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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
DETAIL DESIGN
GULL LAKE NARROWS BRIDGE
SOUTHBOUND LANE STRUCTURE WIDENING
HIGHWAY 11 / MUSKOKA ROAD 169 INTERCHANGE
GRAVENHURST, ONTARIO
G.W.P. 314-00-00

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PART A

FOUNDATION INVESTIGATION REPORT
DETAIL DESIGN
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) to provide foundation engineering services for the following components for the Highway 11 Interchange with Muskoka Road 169 (G.W.P. 314-00-00) in Gravenhurst, Ontario:

- Rehabilitation of the existing Gull Lake Narrows Northbound and Southbound Lane Bridges and proposed widening of the Southbound Lane Bridge structure;
- Highway 11 and Pinedale Road/Hewitt Street underpass structure;
- Highway 11 and Muskoka Road 169 underpass structure; and
- Swamp crossing between approximate Hwy 11 Northbound Lane (NBL) centreline Stations 11+510 and 11+940 and Hwy 11 Southbound Lane (SBL) centreline Stations 11+550 to 11+970.

This report addresses the new structure proposed as part of the Gull Lake Narrows Bridge SBL deck rehabilitation and widening on Highway 11. A foundation investigation has been carried out to assess the subsurface conditions at this site. The foundation investigations for the related PinedaleRoad / Hewitt Street underpass structure, swamp crossing, and Highway 11 / 169 underpass structure for the project are provided in separate reports.

The terms of reference for the scope of work are outlined in Golder's proposal P41-1349 dated May 2004 that formed part of the Consultant's Agreement (P.O. Number 5005-A-000363) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated August 2004. The general arrangement drawing for the proposed new Hwy 11 SBL widening structure over Gull Lake was provided to Golder by MRC in March, 2006.

The purpose of this investigation is to establish the subsurface conditions at the proposed structure site by borehole drilling, rock coring, dynamic cone penetration tests (DCPT), in-situ testing and laboratory testing on selected samples. The boreholes and DCPT's for the current investigation were located in the field by a member of Golder's staff based on the information and survey layout provided by MRC. The general location of the investigated area is shown in the Key Plan on Drawing 1.

The investigation was supplemented with information from the following previous reports, drawings, and/or investigations:

- Report titled "Shoreline Bedrock Profiles, Proposed Bridge – Gull Lake, Highway No. 11 – District No. 11, W.P. 246-60", William Trow Associates Limited, dated September 1967.

- Report titled "Foundation Investigation, Proposed Crossing – Gull Lake, Highway No. 11 (400), District No. 11 (Huntsville), W.P. 246-60", William Trow Associates Limited, dated April 1967.

- Report titled "Additional Boreholes carried out by Foundation Section at Gull Lake and Hwy. (400) – 11 Line 'D' for Centre Pier Locations of the Northbound and Southbound Lane Structures", Department of Highways Ontario (DHO), dated February 1968.

- Gull Lake Bridge SBL and NBL Design Drawing Nos. D6107-1 to D6107-17, Department of Highways Ontario, dated March and May, 1968.

2.0 SITE DESCRIPTION

The site is located about 250 metres north of the existing at-grade intersection of Pinedale Road / Hewitt Street and Highway 11 in Gravenhurst, Ontario. The northbound and southbound lanes of Highway 11 presently extend over Gull Lake as two separate structures, located about 18 m apart. Both structures are two-span with a centre pier located within Gull Lake. Gull Lake narrows to about 75 m from shoreline to shoreline in this area. The existing SBL structure extends from about Station 12+878 to 12+976 along the Hwy 11 SBL centreline chainage.

In general, the site consists of flat terrain consisting of the existing Hwy 11 roadway and grassy centre median. Bedrock outcrops are exposed on both sides of Gull Lake; however, the majority of the south abutment location is covered with rock fill. Bedrock cuts were evident on both the east and west side of the existing Highway 11 in the vicinity of the north and south abutment locations, indicating a significant amount of rock blasting had been undertaken as part of the original construction of the existing bridges. Rock cuts near the abutment locations rise to elevations varying between 259 m to 267 m at the south and north abutment locations, respectively. Beyond the existing bridge and highway location, the site consists of rolling terrain with numerous bedrock cuts/outcrops, densely treed areas, and low-lying swamps. The existing ground surface in the area of the SBL bridge ranges from approximately Elevation 245 m (lakebed) at the pier location to Elevation 257 m at the abutment locations. The existing Highway 11 top of pavement grade is at about Elevation 258 m to 259 m.

The proposed bridge abutments are located within the centre grassy median and the proposed pier is located within Gull Lake, in about 2 m to 3 m of water. At the south abutment location, rock fill is present within the majority of the abutment footprint which is located directly on the crest of the existing foreslope, which slopes steeply (approximately 1.5H:1V) towards Gull Lake. At the north abutment location, exposed bedrock outcrops are evident along the southern face of the foreslope which also slopes steeply (approximately 0.5H:1V in some areas) towards Gull Lake.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the Hwy 11 SBL widening (and consequent foundation widening) investigation was carried out between November 22, 2004 and February 22, 2005 during which time a total of five sampled boreholes (BH04-15 to BH04-19) and seven DCPT's (DCPT05-1 to DCPT05-7) were put down at the site. Two boreholes were drilled at each of the proposed north and south abutment locations. One borehole and seven DCPT's were drilled within the limits of the proposed centre pier foundation. Bedrock coring was carried out for a minimum length of 3 m in all of the boreholes.

The field investigation was carried out using a track-mounted CME 55 drill rig, skid-mounted D-25 drill rig, and skid-mounted tripod (for DCPT's only) supplied and operated by Walker Drilling Ltd. of Barrie, Ontario. The boreholes put down with the drill rigs were advanced using either 108 mm outer diameter (O.D.) solid stem augers or 75 mm O.D. 'N' casing. Soil samples were obtained, where possible, continuously or at intervals of about 0.75 m to 1.5 m depth, using a 50 mm O.D. split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes were all advanced to auger and/or sampler refusal on bedrock which occurred at depths ranging from 0.7 m to 3.2 m below the existing ground surface (not including rock coring) at the abutment locations and to a depth of about 15.9 m below ice surface at the proposed pier location. At all borehole locations, the drilling was further advanced into the bedrock by coring about 3.0 m to 4.5 m. The DCPTs were all advanced to cone refusal, which ranged from a depth of 5.2 m to 19.3 m below ice surface. It should be noted that three of the DCPTs were terminated on inferred bedrock, whereas four of the DCPTs were terminated on inferred cobbles/boulders (i.e. obstructions) within the probable fills on the existing lakebed. The groundwater level in the open boreholes was observed throughout the drilling operations.

