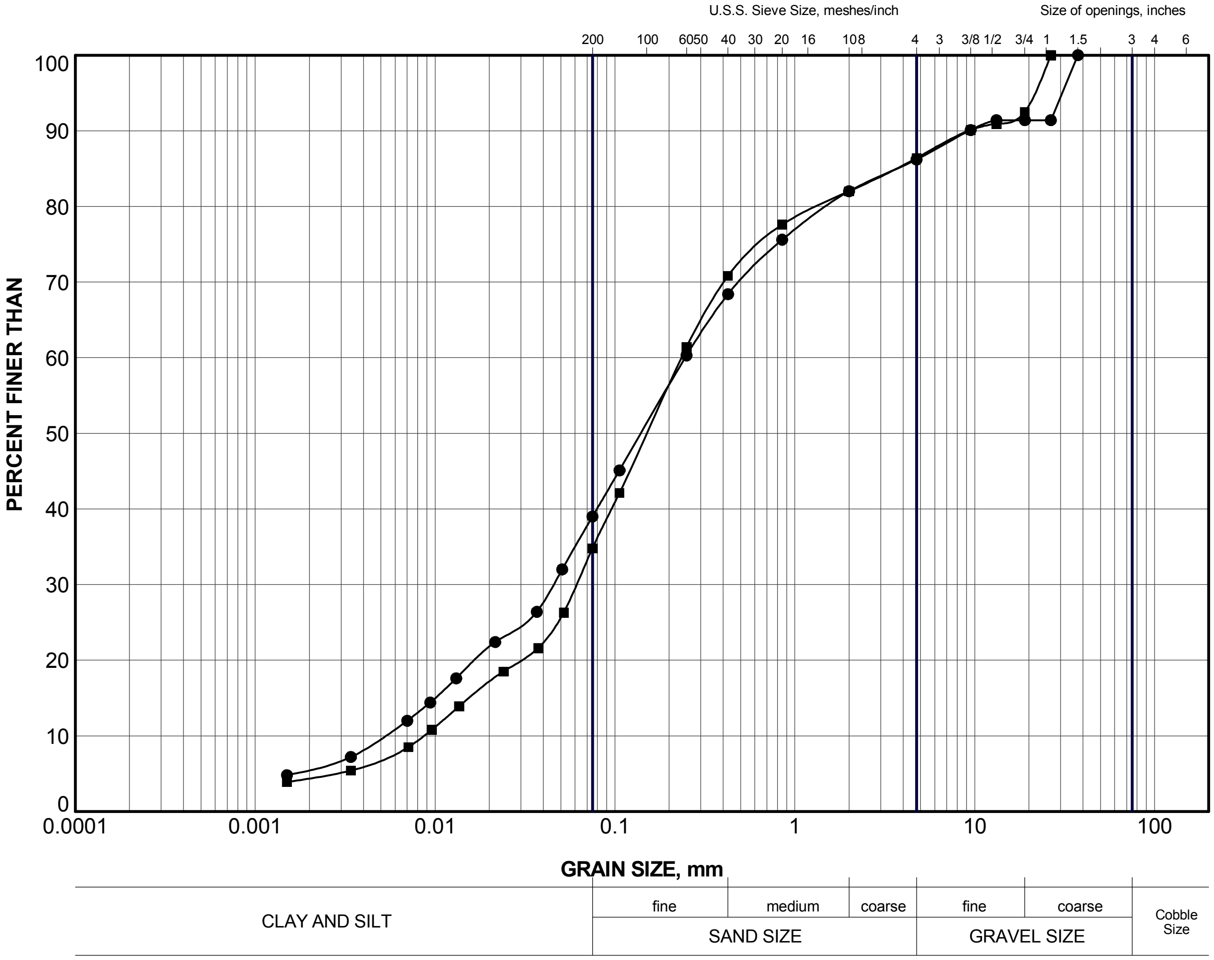
SANDY CLAYEY SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ
DRAWN	JJL	Nov 2015	SCALE N/A	REV.
CHECK	AB	Nov 2015		
APPR	JMAC	Nov 2015		

FIGURE A2

SUD-MTO GSD (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	15-1	8	200.8
■	15-2	7	201.6

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

SILT and SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ
DRAWN	JJL	Nov 2015	SCALE N/A	REV.
CHECK	AB	Nov 2015		
APPR	JMAC	Nov 2015	**FIGURE A3**	

SUD-MTO GSD (NEW) GLDR_LDN.GDT

GHR-1B

Box 1: 10.2 m – 13.4 m

0 m | 0.25 m | 0.5 m | 0.75 m | 1.0 m | 1.25 m | 1.5 m

0 ft | 1 ft | 2 ft | 3 ft | 4 ft | 5 ft

Scale

PROJECT

Groundhog River Bridge
Highway 11
GWP 5049-07-00

TITLE

Bedrock Core Photographs
Borehole GHR-1B

Golder Associates

PROJECT No.	11-1191-0025		FILE No.	----	
DESIGN	KP	SEP 15	SCALE	NTS	REV.
CADD	-- --				
CHECK	AB	SEP 15	**FIGURE A4**		
REVIEW	JMAC	SEP 15			

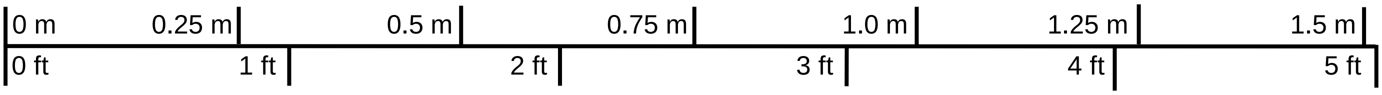
REVISION DATE: March 23, 2015 BY: MCK Project: 09-1111-6014

Borehole 15-1

Box 1: 16.3 m – 19.8 m

Borehole 15-3

Box 1: 3.1 m – 6.5 m



Scale

PROJECT
Groundhog River Bridge
Highway 11
GWP 5049-07-00

TITLE
Bedrock Core Photographs
Boreholes 15-1 and 15-3

Golder Associates

PROJECT No.	11-1191-0025		FILE No.	----	
DESIGN	KP	SEP 15	SCALE	NTS	REV.
CADD	-- --		**FIGURE A5**		
CHECK	AB	SEP 15			
REVIEW	JMAC	SEP 15			

REVISION DATE: March 23, 2015 BY: MCK Project: 09-1111-6014

APPENDIX B

Groundhog River Bridge – Previous Investigation

PROJECT 11-1191-0025

RECORD OF BOREHOLE No GHR-1

1 OF 1 **METRIC**

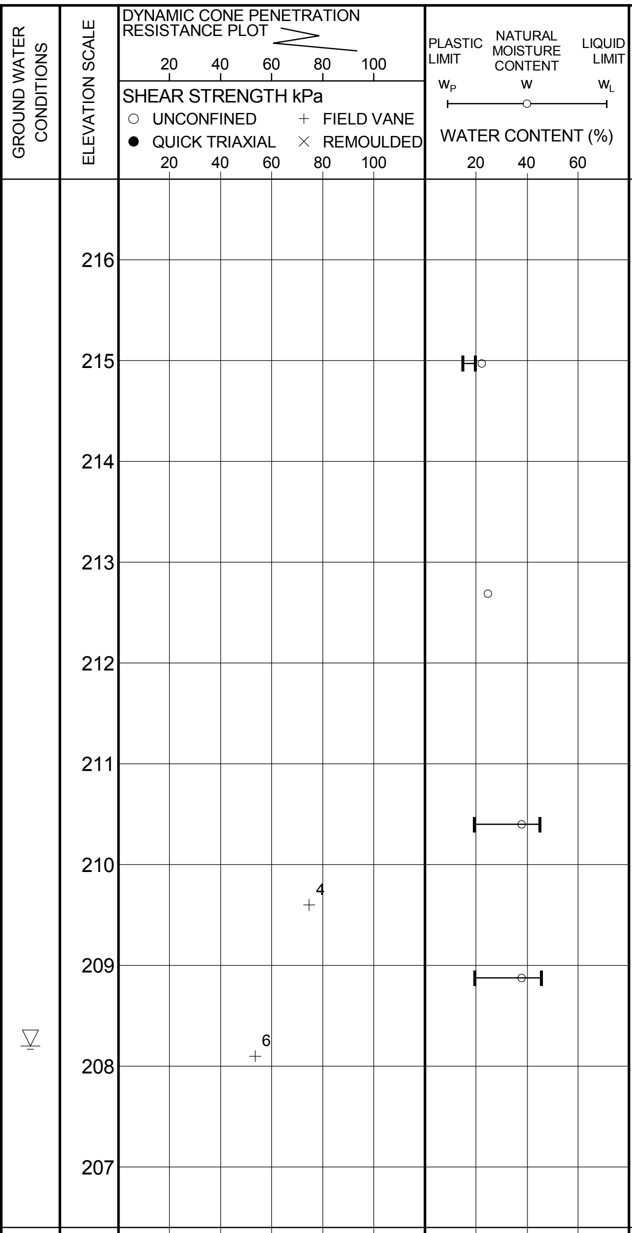
W.P. 5049-07-00
LOCATION N 5464438.5; E 229164.2
ORIGINATED BY EHS
DIST
HWY 11
BOREHOLE TYPE 108 mm I.D. HOLLOW STEM AUGERS
COMPILED BY AC
DATUM GEODETIC
DATE JULY 25, 2012
CHECKED BY AB

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
216.8	GROUND SURFACE
0.0	Silty topsoil (FILL)
0.2	Silt, some sand, trace gravel, trace organics (FILL) Loose to compact Brown Moist to wet
215.1 1.7	CLAYEY SILT, some sand, trace organics Very soft Brown Wet
214.6 2.2	SAND and SILT, some clay, trace organics, clay seams / layers Very loose Brown to grey Wet
211.2 5.6	SILTY CLAY, trace sand Firm to stiff Brown to grey Wet
207.3 9.5	Silty SAND, some gravel, trace clay Grey Wet
206.4 10.4	END OF BOREHOLE AUGER REFUSAL