The field work was supervised throughout by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and appropriate laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing such as water content, Atterberg Limits tests and grain size distribution were carried out on samples of the overburden soils. Strength testing such as point load index were carried out on specimens from the rock core.

On completion of the field work, all investigated borehole/DCPT locations were surveyed using the NAD83 MTM co-ordinate system and the geodetic datum for elevation. The surveying of the ground surface elevations of the as-drilled boreholes/DCPT's was carried out by members of our engineering staff, referenced to benchmark geodetic elevations provided by MRC. The northing and easting coordinates of the borehole/DCPT locations were calculated based on measurements from adjacent survey control points provided by J.D. Barnes Ltd. The borehole and DCPT locations are summarized in the following table and are shown on Drawing 1.

Borehole Number (BH)	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
04-15	South abutment	4974592.6	316849.5	257.0
04-16	South abutment	4974595.5	316846.8	257.4
04-17	North abutment	4974669.2	316919.4	257.0
04-18	North abutment	4974666.6	316922.5	256.9
04-19	Center Pier	4974629.4	316887.6	247.3

DCPT Number	DCPT Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
05-1	Centre Pier	4974634.7	316884.0	247.3
05-2	Centre Pier	4974635.9	316891.1	247.3
05-3	Centre Pier	4974630.2	316890.5	247.3
05-4	Centre Pier	4974625.7	316890.3	247.3
05-5	Centre Pier	4974630.2	316882.1	247.3
05-6	Centre Pier	4974629.4	316884.9	247.3
05-7	Centre Pier	4974631.2	316894.0	247.3

3.2 Bedrock Mapping

The exposed bedrock condition at the proposed Hwy 11 SBL widening north abutment location was assessed based on visual observations and detailed geotechnical mapping of the rock outcrop along Gull Lake in the immediate area of the proposed bridge. In addition, the geotechnical logging of the rock core from the boreholes was reviewed by a rock mechanics engineer.

The detailed geotechnical field mapping of the exposed rock conditions was carried out by one of Golder's rock mechanics engineers on November 19, 2004. In general, the orientation (dip/dip direction with respect to magnetic north) of the major discontinuities, including representative joint sets, was measured (refer to Figure B1). The nature of the various discontinuities was also noted including the persistence, shape, roughness and infilling as well as any groundwater seepage.

The results from the visual inspection and the detailed geotechnical mapping were used to assess the foundation conditions at the abutment.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geology

From published geologic information, the site is mainly located in the physiographic region known as the Number 11 Strip and portions of Highway 11 are in contact with the Georgian Bay Fringe region. The Number 11 Strip is a narrow belt that extends from Gravenhurst to North Bay and is categorized by deposits of sand, silt and clay between outcrops. The Georgian Bay Fringe is a broad belt characterized by shallow soil and bare bedrock knobs and ridges (The Physiography of Southern Ontario; Third Edition). Quaternary deposits of lacustrine and fluvial origin together with more recent swamp sediments have been accumulated between the bedrock ridges and, consequently, the overburden thickness and bedrock surface can be variable. The bedrock in the area is typically highly deformed gneiss of the Moon River Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province (Geology of Ontario; OGS Special Volume 4). Deposition of Paleozoic strata and later erosion during glaciation left behind these Precambrian rocks covered only in a few places by the flat-lying Palaeozoic bedrock strata.

4.2 Subsurface Conditions and General Overview

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and in Appendix A following the text of this report. The Record of Borehole sheets and laboratory tests from the 1968 and 1967 investigations are included in Appendix C.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). 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. The inferred soil stratigraphy based on the results of the boreholes and DCPT's at the proposed bridge location are shown on Drawings 1 and 2.

In general, the subsoils at the proposed bridge abutments generally consist of sand fill material containing trace to some amounts of topsoil and/or organics or rockfill with sand, containing cobbles and boulders. The fill is underlain by bedrock which was typically encountered between Elevations 253.8 m and 256.7 m. The total fill thickness ranges from 0.7 m to 3.2 m below ground surface at the south abutment and north abutment, respectively. Rockfill was present at depth at the north abutment borehole locations; whereas rockfill was visually evident at the ground surface within areas of the south abutment location. The subsoils encountered at the pier location generally consisted of sand to sand and gravel fill. The fill contained cobbles and boulders below about Elevation 242 m. The fill was underlain by a deposit of compact sand, underlain by stiff clayey silt containing sand seams. Beneath the cohesive deposit, thin layers of sandy silt to sand were encountered which were underlain by bedrock. A bedrock surface contour plan has been developed based on available borehole data and is shown on Figure 1.

The subsurface information described in the 1968 and 1967 investigations are in general agreement with the subsoils encountered during the current investigation at the pier location. However, it should be noted that the sand to sand and gravel (probable fill) encountered on the lakebed during the current investigation was probably granular material placed after the previous investigations were performed as part of the construction for the existing pier (as shown in the original design drawings from 1968). In addition, the bedrock contours at the north and south abutment locations, as shown in the previous investigations, have been altered considerably as a result of rock excavation and fill placement during past construction activities for the existing bridge. This is evident from the rock cuts and surficial fill evident at both the north and south abutment locations.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Ice and Water

Gull Lake was frozen at the time of the current investigation and ice was encountered in Borehole BH04-19 and at all DCPT locations (DCPT05-1 to DCPT05-7). The ice surface was encountered at about Elevation 247.3 m and the thickness ranged from 0.4 m to 0.5 m. The water was about 1.7 m to 3.4 m deep in this area (i.e. from top of ice to top of lakebed).

4.2.2 Sand / Sand and Gravel (Fill / Probable Fill)

A surficial sand fill deposit was encountered in all four boreholes put down during the current investigation at the abutment locations (BH04-15 to BH04-18, inclusive). The fill consisted of brown sand, trace to some gravel-sized granitic fragments, trace to some silt, and trace to some organics. The organics were typically within the top 0.3 m of the fill; however, organic pockets were encountered to a depth of up to 0.6 m. The surface of the sand fill ranged from Elevation 256.9 m to Elevation 257.4 m and the sand fill was 0.7 m to 1.5 m thick. Standard Penetration Testing (SPT) measured ‘N’ values in the sand fill typically ranged between 3 and 14 blows per 0.3 m of penetration, indicating a very loose to compact state of packing. Higher ‘N’ values were measured near the bottom of the sand deposit which may be influenced by the underlying rock fill or bedrock surface. The water content measured on two select samples of the sand fill were 4 and 5 percent. A grain size distribution curve on a select sample of the sand fill is shown on Figure A1.

At the pier location, a deposit of sand to sand and gravel (probable fill) was encountered at the lakebed in Borehole BH04-19 at a depth of 2.4 m below the ice surface (i.e. at Elevation 244.9 m). The sand to sand and gravel soil contained cobbles and boulders below about Elevation 242 m. Based on the lakebed profile from the previous investigations original design drawings (Drawing No. D-6107-1 titled “General Layout”, dated May 1968), it is likely that this sand to sand and gravel is granular fill material placed during the construction of the existing piers. The thickness of the probable fill materials encountered extended to a depth of about 8.4 m below ice surface (i.e. to Elevation 239.0 m). DCPT05-1, and DCPT05-4 to DCPT05-6 were terminated upon effective refusal of the cone (i.e. greater than 100 blows per 0.3 m of penetration) within the probable fill deposit between depths of 5.2 m and 6.8 m below the ice surface (Elevation 240.5 m to Elevation 242.1 m). Above a depth of about 5.3 m or Elevation 242 m (i.e. above the cobbles/boulders), Standard Penetration Test (SPT) “N” values measured within the probable fill typically ranged between 4 and 10 blows per 0.3 m of penetration, indicating a very loose to compact state of packing. Below about Elevation 242 m, the probable fill deposit is typically compact to very dense based on one SPT “N” value of 21 blows per 0.3 m of penetration and the fact that coring equipment was required to advance through the cobbles/boulders. The natural water contents obtained from three samples of the sand to sand and gravel (probable fill) ranged from 18 to 25 percent. One grain size distribution curve for a selected sample of the sand and gravel (probable fill) is shown on Figure A2.

4.2.3 Rock Fill

Rock fill material with sand was encountered at the north abutment location (i.e. Boreholes BH04-17 and BH04-18) underlying the sand fill. The rock fill contained cobble to boulder sized rock fragments. The solid stem augers achieved effective refusal to further penetration within the rock fill; as a result, the rock fill was cored using bedrock coring equipment. The rock fill was encountered at a depth of 1.5 m (i.e. at Elevation 255.5 m) and 0.8 m (Elevation 256.1 m) below ground surface and extended to depths of 3.2 m (i.e. at Elevation 253.8 m) and 2.3 m (Elevation 254.6 m) resulting in a thickness of 1.7 m and 1.5 m at Boreholes BH04-17 and BH04-18, respectively. Rock fill was not encountered within the boreholes advanced near the south abutment location (i.e. BH04-15 and BH04-16); however, rock fill was visually evident at the ground surface within parts of the proposed south abutment foundation footprint (i.e. at the crest of the foreslope) and covering most of the north facing foreslope.

4.2.4 Sand

A native sand layer containing some silt, trace gravel and clay was encountered below the probable sand to sand and gravel fill deposit in Borehole BH04-19 (located at the pier location) during the current investigation. The top of the sand layer was encountered at a depth of 8.4 m below the ice surface (Elevation 239.0 m) and the sand layer was found to be about 4.9 m thick. SPT “N” values ranged between 12 and 25 blows per 0.3 m of penetration indicating a compact relative density. Natural water contents obtained from two samples of the native sand material measured 16 and 19 percent. One grain size distribution curve on a selected sample of the sand layer is shown on Figure A3.

Boreholes 67-4, 68-4, and 68-5 from previous investigations generally agree with the results of BH04-19. A sand to silty sand layer was encountered at the lakebed surface at the time of the 1967 and 1968 investigations (i.e. prior to the inferred placement of the probable fill material). The top of the native sand to silty sand layer was encountered at a depth ranging from 7.3 m to 8.4 m below the ice surface corresponding to elevations ranging from El. 239.1 m to El. 240.2. The thickness of the sand to silty sand layer ranged from 4.8 m to 8.5 m. SPT ‘N’ values measured in the sand layer ranged between 10 and 16, indicating a compact relative density, consistent with the results of the current investigation.

4.2.5 Clayey Silt

Underlying the native sand deposit in Borehole BH04-19, a layer of clayey silt containing sand seams was encountered. The top of the clayey silt deposit was encountered at a depth of 13.3 m (Elevation 234.1 m) and the deposit was 2 m thick. One SPT ‘N’ value carried out within the cohesive deposit measured 15 blows per 0.3 m of penetration. Field vane testing carried out within the clayey silt layer measured undrained shear strengths of 65 kPa and 61 kPa, indicating a stiff consistency. The results of the field vane testing gave sensitivity values of 3.8 and 3.2, indicating the clayey silt has a medium sensitivity. The SPT “N” value and field vane tests performed in the clayey silt may have been influenced by the sand seams. A natural water content measured on one sample of clayey silt was 35 percent. Atterberg limit testing carried out a sample of the clayey silt measured a liquid limit of 28 percent and a plastic limit of 15 percent, corresponding to a plasticity index of 13 percent and indicating a clayey silt of low plasticity. The results of the Atterberg Limits test is illustrated on the plasticity chart on Figure A4 in Appendix A.

Boreholes 67-4, 68-4, and 68-5 from previous investigations generally agree with the results of BH04-19, encountering a clayey silt with sand layer below the sand to silty sand layer. The top of the clayey silt layer was encountered at depths ranging from 13.1 m to 15.8 m below ice surface, corresponding to elevations ranging from El. 234.3 m to El. 231.7 m. The thickness of the clayey silt layer ranged from 0.3 m to 3.7 m. One SPT ‘N’ value carried out within the clayey silt measured 2 blows per 0.3 m of penetration. A field vane test carried out within the clayey silt layer measured an undrained shear strength of 30 kPa, indicating a firm consistency. The field vane test gave a sensitivity value of 2.9, indicating a medium sensitivity. Natural water contents measured on three samples of the clayey silt ranged from 22 percent to 30 percent. The natural unit weight measured from a Shelby tube sample of the clayey silt gave a value of 19.8 kN/m^3.