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	11		
2	SS	8		
3	SS	WH	OC=0.5%	
4	SS	WH		
5	SS	WH		
6	SS	WH		0 53 34 13
7	SS	2		
8	SS	5		
9	SS	2		
10	TO	PH		
11	AS	-		



Note:

1. Water level at a depth of 8.6 m below ground surface (Elev. 208.2 m) upon completion of drilling.

2. On December 13, 2012, Borehole GHR-1a advanced 1.2 m north of Borehole GHR-1.

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

RECORD OF BOREHOLE No GHR-1a

1 OF 1 **METRIC**

PROJECT 11-1191-0025			
W.P. 5049-07-00	LOCATION N 5464439.7; E 229164.2	ORIGINATED BY ID
DIST	HWY 11	BOREHOLE TYPE HOLLOW STEM AUGERS, NW CASING, NQ CORING	COMPILED BY MT
DATUM GEODETIC		DATE December 13, 2012	CHECKED BY AB

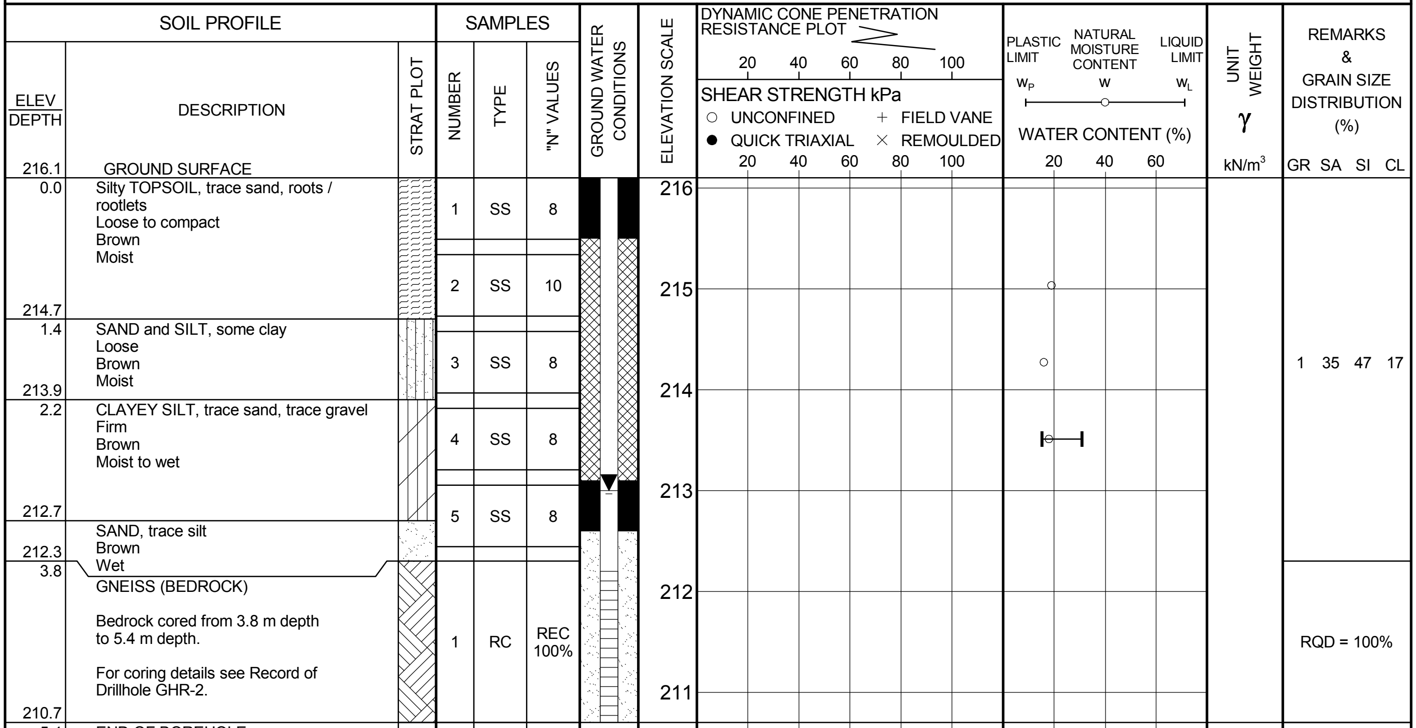
SOIL PROFILE ELEV / DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.8	GROUND SURFACE										
0.0	For stratigraphy details refer to Record of Borehole GHR-1.										
							216				
							215				
							214				
							213				
							212				
							211				
							210				
							209				
							208				
							207				
206.8 10.0	BOULDERS, gravel seams		1	RC	REC 100%						
			2	RC	REC 100%		206				
205.4 11.4	END OF BOREHOLE										

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

RECORD OF BOREHOLE No GHR-2

1 OF 1 **METRIC**

PROJECT 11-1191-0025
W.P. 5049-07-00
DIST
HWY 11
DATUM GEODETIC
LOCATION N 5464505.7; E 228986.5
BOREHOLE TYPE 108 mm I.D. HOLLOW STEM AUGERS
DATE JULY 25 and 26, 2012
ORIGINATED BY EHS
COMPILED BY AC
CHECKED BY AB



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.1	GROUND SURFACE				
0.0	Silty TOPSOIL, trace sand, roots / rootlets Loose to compact Brown Moist	1	SS	8	
		2	SS	10	
214.7 1.4	SAND and SILT, some clay Loose Brown Moist	3	SS	8	1 35 47 17
213.9 2.2	CLAYEY SILT, trace sand, trace gravel Firm Brown Moist to wet	4	SS	8	
		5	SS	8	
212.7	SAND, trace silt Brown Wet				
212.3 3.8	GNEISS (BEDROCK) Bedrock cored from 3.8 m depth to 5.4 m depth. For coring details see Record of Drillhole GHR-2.	1	RC	REC 100%	RQD = 100%
210.7 5.4	END OF BOREHOLE				

Note:

1. Water level at a depth of 2.4 m below ground surface (Elev. 213.7 m) upon completion of drilling.

2. Water level in piezometer at a depth of 3.1 m (Elev. 213.0 m) on August 1, 2012.

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

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

PROJECT: 11-1191-0025

RECORD OF DRILLHOLE: GHR-2

SHEET 1 OF 1

LOCATION: N 5464505.7 ;E 228986.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: JULY 25 and 26, 2012

DRILL RIG: CME-850

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.	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE		212.3																
4 5	NW NQ Coring JULY 26, 2012	GNEISS Very coarse grained Fresh Strong Grey		3.8 210.7	1	GREY 100%							JIR							UCS=97 MPa
6 7 8 9 10 11 12 13		END OF DRILLHOLE		5.4																

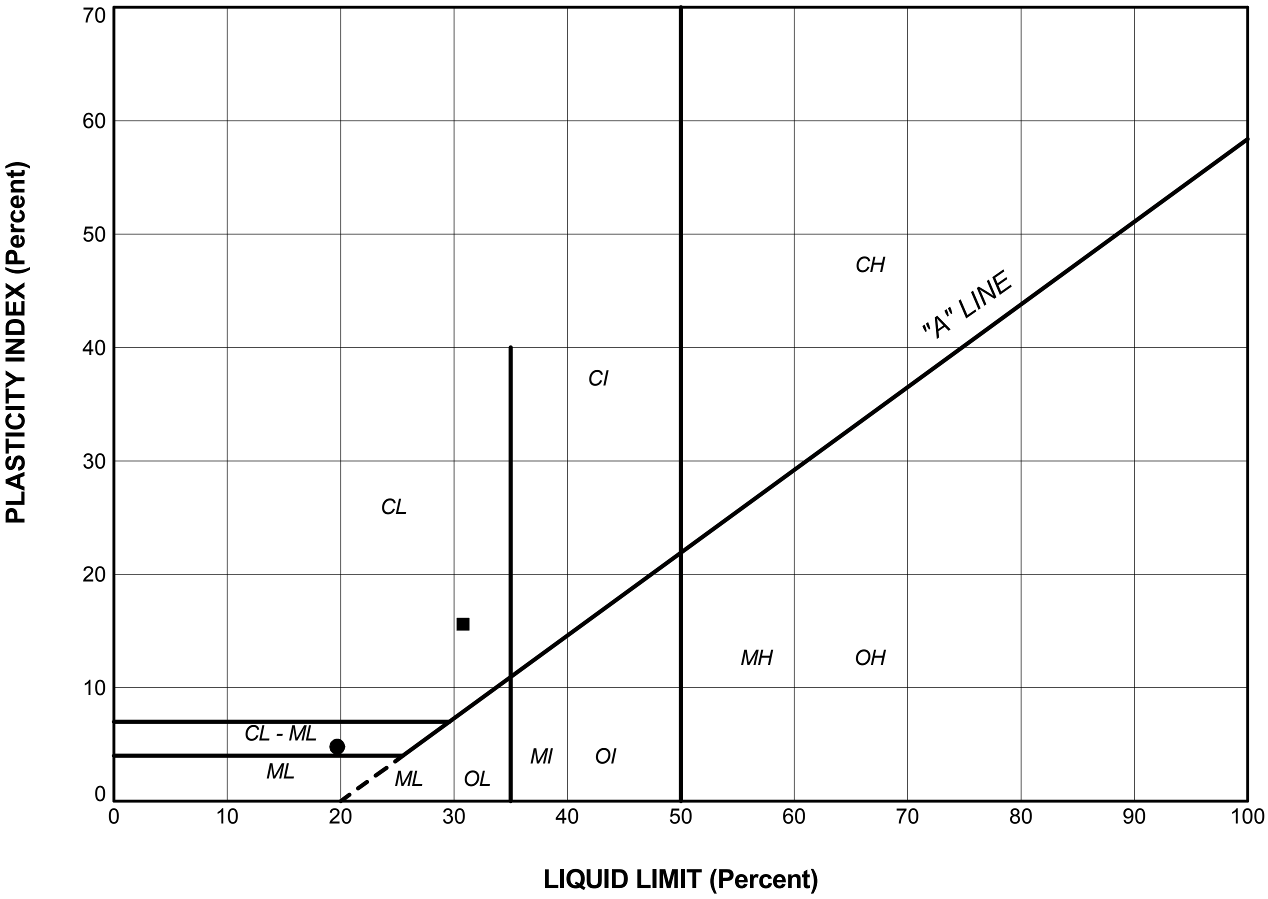
DEPTH SCALE

1 : 50

LOGGED: EHS

CHECKED: AB

SUD-RCK 11-1191-0025.GPJ GAL-MISS.GDT 03/05/13 DATA INPUT:



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	GHR-1	3	20	15	5
■	GHR-2	4	31	15	16

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

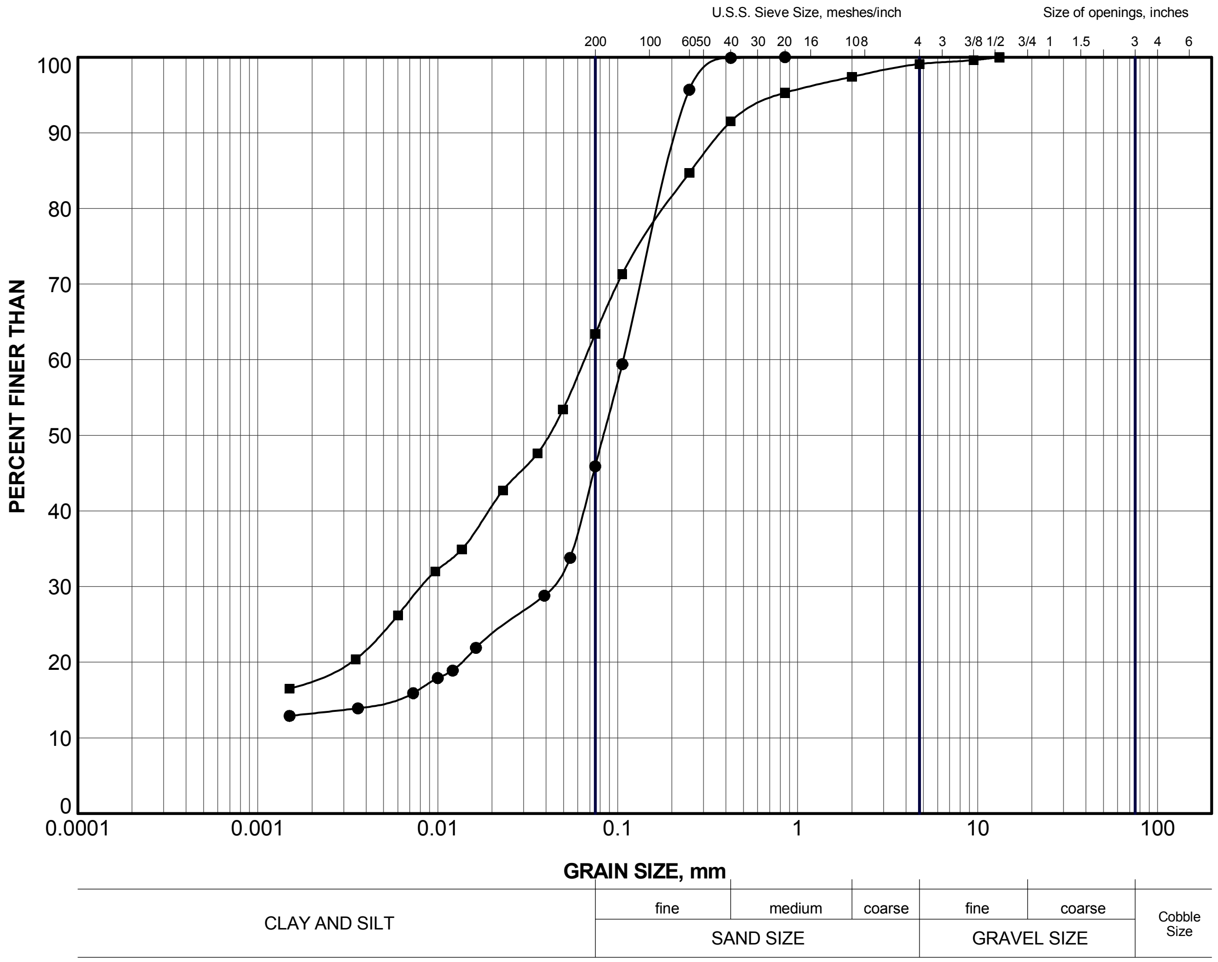
PLASTICITY CHART

CLAYEY SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ
DRAWN	JJL	Nov 2012	SCALE N/A	REV.
CHECK	AB	Nov 2012		
APPR	JMAC	Nov 2012	**FIGURE B1**	

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	GHR-1	6	212.7
■	GHR-2	3	214.3

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

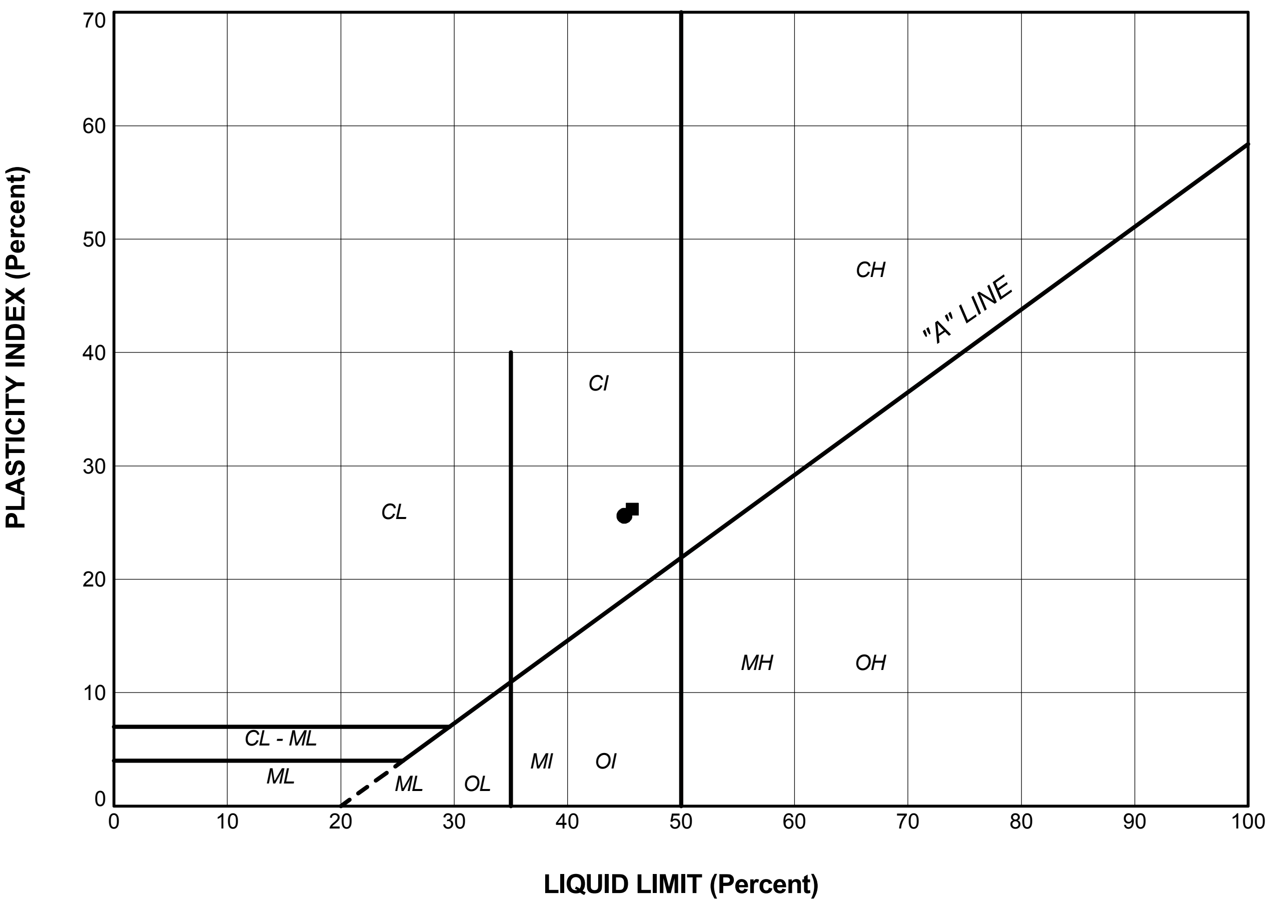
GRAIN SIZE DISTRIBUTION

SAND AND SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		11-1191-0025	FILE No.	11-1191-0025.GPJ	
DRAWN	JJL	Nov 2012	SCALE	N/A	REV.
CHECK	AB	Nov 2012	FIGURE B2		
APPR	JMAC	Nov 2012			

SUD-MTO GSD (NEW) 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
●	GHR-1	8	45	19	26
■	GHR-1	9	46	20	26

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

PLASTICITY CHART

SILTY CLAY

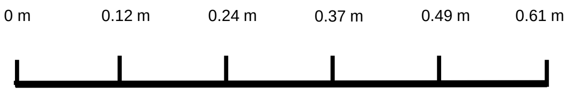
Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ
DRAWN	JJL	Nov 2012	SCALE N/A	REV.
CHECK	AB	Nov 2012		
APPR	JMAC	Nov 2012	**FIGURE B3**	