4.2.6 Sandy Silt to Sand

Thin layers of sandy silt to sand were encountered in Borehole BH04-19 below the clayey silt layer and directly above the underlying bedrock. The top of the sandy silt to sand layer was encountered at a depth of 15.2 m (Elevation of 232.1 m) and the layer was 0.7 m thick. A single natural water content measured on the sandy silt to sand layer was 26 percent. One SPT ‘N’ value of 100 blows per 0.1 m of penetration was recorded within the sandy silt to sand, indicating the soil is very dense. This high SPT ‘N’ value was likely influenced by the underlying bedrock surface.

Borehole 67-4 from the previous investigation similarly encountered a sandy silt and sand layer below the clayey silt layer and above the bedrock. The sandy silt and sand layer is described as containing numerous cobbles and small boulders below about Elevation 229.4 m. The top of the sandy silt and sand layer was encountered at a depth of 16.8 m (Elevation 230.6 m) and the deposit was about 2.4 m thick. Two natural water contents measured on samples of the sandy silt and sand layer were 16 and 22 percent. The natural unit weight measured from a Shelby tube sample of the sandy silt and sand layer gave a value of 20 kN/m^3.

4.2.7 Bedrock

Bedrock was encountered and cored in all five boreholes during the current investigation. At the south abutment location, the top of the bedrock surface was encountered at a depth of 0.9 m (Elevation 256.0 m) and 0.7 m (Elevation 256.7m) in Boreholes 04-15 and 04-16, respectively. At the north abutment location, the top of the bedrock surface was encountered at a depth of 3.2 m (Elevation 253.8 m) and 2.3 m (254.6 m) in Boreholes 04-17 and 04-18, respectively. It should be recognized, however, that the bedrock surface elevation at the abutment foundation footprints could vary considerably beyond the borehole locations depending on the rock excavation techniques which were adopted during the previous highway construction. Typically, the upper 0.3 m of the bedrock at the abutment locations contained broken rock zones, with up to 0.6 m of broken rock encountered in BH04-18 located at the north abutment location. As such, it is thought that this disturbed upper bedrock is related to the blasting/rock shattering which was probably carried out during construction of the highway to achieve design grades.

At the centre pier location (BH04-19), the top of the bedrock surface was encountered at a depth of 15.9 m (Elevation 231.4) and confirmed by coring 3 m into the rock. DCPTs 05-2, 05-3, and 05-7 were terminated on inferred bedrock (i.e. cone refusal) at depths of 19.1 m (Elevation 228.3), 17.5 m (Elevation 229.8 m), and 19.3 m (Elevation 228.0 m), respectively.

Boreholes 67-4, 68-4, 68-5 and DCPT 67-12 from the previous investigations are described as encountering bedrock at depths ranging from 15.9 m to 19.2 m, corresponding to elevations ranging from El. 228.2 m to El. 231.6 m. It should be noted that the bedrock surface profile is highly variable in the vicinity of the proposed pier foundation footprint. The borehole/DCPT locations and depths to bedrock described in the previous investigations were combined with the results of the current investigation to produce an estimated bedrock surface contour map as shown on Figure 1.

The bedrock encountered and cored in the boreholes put down during the current and previous investigations (i.e. Boreholes 04-15, 04-16, 04-17, 04-18, 04-19, 67-4, and 67-5) is typically described as fresh to weathered, foliated blackish grey and pink, fine to medium grained, medium strong to very strong granite gneiss and/or biotite gneiss. The granite gneiss bedrock samples typically contained distinct foliation planes and medium to coarse grained quartz and feldspar veins; whereas the bitotite gneiss samples typically contained biotite bands/clusters and thinly banded quartz. The Rock Quality Designation (RQD) measured on the core samples typically ranged from about 72 to 97 percent, indicating a rock mass of fair to excellent quality. The Total Core Recovery was between about 84 percent and 100 percent. However, in Boreholes 04-15, 04-16, 04-17, and 04-18 (advanced during the current investigation at the south and north abutment locations), the RQD measured on core samples within the upper 0.2 m to 0.6 m typically ranged from about 36 to 40 percent, indicating a rock mass of poor quality.

Point load strength tests were performed on samples of the rock core from the current investigation. Axial and diametral point load strength index values are shown on the Record of Drillhole Sheets and on Table 1 following the text of this report. The point load index (Is_{50}) results from the laboratory tests on the bedrock range from approximately 1.8 MPa to 7.7 MPa with an average of about 5.6 MPa for diametral tests (i.e. testing carried out perpendicular to the core axis) and range from approximately 1.6 MPa to 6.5 MPa with an average of about 5.0 MPa for axial tests (i.e. testing carried out parallel to the core axis). The lower point load index values were typically noted within the biotite gneiss bedrock containing bands of medium to coarse biotite. It should be noted that within Boreholes 04-15, 04-16, 04-17, and 04-18, the zone containing broken rock pieces (i.e. the upper 0.6 m of rock core) did not have sufficiently sized samples for accurate point load testing and as a result no point load tests were performed in this region. As such, the strength results from the point load tests performed on the intact portions of the bedrock tend to be somewhat biased toward the high end of the rock mass strength range.

A summary of the average point load index values on the rock core from the five boreholes where coring was carried out is shown in the following table.

Borehole (Drillhole) No.	Average Axial Point Load Index Is_{50} (MPa)	Average Diametral Point Load Index Is_{50} (MPa)
04-15	6.2	6.9
04-16	-	6.7
04-17	1.6	3.9
04-18	5.4	4.4
04-19	5.5	6.1

Based on the laboratory point load testing results and approximate field measurement techniques (see Drillhole Sheets), the estimated intact strength of the granite gneiss bedrock typically varies from strong (50 MPa < UCS < 100 MPa) to very strong (100 MPa < UCS < 250 MPa), and the intact strength of the biotite gneiss bedrock typically varies from medium strong (25 MPa < UCS < 50 MPa) to strong (50 MPa < UCS < 100 MPa).