SUD-MTO PL (NEW) GLDR_LDN.GDT

Borehole GHR-1
Elevation 206.8 m to 205.4 m

Borehole GHR-1a
Elevation 212.3 m to 210.7 m



PROJECT
Groundhog River Bridge
Highway 11
GWP 5049-07-00

TITLE
Bedrock Core Photographs
Borehole GHR-1A

Golder Associates

PROJECT No.	11-1191-0025		FILE No.	----
DESIGN	AC	Seo 2015	SCALE AS SHOWN	REV.
CADD	--		**FIGURE B4**	
CHECK	AB	Sep 2015		
REVIEW	JMAC	Sep 2015		

APPENDIX C

Culvert at STA 10+926

MINISTRY OF TRANSPORTATION, ONTARIO

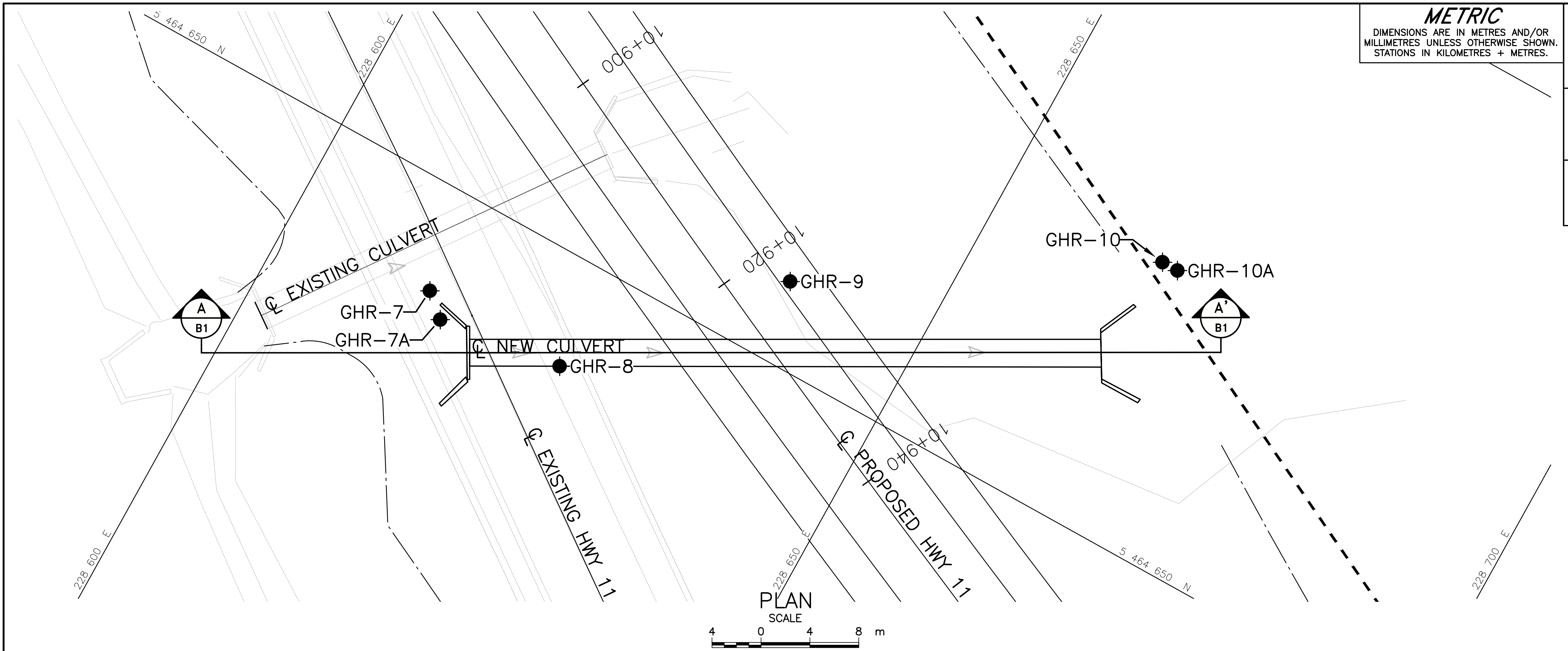
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

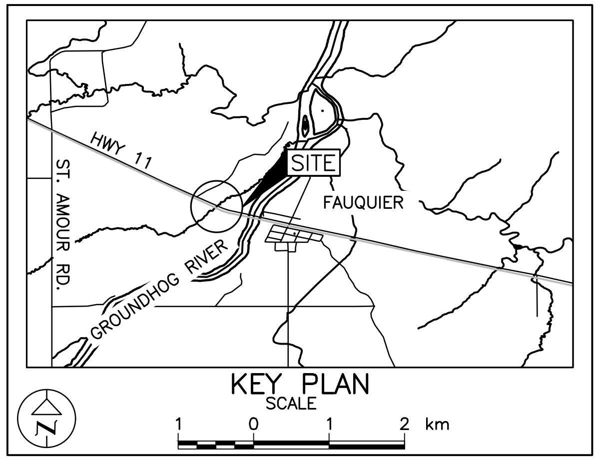
CONT No.
GWP No.5049-07-00

HIGHWAY 11
CULVERT AT STA 10+926
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates





LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
R	Refusal
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
REC %	Recovery
≚	WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
GHR−7	223.8	5464641.6	228612.9
GHR−7A	223.8	5464640.0	228614.8
GHR−8	223.7	5464641.4	228625.2
GHR−9	216.1	5464656.6	228638.3
GHR−10	216.0	5464672.8	228664.2
GHR−10A	216.0	5464672.8	228665.6

NOTES

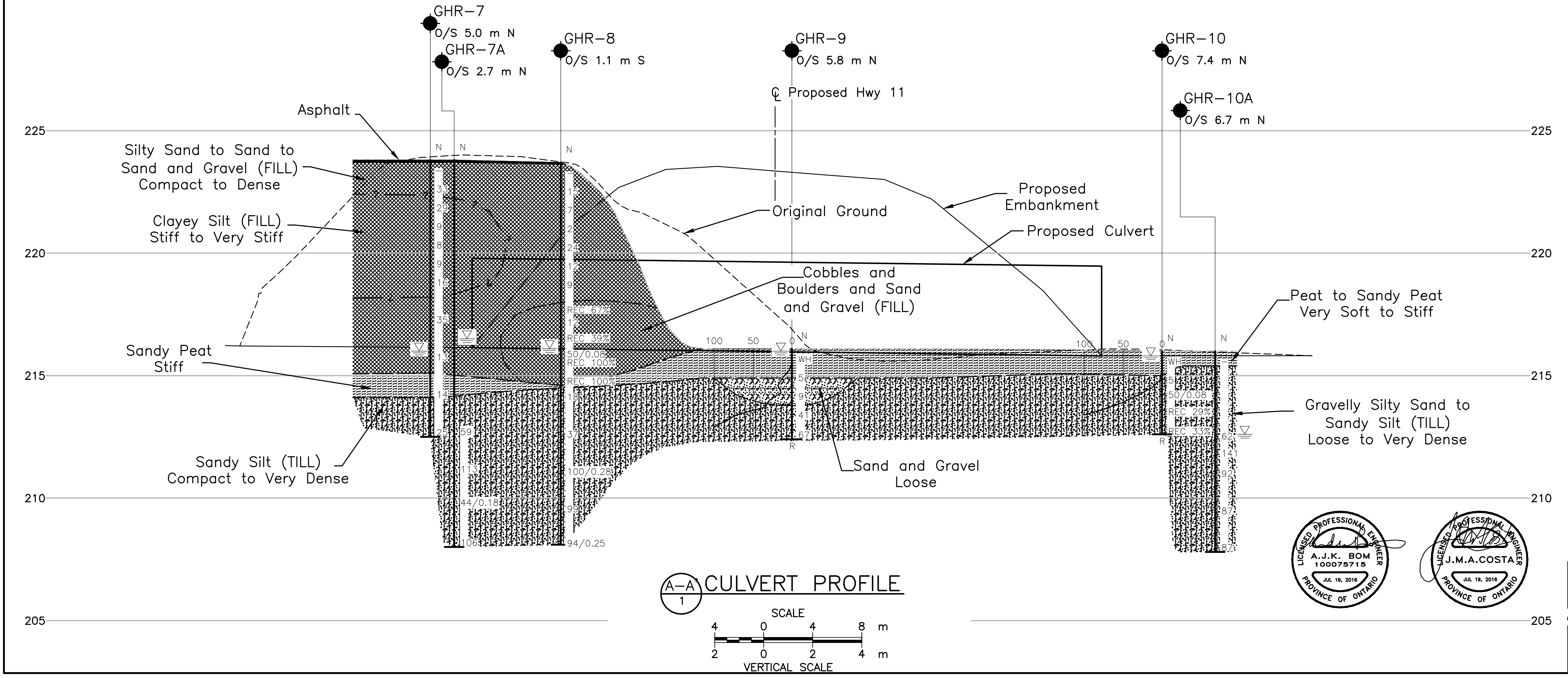
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. GA_Groundhog River_3B.dwg, received APR 12, 2013.