As discussed earlier, the existing rock cuts and outcrops in the area of the north abutment location were mapped by a rock mechanics engineer. Based on the geotechnical mapping of the existing rock cuts, there are 4 main joints sets including the foliation. The first joint set dips steeply to the SW (dip/dip direction 80º/205º), the second dips steeply to the NW or SE (dip/dip direction 87º/332º), the third (foliation) has a shallow dip to the NE (dip/dip direction 37º/071º) and the fourth set has a shallow dip to the SW (dip/dip direction 23º/244º). In general, most of the exposed rock outcrop at the north abutment location is comprised of a relatively clean rock face with widely spaced joints. One detached or partially detached block of rock was noted at the crest of the slope in the area of the abutment (refer to Figure B1). It appears that the detached or partially detached block extends back approximately 2 m from the crest of the rock cut.

4.2.8 Groundwater Conditions

The boreholes at the abutment locations (04-15, 04-16, 04-17, and 04-18) were dry upon completion of drilling; however, it should be noted that the proposed abutment footprints are located within the existing median storm water ditch centerline which drains into Gull Lake. Borehole 04-19 was advanced on top of the Gull Lake ice surface, which was at about Elevation 247.3 m in February, 2005. The Gull Lake ice surface during the previous investigations at the site was at about Elevation 247.4 m and 247.5 m in March 1967 and February 1968, respectively. Details of the water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report.

It should be expected that perched water conditions may exist within the existing Highway 11 stormwater drainage paths (near the abutments), on top of the bedrock surface. It should be noted that water levels are subject to seasonal fluctuations.

4.3 Closure

The field technician supervising the drilling program was Mr. Suresh Bainey. The rock mechanics engineer that performed the detailed bedrock mapping was Mr. Mark J. Telesnicki, P.Eng. This report was prepared by Ms. Shannon Palmer, EIT and Mr. Kevin J. Bentley, P.Eng., a geotechnical engineer, and reviewed by Ms. Anne S. Poschmann, P.Eng., and quality control review was provided by Mr. Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder.

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PART B

FOUNDATION DESIGN REPORT
DETAIL DESIGN
GULL LAKE NARROWS BRIDGE
SOUTHBOUND LANE STRUCTURE WIDENING
HIGHWAY 11 / MUSKOKA ROAD 169 INTERCHANGE
GRAVENHURST, ONTARIO

G.W.P. 314-00-00

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of the proposed widening and rehabilitation of Gull Lake Bridge – Southbound Lane Structure on Highway 11. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes and DCPTs advanced during the current and previous subsurface investigations.

The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

5.1 General

Currently, there are two separate bridges crossing Gull Lake which carry the Highway 11 Northbound Lanes and Southbound Lanes traffic, respectively. Both structures are two-span with a centre pier located within Gull Lake. The existing ground surface in the area of the proposed bridge foundations ranges from approximately Elevation 245 m on the lakebed at the pier location to Elevation 257 m at the proposed abutment locations.

It is understood that the existing Southbound Lane (SBL) structure is to be widened on the east side (i.e. within the median separating the two bridges) with consequential widening of the footings. The proposed bridge abutments are located within the centre grassy median and the proposed pier is located within Gull Lake, in about 2 m to 3 m of open water. The existing Highway 11 southbound lanes top of pavement is at about Elevation 259.0 m at the south abutment location and Elevation 258.0 m at the north abutment location. The proposed two span SBL widening structure is to have span lengths about 50 m long (same as existing), with the 9 m widening, the final SBL structure (existing and new SBL bridge) will be about 22 m wide. The proposed elevation of the SBL widening is to match the existing SBL pavement grade.

The following information on the existing Hwy 11 SBL bridge at this site is based on available drawings (Gull Lake Bridge SBL and NBL Design Drawing Nos. D6107-1 to D6107-17, Department of Highways Ontario, dated March and May, 1968):

- The existing Hwy 11 SBL structure south and north abutment spread footings are founded at about Elevations 253.1 m and 254.7 m, respectively. The abutment footings are indicated to be placed on “sound” bedrock.

- The existing Hwy 11 SBL centre pier is supported on steel H-piles (12-BP-74) fitted with "Oslo Points"; apparently the piles are driven into bedrock. The design load (assumed to be a working stress design load) of each pile is 750 kN (85 tons) and the piles are battered at a 1H:3.75V slope.

The overburden soils at the proposed abutment locations consist predominantly of sand fill containing trace to some organics and/or rock fill typically underlain by strong to very strong granite or biotite gneiss bedrock of fair to excellent quality; with the exception of the upper 0.2 m to 0.6 m of bedrock which contains broken rock of poor quality. It is considered that this upper zone (i.e. upper 0.2 m to 0.6 m at the abutment locations) is related to the blasting / rock shattering that was probably carried out during construction of the existing highway bridge. It is also possible that the broken rock could be due to frost penetration/action as some of the boreholes in this area were located in the existing median storm water drainage path for Highway 11.

The overburden soils at the pier location consist predominantly of sand to sand and gravel fill, containing cobbles/boulders, overlying native sand, clayey silt, and sandy silt over fresh, sloping bedrock.

5.2 Bridge Foundation Options

Various alternatives for the abutment and pier foundations are considered in the sections below and summaries of these alternatives are presented in Tables 2 and 3, respectively, following the text of this report. At the north and south abutment locations, spread footings founded on bedrock is considered to be the most feasible option from a geotechnical / foundation perspective.

At the pier location, given that the footing is located within Gull Lake in about 2 m to 3 m of water and the poor subsoil conditions encountered, spread footings are not considered feasible. Given the deep variable bedrock surface, the use of steel H-piles driven to bedrock is considered to be the most feasible option from a geotechnical / foundation perspective.

5.3 Spread Footings

The bridge abutments may be supported on spread footings placed on the properly prepared granite and/or biotite gneiss bedrock. The range in bedrock surface elevation as encountered in the boreholes and DCPTs at the abutment locations is summarized in the following table.

Foundation Element	*Borehole (BH)/ Dynamic Cone Penetration Test (DCPT) Numbers*	*Depth to Bedrock (below ground surface)*	*Bedrock Surface Elevation*
North Abutment	BH04-17 and BH04-18	3.2 m and 2.3 m	253.8 m and 254.6 m
South Abutment	BH04-15 and BH04-16 and information from previous investigations	0.7 m to 2 m	253.5 m to 256.7 m

Based on the current borehole results and exposed rock mapping, and borehole/topographic information from previous investigations, there is variability in the bedrock surface within the limits of each abutment foundation element. In addition, all loose or fractured rock encountered at the bedrock surface will need to be subexcavated and removed which may result in lower footing founding elevations than those indicated in the table above. As such, the footing founding elevation for the abutments may require a combination of overburden/bedrock excavation, mass concrete placement or both.

Based on the two boreholes put down during the current investigation at the north abutment location, there is potentially less than about 1 m variation in the bedrock surface elevations. It should be noted that the original bedrock contours in this area were much higher and significant rock cut has been undertaken during previous construction of Highway 11 and locally at the existing abutment foundation. Depending on the methods of rock excavation (i.e. blasting practices) and neatness of rock excavation, some areas within the proposed new abutment footprint may be highly variable in terms of elevation and rock soundness. In this case, the best option is probably Option No. 1, as outlined below, since this provides a founding level near the elevation of the adjacent existing abutment foundation and provides for more flexibility for variation in the bedrock surface.

Based on the two boreholes put down near the south abutment location, and correlating the results of the current investigation with the original bedrock contour mapping provided in previous investigations, the bedrock surface within the proposed south abutment footing footprint appears to slope downward from south to north, and from west to east. Based on the original Gull Lake SBL design Drawing No. D6107-2, titled “Southbound Lane – Gull Lake”, dated March 1967, the bedrock elevations within the proposed new south abutment footing footprint are at about Elevation 255.0 m, 254.5 m , 254.0 m and 253.5 m at the southwest, southeast, northwest, and northeast corners. In this case, the most suitable option is probably Option No. 2 or No. 3, as outlined below, since this provides a founding level near or at the elevation of the adjacent existing abutment foundation and helps make sure of removal of the upper broken rock on the steeply sloping bedrock.

For design of the abutment foundations, consideration could be given to three options for founding levels as described below. These options essentially vary the potential amount of bedrock excavation and/or mass concrete placement required. The proposed foundation elevations at the north abutment are based on the highest (Option No. 1) and lowest (Option No. 2) bedrock elevations encountered in the boreholes; the highest elevation encountered in the boreholes was raised by 0.1 m to match the existing design abutment founding level. The proposed foundation elevations at the south abutment are based on the highest (Option No. 1) and lowest (Option No. 2) bedrock elevations as shown on the bedrock contour map (Figure 1), which correspond well with the bedrock elevations encountered in the boreholes which were offset from the proposed foundation location.

Option No. 1 - The following foundation elevations may be assumed:

North Abutment: Elevation 254.7 m
South Abutment: Elevation 255.0 m

In this case, following the removal of the overburden, the bedrock surface would have to be cleaned and then mass concrete would be placed to raise the grade to the founding level. A Non-Standard Special Provision (NSSP) should be made in the Contact Documents for additional mass concrete placement to accommodate variations in the bedrock surface (an example is provided in Appendix D).

At the south abutment footprint, the sloping bedrock (steeper than 1.25H:1V) will require installation of dowels between the bedrock and concrete to increase sliding resistance (see Section 5.3.2). Also, at the south abutment we understand the existing SBL foundation is founded at Elevation 253.1 m; thus, the proposed founding elevation is estimated to be about 1.9 m higher than the existing footing. Assuming that the existing south abutment is founded on the bedrock, there will be a requirement for up to 1.9 m of mass concrete placement under the proposed footing unless a stepped footing is used.

The benefit of this general approach is that excavation into the strong to very strong bedrock is limited or avoided. In addition, at the north abutment location, the new footing founding elevation will match the existing footing founding elevation.

Option No. 2 - Alternatively, the following design founding levels may be assumed:

North Abutment: Elevation 253.8 m
South Abutment: Elevation 253.5 m

In this case, following the removal of the overburden, excavation of the higher portions of the bedrock will be required within the foundation footprints. Based on the borehole results, subexcavation of up to about 1.5 m of bedrock will be required in some foundation areas. It is noted that the bedrock is classified as medium strong to very strong (i.e. estimated unconfined compressive strengths in the range of about 40 MPa to 170 MPa) and the level of fracturing in the upper portions of the rock is variable. This will make excavation potentially difficult particularly in areas where only small depths and narrow zones of removal are needed, especially at the south abutment location. Bedrock excavation would likely have to be carried out using line drilling and pre-shearing techniques (see recommendations in Section 5.10). This method would provide better control over the configuration of the founding surface, and minimize blast damage to the rock.

It is noted that this design founding level for the south abutment is still higher than the existing footing level but the design founding level for the north abutment is lower than the existing north abutment founding level. Therefore, this option is not preferred unless stepping of the footing can be accommodated.

Option No. 3 - As a third option, an intermediate founding level may be assumed for design. In this case, a combination of bedrock subexcavation and mass concrete placement will be required. This option may be preferable at the south abutment location where the founding elevation can be lowered (compared to Option No. 2) in order to match the existing footing founding elevation.

The simplest and preferred option for the bridge north abutment footings, from a foundation perspective, is Option No. 1 or a variation on Option No. 1 with spread footings placed either directly on the properly prepared bedrock surface or placed on mass concrete constructed on the properly prepared bedrock surface which should minimize the bedrock excavation difficulties and allow for the new footing to be founded at the same elevation as the existing adjacent footing. At the south abutment, Option No. 3 is considered the preferred option (from a foundation perspective) as it allows for the new footings to be founded at the same elevation as the existing adjacent footings.

All bedrock excavation within and near the footing areas should be carried out using line drilling and pre-shearing techniques to minimize shattering and over-break and new abutment footings should be located no closer to the crest of the fore-slope than the existing abutment foundations. A pre-blast survey and vibration monitoring should be carried out at the existing bridge structures (i.e. specifically at each bridge abutment foundation/stem) prior to and during bedrock excavation/blasting, as outlined in Section 5.10. An NSSP should be included in the Contract Documents, an example is included in Appendix D.

In all areas where mass concreting is to be employed, it will be necessary to clean, scale and remove any loose debris to ensure a proper bond to the bedrock. In addition, a check on the sliding resistance between the mass concrete and the bedrock should be carried out (in accordance with the recommendations provided in Section 5.3.2).