A.J.K. BOM
100075715
JUL 19, 2016

J.M.A.COSTA
JUL 19, 2016

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 42G−62

HWY. 11		PROJECT NO. 11−1191−0025	DIST. .
SUBM'D. MT	CHKD. .	DATE: 7/19/2016	SITE: 39W−093
DRAWN: JJL	CHKD. AB	APPD. JMAC	DWG. C1

PLOT DATE: July 20, 2016
FILENAME: H:\Projects\2011\11−1191−0025 hwy 11 onr and ghr\2−GHR\11119100025AA004.dwg

RECORD OF BOREHOLE No GHR-7

1 OF 1 **METRIC**

PROJECT 11-1191-0025

G.W.P. 5049-07-00

DIST ___ HWY 11

DATUM GEODETIC

LOCATION N 5464641.6; E 228612.9

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

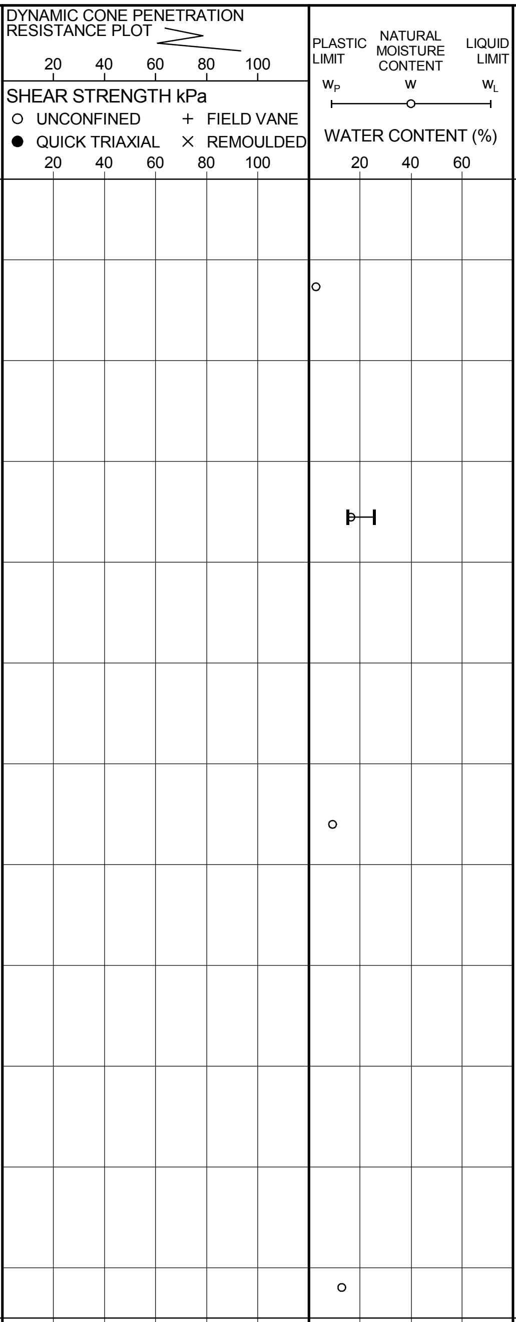
DATE November 19, 2014

ORIGINATED BY TM

COMPILED BY MT

CHECKED BY AB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	UNIT WEIGHT γ kN/m³
223.8	GROUND SURFACE						
0.0	ASPHALT (100 mm)						
0.1	Sand and gravel, trace silt (FILL) Dense Brown Moist	1	AS	-			
		2	SS	33			
222.4 1.4	Clayey silt, some sand (FILL) Stiff to very stiff Brown Moist	3	SS	29			
		4	SS	9			
		5	SS	8		0 15 62 23	
		6	SS	9			
		7	SS	16			
218.2 5.6	Sand to sand and gravel, trace to some silt, trace to some clay (FILL) Compact to dense Brown to grey Moist to wet	8	SS	35			
		9	SS	13	▽		
215.1 8.7	Sandy PEAT (Amorphous) Stiff Dark brown Wet	10	SS	14			
214.1 9.7	SILT and SAND, some clay, trace gravel (TILL) Compact Grey Wet	11	SS	25		2 38 47 13	NP
212.5 11.3	END OF BOREHOLE REFER TO RECORD OF BOREHOLE GHR-7A						



Note:

1. Water level at a depth of 7.8 m below ground surface (Elev. 216.0 m) upon completion of drilling.

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

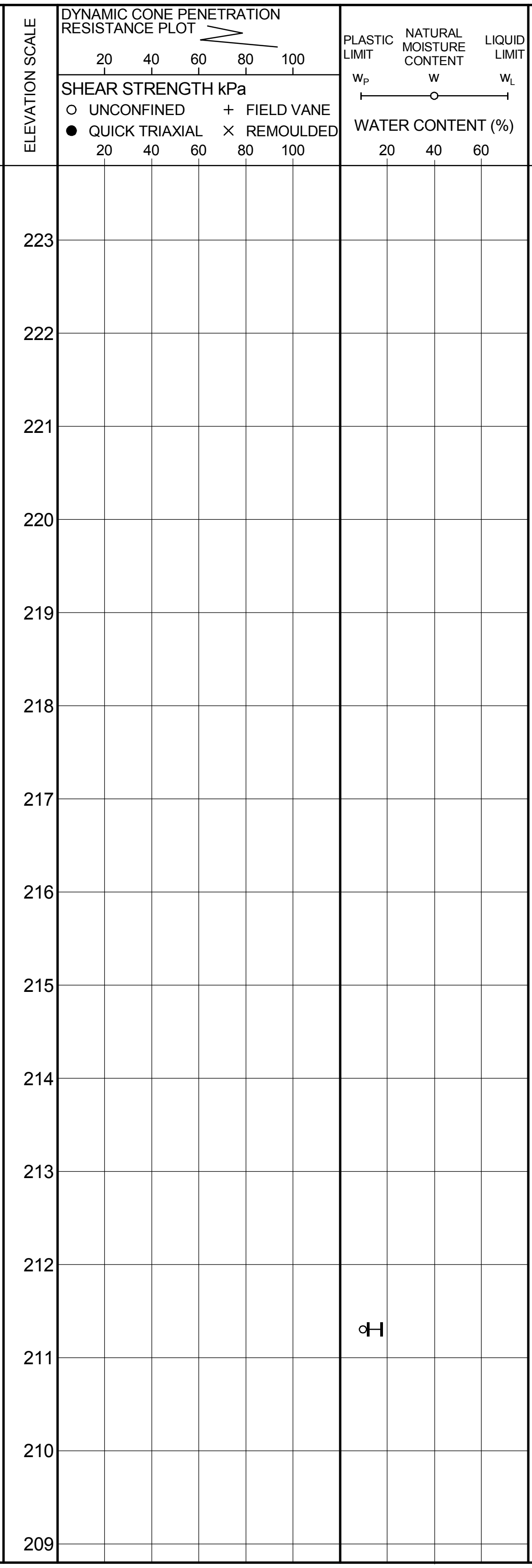
$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No GHR-7A

1 OF 2 **METRIC**

PROJECT 11-1191-0025
G.W.P. 5049-07-00
LOCATION N 5464640.0; E 228614.8
ORIGINATED BY MT
DIST HWY 11
BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring
COMPILED BY MT
DATUM GEODETIC
DATE September 2, 2015
CHECKED BY AB

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – "N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
223.8	GROUND SURFACE										
0.0	For stratigraphy above 10.7 m depth, refer to Record of Borehole GHR-7.										
						∇					
213.1 10.7	Sandy SILT, some clay, trace gravel (TILL) Very dense Grey Wet		1	SS	59						
			2	SS	113			2	25	56	17
			3	SS	44/0.18						
	125 mm and 75 mm cobbles at 14.0 m and 14.3 m depth, respectively.										



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

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

Golder Associates

RECORD OF BOREHOLE No GHR-7A

2 OF 2 **METRIC**

PROJECT 11-1191-0025			
G.W.P. 5049-07-00	LOCATION N 5464640.0; E 228614.8	ORIGINATED BY MT
DIST HWY 11	BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring	COMPILED BY MT
DATUM GEODETIC	DATE September 2, 2015	CHECKED BY AB

SOIL PROFILE			SAMPLES			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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
208.0			4	SS	106						
15.8	END OF BOREHOLE Note: 1. Water level at a depth of 7.3 m below ground surface (Elev. 216.5 m) upon completion of drilling.						208				

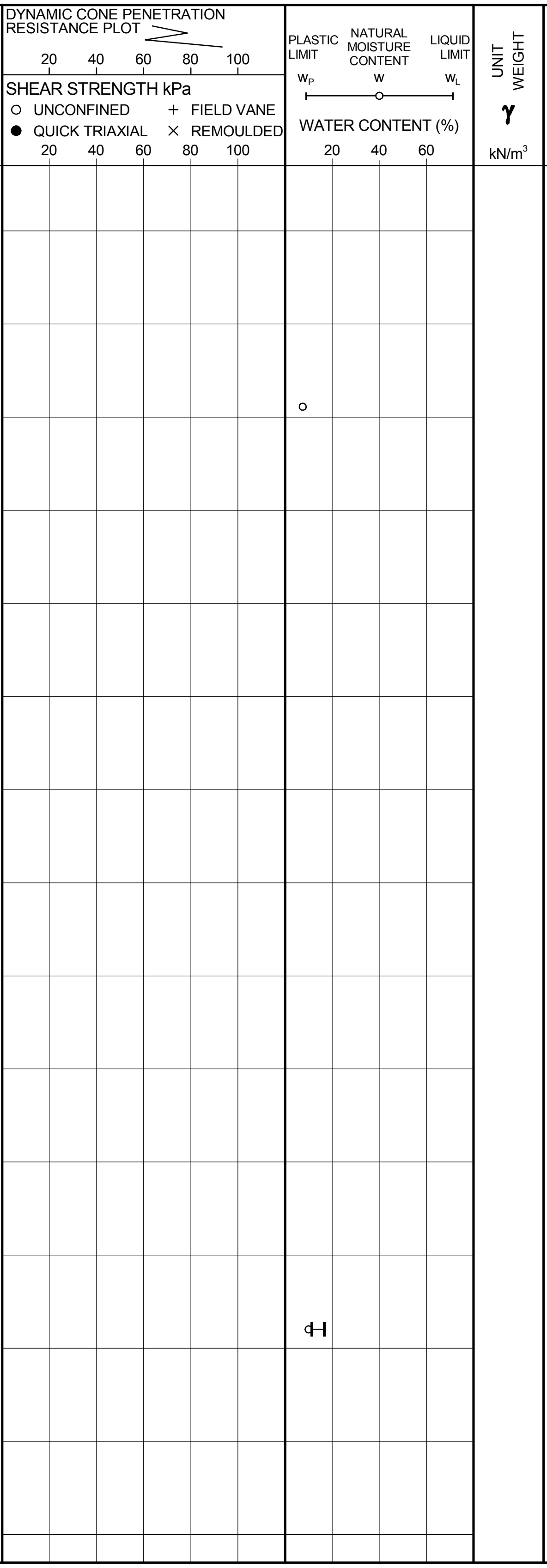
+³, ×³: Numbers refer to Sensitivity   ○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No GHR-8