5.3.1 Geotechnical Resistance

Spread footings placed on the surface of the properly prepared "intact" granite or biotite gneiss bedrock may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 10,000 kPa. For footings placed on a mass concrete pad, the factored geotechnical resistance at Ultimate Limit States (ULS) is as given above for bedrock assuming that the strength of the concrete used to form the pad is at least 25 MPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored geotechnical resistance at ULS, since the gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

All loose, shattered and/or fractured rock within the foundation footprint, and at and below (if disturbed during excavation practices) the design founding level should be removed and scaled prior to replacement with concrete and in accordance with OPSS 902 and Special Provision No. 902S01.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary.*

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base of the concrete footings and the granite and/or biotite gneiss bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. In the case of mass concrete placed on the bedrock surface, the design must check the sliding resistance between the base of the concrete footings and the top of the mass concrete, and between the base of the mass concrete and the bedrock. The coefficient of friction, tan δ, may be taken as 0.62 between the base of the concrete footings and mass concrete, and as 0.70 between the base of mass concrete/concrete footings and bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the intact rock mass is essentially as strong as or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the unfractured (intact) bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded.

A ULS design value of 400 kPa may be assumed for the grout-to-rock bond strength, based on applying a resistance factor of 0.4 (according to Table 6.6.2.1 of the *CHBDC*) to the ultimate bond strength of 1,000 kPa. The geotechnical resistance at Serviceability Limit State (SLS) for 25 mm of displacement will be greater than the factored resistance at ULS; as such, ULS conditions will govern for this installation. The upper 0.5 m of the bond length should be ignored in the calculation of required bond length since the rock near surface is typically weathered or disturbed. The actual bond strength for the rock – grout interface may vary from the typical design value given and should be verified in the field. Dowels should be checked to ensure that the rock mobilized around the anchor can support the design load (i.e. check against conical rock mass failure). Closely spaced dowels should be checked for group interaction. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Document to specify the installation, materials and testing of the dowels (an example is provided in Appendix D).

5.3.3 Frost Protection

For spread footings or mass concrete founded on the properly prepared intact granite/biotite gneiss bedrock at this site, frost susceptibility is not an issue.

5.4 Steel H-Pile Foundations

Steel H-piles are recommended for support of the centre pier foundation. At the pier foundation footprint, the bedrock surface slopes gently to the south of the pier and slopes steeply (up to 1H:1V) to the north side of the pier (see Figure 1). Based on the subsurface conditions, steel H-piles driven to refusal on the granite / biotite gneiss bedrock is recommended.

It should be noted that within Borehole BH04-19, advanced at the pier location, cobbles / boulders were encountered within the sand to sand and gravel (probable fill) between about Elevation 242 m and 239 m. In addition, four of the seven DCPTs (05-1, 05-4, 05-5, and 05-6) put down during the current investigation achieved cone refusal (i.e. greater than 100 blows / 0.3 m of penetration) within the sand to sand and gravel (probable fill) at about Elevation 242 m to 241 m, and may be indicative of potentially difficult driving conditions and/or the presence of gravel/cobbles/boulders. Borehole 68-4 drilled during a previous investigation also described "numerous" cobbles and boulders below a depth of 18 m (Elevation 229.4); bedrock was encountered at 19 m depth (Elevation 228.4). An NSSP alerting the Contractor of the presence of cobbles/boulders should be included in the Contract Specifications; an example is provided in Appendix D.

For design, a pile tip level at 232 m at the south side of the pier and a pile tip level of 228 m on the north side of the pier may be assumed for these piles. There should be provision made in the contract for dealing with varying pile lengths and piles should be fitted with appropriate rock points (i.e. Titus "Rock Injector Design", Oslo Points as per OPSD 3000.201, or equivalent) due to the presence of cobbles and boulders as well as the sloping bedrock. A NSSP should be included in the contract to address this issue and is included in Appendix D for reference. Pile installation and rock points should be in accordance with Special Provision SP903S01.

The water at the pier location is about 2 m to 3 m deep; thus, groundwater control measures in the form of a temporary sheetpile cofferdam, a tremied concrete seal, and dewatering will be required in order to complete construction of the pile cap in the dry.

5.4.1 Axial Geotechnical Resistance

For steel HP 310 x 110 piles driven to refusal on the granite and/or biotite gneiss bedrock, the factored axial resistance at Ultimate Limit States (ULS) of 1,600 kN may be assumed for design. The ULS value of 1,600 kN has been reduced to account for the potential for difficulties in dealing with the steeply sloping bedrock in some areas and potential for the piles sliding along the bedrock surface. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the granite / biotite gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

5.4.2 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. The maximum pile batter should be 1H:3.75V in order to match the existing pile configuration and reduce the potential for new piles sliding along the sloping bedrock surface. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory, where the coefficient of horizontal subgrade reaction, k_h (MPa/m) for pile width B (m), is based on the equations given below:

For cohesive soils:

$$k_h = \frac{67\tau_u}{B}$$ where B is the pile diameter (m) and τ_u is the undrained shear strength of the soil (MPa).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$ where n_h is the constant of horizontal subgrade reaction as given below; z is the depth (m); and B is the pile diameter (m).

The following table provides the recommended range for the value of τ_u and n_h to be used in the structural analysis. The range in values reflects the variability in the subsurface conditions. Design values are provided for the full stratigraphic sequence at the site, even though it is likely more than what is needed for the design of the H-piles.

Soil Unit	***n_h (MPa/m)***	***τ_u (MPa)***
Existing very loose to compact sand to sand and gravel (probable fill) at pier location (above El. 239 m)	2 to 5	-
Compact Sand (above El. 234 m)	4 to 6	-
Firm to Stiff Clayey Silt (above El. 230 m)		0.030 – 0.060
Compact to very dense sandy silt to sand (above bedrock and below clayey silt)	10 to 15	-

A maximum lateral resistance of 100 kN at ULS and 25 kN at SLS is recommended for vertical HP 310x110 piles driven to bedrock at the pier location. Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	***Reduction Factor***
8d	1.0
6d	0.7
4d	0.4
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

5.4.3 Frost Protection

The base of the proposed pile cap should match the base of the existing SBL centre pier pile cap elevation (approximately Elevation 245.7 m) and be designed to be below the anticipated maximum ice thickness to reduce uplift forces.