1 OF 2 **METRIC**

PROJECT 11-1191-0025
G.W.P. 5049-07-00
LOCATION N 5464641.4; E 228625.2
ORIGINATED BY MT
DIST HWY 11
BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring
COMPILED BY MT
DATUM GEODETIC
DATE August 27 and 28, 2015
CHECKED BY AB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
223.7	GROUND SURFACE									
0.0 0.1	ASPHALT (65 mm)									
	Silty sand and gravel to sand (FILL) Compact Brown Moist	1	AS	-						
						223				
		2	SS	15						
						222				
		3	SS	7						
		4	SS	2		221	38	32	23	7
		5	SS	24						
						220				
		6	SS	12						
						219				
		7	SS	9						
218.2 5.5	Cobbles and Boulders and sand and gravel (FILL)									
		-	RC	REC 67%		218				
		8	SS	12						
						217				
		-	RC	REC 39%						
					▽					
		9	SS	50/0.08		216				
		-	RC	REC 100%						
						215				
		-	RC	REC 100%						
214.6	PEAT (Amorphous)									
9.2	SILT and SAND, some clay, trace gravel (TILL) Compact to very dense Grey Wet	10	SS	18						
						214				
						213				
		11	SS	37						
						212				
		12	SS	100/0.28			3	34	48	15
						211				
						210				
		13	SS	99						
						209				



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

RECORD OF BOREHOLE No GHR-8

2 OF 2 **METRIC**

PROJECT 11-1191-0025
G.W.P. 5049-07-00
DIST
HWY 11
DATUM GEODETIC
LOCATION N 5464641.4; E 228625.2
BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring
DATE August 27 and 28, 2015
ORIGINATED BY MT
COMPILED BY MT
CHECKED BY AB

SOIL PROFILE			SAMPLES			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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
			14	SS	94/0.25						
208.1 15.6	END OF BOREHOLE Note: 1. Water level at a depth of 7.6 m below ground surface (Elev. 216.1 m) upon completion of drilling.										

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

SUD-MTO 001 11-1191-0025.GPJ GAL-MISS.GDT 03/12/15 DATA INPUT:

RECORD OF BOREHOLE No GHR-9

1 OF 1 **METRIC**

PROJECT 11-1191-0025

G.W.P. 5049-07-00

LOCATION N 5464656.6; E 228638.3

ORIGINATED BY TM

DIST ___ HWY 11

BOREHOLE TYPE Portable Equipment

COMPILED BY MT

DATUM GEODETIC

DATE November 19, 2014

CHECKED BY AB

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
216.1	GROUND SURFACE										
0.0	PEAT (Fibrous) Very soft to firm Black Wet	1	SS	WH	▽	173.6					
		2	SS	5							
214.9 1.2	SAND and GRAVEL Loose Brown to grey Wet	3	SS	9							
213.9 2.2	SILT and SAND, some clay, trace gravel (TILL) Dense to very dense Grey Wet	4	SS	41				3	39	44	14
		5	SS	67							
212.4 3.7	END OF BOREHOLE REFUSAL TO FURTHER CASING ADVANCEMENT										

Note:

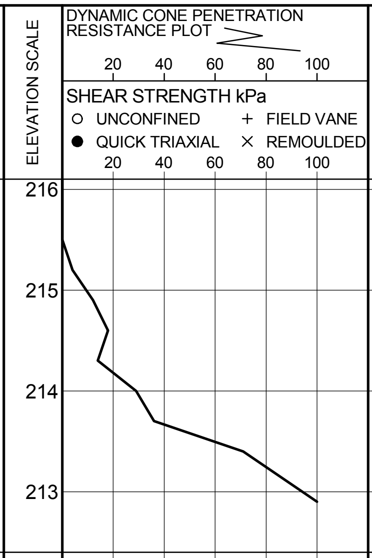
1. Water level at a depth of 0.1 m below ground surface (Elev. 216.0 m) upon completion of drilling.

2. Dynamic cone penetration test advanced 0.5 m south of borehole.

DYNAMIC CONE PENETRATION RESISTANCE PLOT

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L



+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

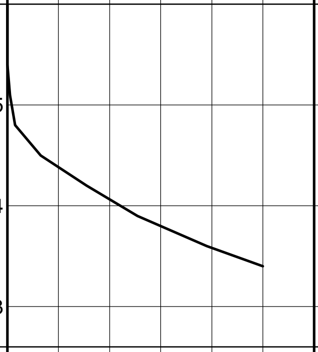
Golder Associates

RECORD OF BOREHOLE No GHR-10

1 OF 1 **METRIC**

PROJECT 11-1191-0025
G.W.P. 5049-07-00
LOCATION N 5464672.8; E 228664.2
ORIGINATED BY TM
DIST ____ HWY 11
BOREHOLE TYPE Portable Equipment
COMPILED BY MT
DATUM GEODETIC
DATE November 18, 2014
CHECKED BY AB

SOIL PROFILE			SAMPLES			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	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
216.0	GROUND SURFACE													
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	WH	∇			○ 319.6					
215.0 1.0	Gravelly Silty SAND, trace to some clay (TILL) Very dense Grey Wet		2	SS	5		215							
			3	SS	50/0.08				○		23	45	26	6
			-	RC	REC 29%		214							
212.6			-	RC	REC 33%		213							
3.4	END OF BOREHOLE REFUSAL TO FURTHER CASING ADVANCEMENT REFER TO RECORD OF BOREHOLE GHR-10A													



Note:

1. Water level at a depth of 0.2 m below ground surface (Elev. 215.8 m) upon completion of drilling.

2. Dynamic cone penetration test advanced 1.0 m southwest of borehole.

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

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

RECORD OF BOREHOLE No GHR-10A

1 OF 1 **METRIC**

PROJECT 11-1191-0025

G.W.P. 5049-07-00

LOCATION N 5464672.8; E 228665.6

ORIGINATED BY MT

DIST

HWY 11

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

COMPILED BY MT

DATUM GEODETIC

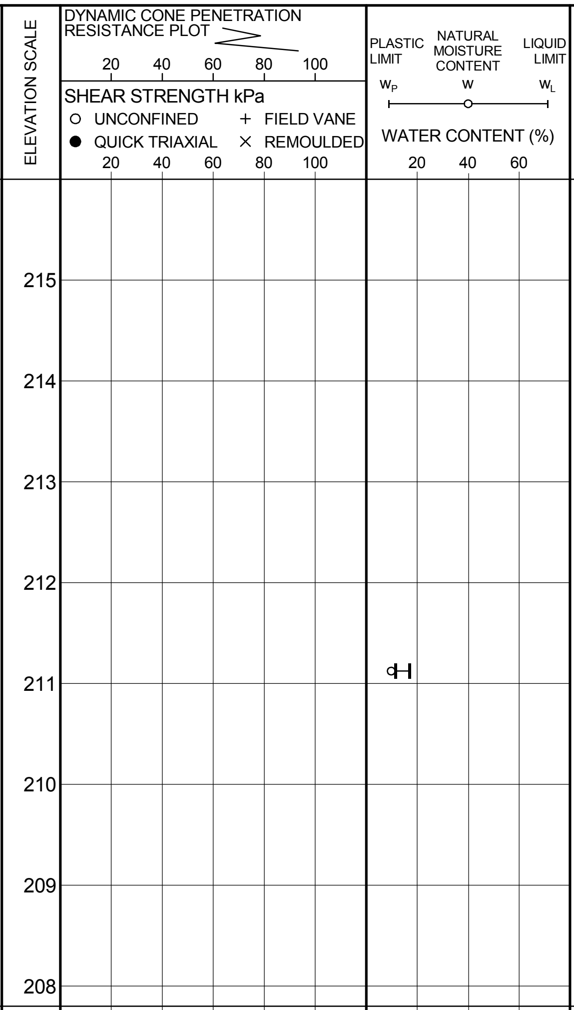
DATE September 1, 2015

CHECKED BY AB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
216.0	GROUND SURFACE									
0.0	PEAT (Fibrous) Black Wet	1	AS	-						
215.4 0.6	For stratigraphy between 0.6 m and 3.0 m depth, refer to Record of Borehole GHR-10.									
213.0 3.0	Sandy SILT, some clay, trace gravel (TILL) Very dense Grey Wet	2	SS	62	∇					
		3	SS	141						
		4	SS	92			2	26	56	16
		5	SS	87						
		6	SS	87						
207.8 8.2	END OF BOREHOLE									

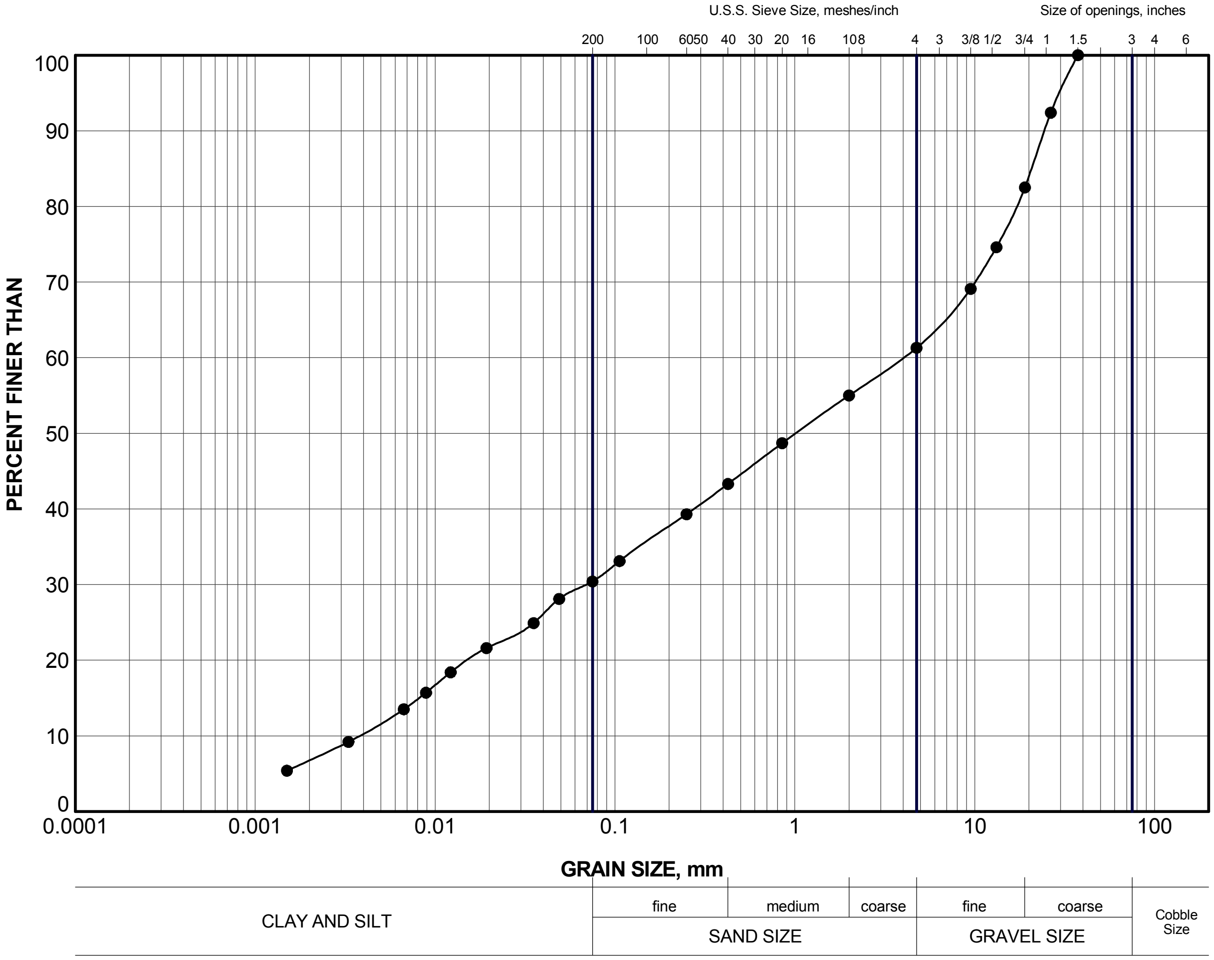
Note:

1. Water level at a depth of 3.4 m below ground surface (Elev. 212.6 m) upon completion of drilling.



+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	GHR-8	4	221.1

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

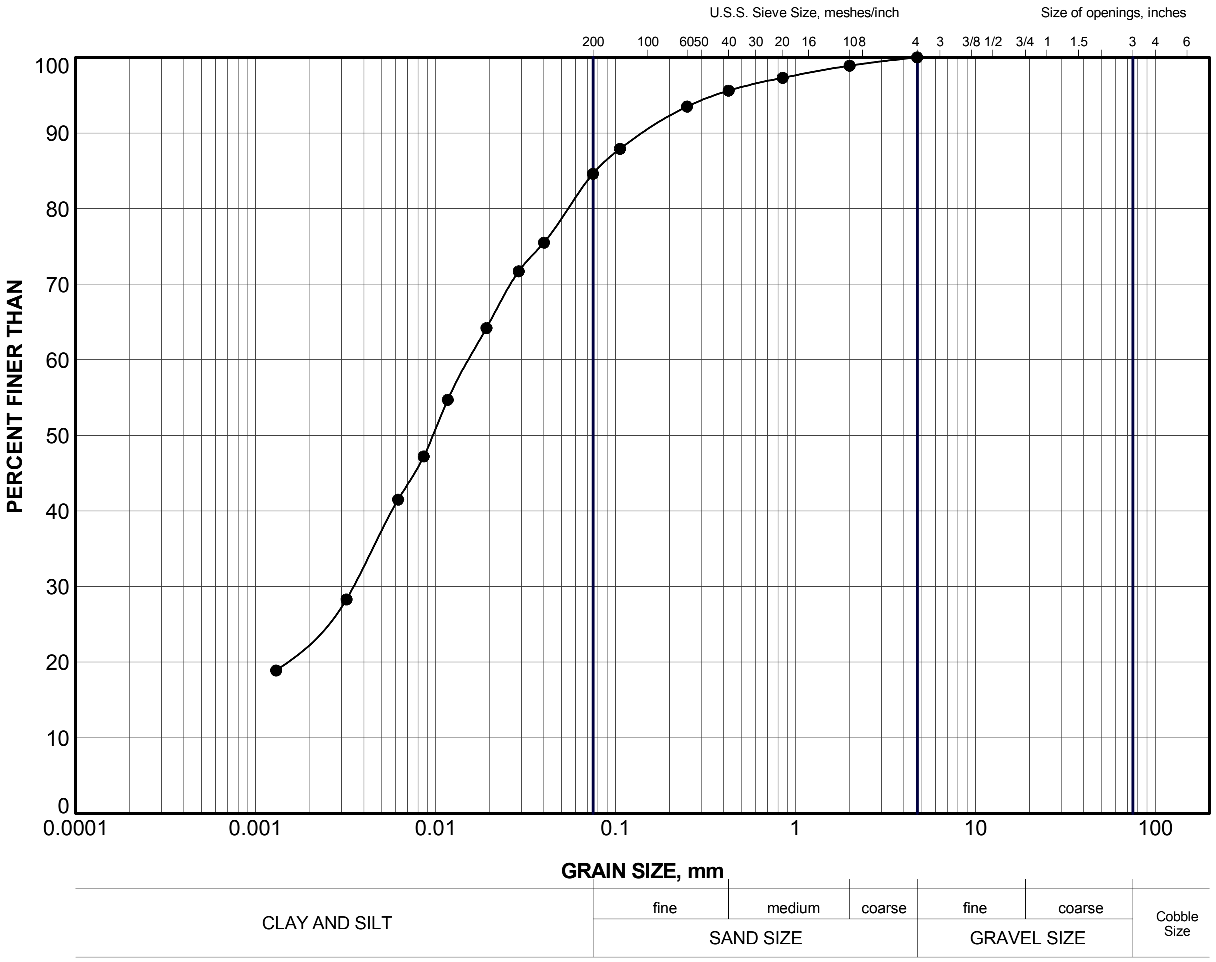
SILTY SAND and GRAVEL (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ
DRAWN	JJL	Nov 2015	SCALE N/A	REV.
CHECK	AB	Nov 2015		
APPR	JMAC	Nov 2015		

FIGURE C1

SUD-MTO GSD (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	GHR-7	5	220.4

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

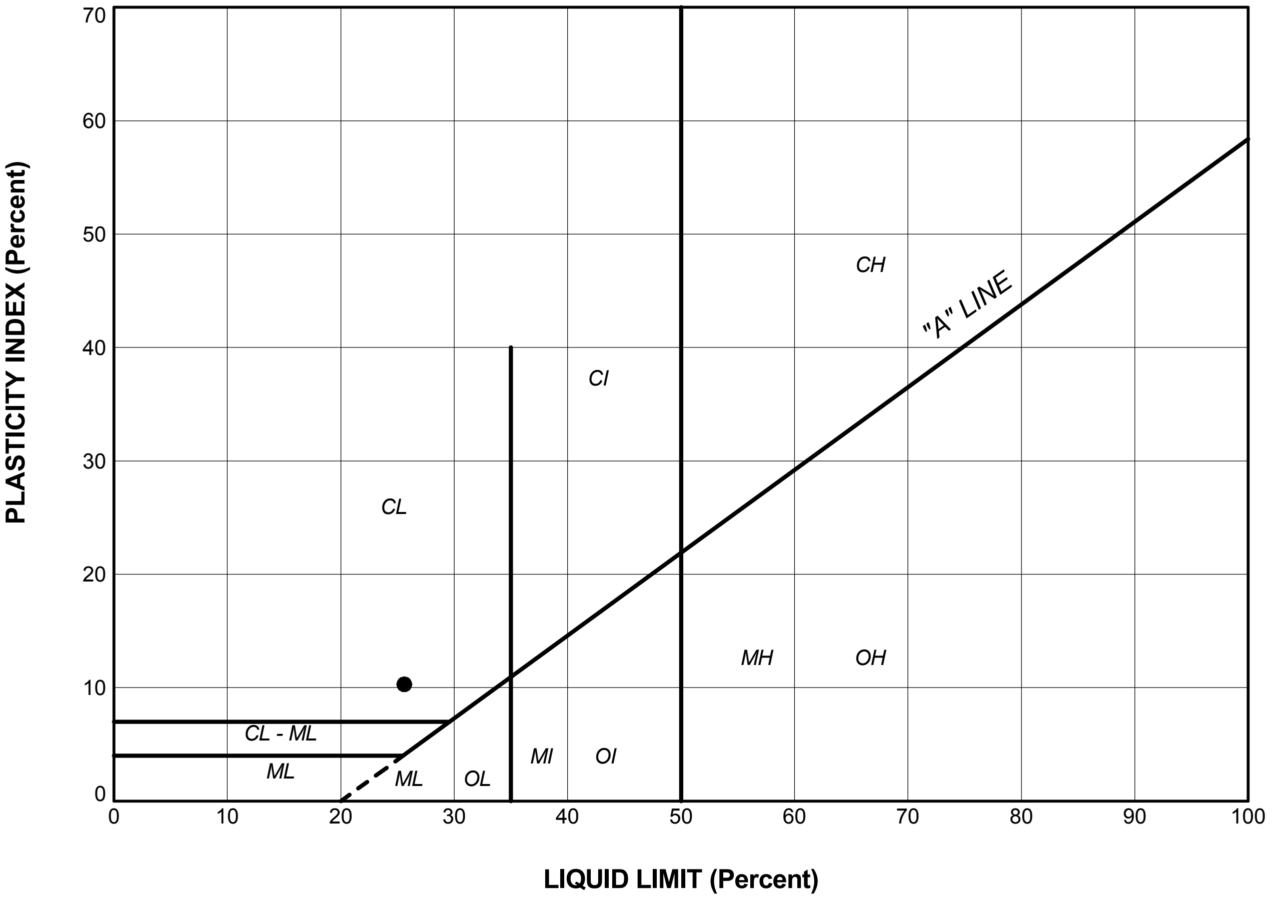
GRAIN SIZE DISTRIBUTION

CLAYEY SILT (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ
DRAWN	JJL	Nov 2015	SCALE N/A	REV.
CHECK	AB	Nov 2015		
APPR	JMAC	Nov 2015	**FIGURE C2**	

SUD-MTO GSD (NEW) 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
●	GHR-7	5	25.6	15.3	10.3

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

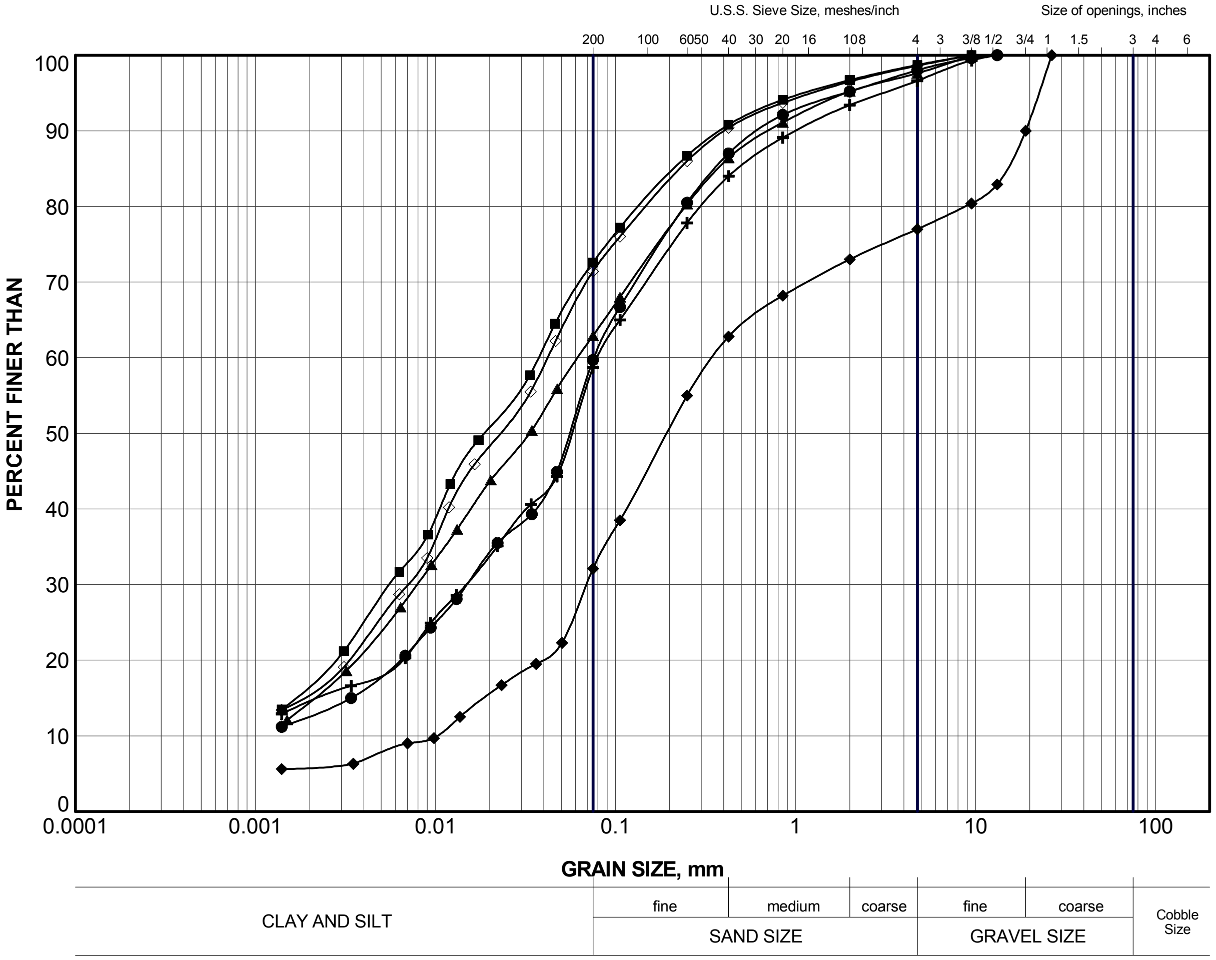
TITLE

PLASTICITY CHART

CLAYEY SILT (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ	
DRAWN	JJL	Nov 2015	SCALE	N/A	REV.
CHECK	AB	Nov 2015			
APPR	JMAC	Nov 2015	**FIGURE C3**		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	GHR-7	11	212.8
■	GHR-7A	2	211.3
▲	GHR-8	12	211.2
+	GHR-9	4	213.5
◆	GHR-10	3	214.3
◇	GHR-10A	4	211.1

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

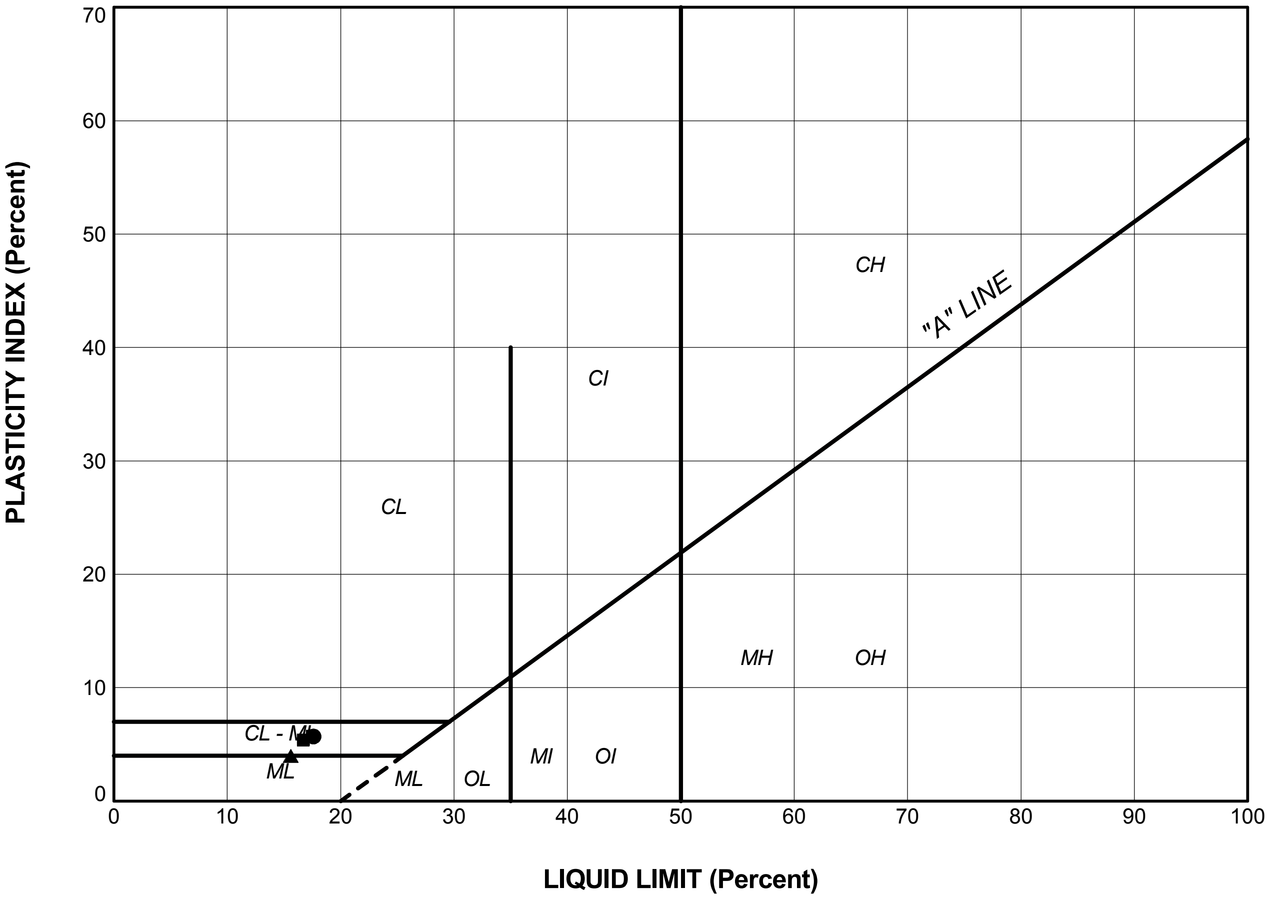
TITLE

GRAIN SIZE DISTRIBUTION

SANDY SILT to GRAVELLY SILTY SAND (TILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ	
DRAWN	JJL	Nov 2015	SCALE	N/A	REV.
CHECK	AB	Nov 2015			
APPR	JMAC	Nov 2015	**FIGURE C4**		



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	GHR-7A	2	17.6	11.9	5.7
■	GHR-8	12	16.7	11.3	5.4
▲	GHR-9	4	15.6	11.6	4.0
+	GHR-10A	4	17.1	11.5	5.6

PROJECT

HIGHWAY 11
GROUNDHOG RIVER BRIDGE

TITLE

PLASTICITY CHART

SANDY SILT to GRAVELLY SILTY SAND (TILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0025		FILE No.	11-1191-0025.GPJ
DRAWN	JJL	Nov 2015	SCALE N/A	REV.
CHECK	AB	Nov 2015		
APPR	JMAC	Nov 2015	**FIGURE C5**	

SUD-MTO PL (NEW) GLDR_LDN.GDT

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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