5.4.4 Existing Pier Foundations

As discussed previously, we understand that the existing Highway 11 SBL pier is supported on steel H-piles (12-BP-74 Imperial designation which corresponds to HP310x110 Metric designation) fitted with “Oslo Points”; apparently the piles are driven into bedrock. The original design drawings indicate a design load of 750 kN (85 tons) per pile was used and the piles are battered at a 1H:3.75V slope with embedment lengths estimated to range from 24 m (80 ft) on the south side to 34 m (112 ft) on the north side. There are no installation records and the as constructed pile tip elevations are not known.

The subsurface conditions at the existing Hwy 11 SBL pier are generally consistent with the results of the current investigation. The bedrock profile within the existing Hwy 11 SBL pier footprint is highly variable; similar to the conditions at the proposed widening structure. Referring to Figure 1, the bedrock slopes steeply in areas both north and south of the existing pier foundation. Also, cobbles / boulders were typically encountered above the bedrock surface in this area as noted from the previous investigations. Specifically, in Boreholes 67-4 and 67-6 (located within the approximate pile footprint), numerous cobbles and boulders were encountered within 1.2 m and 1.8 m of the bedrock surface.

For the existing H-piles at the Highway 11 SBL and NBL pier structure fitted with “Oslo Points - driven into bedrock” (as noted on design Drawing No. D-6107-4, Footing Layout, Gull Lake Bridge, prepared by DHO, dated May 1968), the factored axial resistance at Ultimate Limit States (ULS) will depend on whether the piles are definitely driven to bear on the bedrock or not. Provided that the piles (fitted with Oslo points) have been driven to practical refusal on the bedrock using a suitably sized hammer/pile driving rig, a factored axial resistance at ULS of 1,400 kN can be assumed. The ULS value has been slightly reduced to account for the assumed difficulties in dealing with the sharply sloping bedrock during construction and potential for the piles sliding along the bedrock surface. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the granite / biotite gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

Resistance to lateral loading for the existing piles, if required, can be calculated using the recommendations provided in Section 5.4.2 and using the anticipated embedment depth in the area of the existing pier foundation (see Figure 1).

5.5 Caissons / Drilled Piles

As an alternative to driven piles at the pier location, caissons/drilled piles socketted into the granite / biotite gneiss bedrock could be used for support of the bridge pier; the length could be varied to accommodate the variability of the bedrock surface. However, given the specialized equipment and procedures (and associated high costs) compared to the driven H-pile alternative, this option is not preferred. Consideration could be given to using caissons/drilled piles at the north abutment location; however, the practicality of using caissons depends on being able to achieve sufficient embedment length which depends on abutment stem lengths, pile cap thickness, and assessing the risk of encountering bedrock at sufficiently higher elevations than those encountered in the boreholes. Given the shallow depth and sloping bedrock at the south abutment location (bedrock less than 2 m below ground surface), caissons/drilled piles are not considered practical.

The following bedrock elevations may be assumed at the north bridge abutment and centre pier location, not including socket length into the bedrock. Refer to Figure 1 for a more detailed estimate of bedrock surface elevations.

Foundation Element	***Estimated Bedrock Elevation (not including socket length)***	***Depth to Bedrock Surface (below ground surface, not including socket length)***
North Abutment	253.5 m	2 m to 3 m
Centre Pier	232 m (North side) 228 m (South side)	15 m to 19 m

As discussed in Section 5.4, the presence of rockfill, cobbles and boulders will require appropriate drilling techniques in order to advance the caissons/drilled piles, and also the liners, through the overburden deposits.

The caissons/piles should be socketted into (rather than driven to) the bedrock to achieve a level founding surface at the base of the caisson/drilled pile and to minimize the potential for sliding along the inclined bedrock surface. The sloping bedrock will also present difficulties in the socketting as well as the drilling for the rock anchors since a seal will be required at the base of the caisson or drilled pile to prevent inflow of the surrounding sands and silts during cleaning, rock drilling and placement of concrete. As a result, small diameter (i.e. 324 mm O.D.) concrete-filled pipe piles installed using specialized down-hole hammer drilling techniques are preferred in lieu of larger diameter caissons which are less likely to achieve the required socket and/or watertight seal for rock anchors within the hard, steeply sloping bedrock without more specialized equipment. As a result, if higher capacities are needed (see Section 5.5.1), larger diameter pile/caissons may not be economical and other foundation alternatives should be investigated.

In general, the small diameter (324 mm O.D.), down-hole hammer, drilled pile system uses a four step process. The first step is to weld a non-salvageable ring (i.e. crown) to the end of a steel pipe pile that will be used to drill into the bedrock and allow rotation of the shoe without rotation of the steel pipe. The next step is to insert the pilot bit into the steel pipe pile, which locks into the crown by rotating clockwise. The next step involves drilling through the overburden and bedrock by rotating the lower part of the crown (called the driver) and the pilot bit while the upper part of the crown and the steel pipe casing do not rotate. The last step (after the steel pipe casing reaches the required bedrock socket depth) involves reversing the drill direction to unlock and retrieve the pilot bit, and leaving the steel pipe and non-salvageable crown in place. The steel pipe can than be filled with tremie concrete (if water seeps through the bedrock) and reinforcing steel added, if required.

The caisson/drilled pile excavations must be inspected by qualified geotechnical personnel to ensure that the founding stratum has been reached and is consistent with the design assumptions and that the base has been properly cleaned and is dry. In this regard, temporary liners will be required to permit downhole inspection.

5.5.1 Axial Geotechnical Resistance

The drilled piles or caissons will derive their axial resistance in part from end-bearing and in part from shaft friction. For this site, the majority of the resistance will be derived from base resistance. The factored axial geotechnical resistance at ULS that may be used for design are given in the table below:

Drilled Caisson / Pile Type	***Socket / Anchor Details***	***Axial Resistance***	
		Bedrock	
		ULS	**SLS**
300 mm Diameter Drilled Pile (tremie concrete filled, 13 m thick steel pipe)	Nominal socketing into bedrock; however, small diameter rock bolt installed about 1.5 m into rock	1,200 kN	n/a
300 mm Diameter Drilled Pile (tremie concrete filled, 13 mm thick steel pipe)	Socketed a minimum 0.6 m into bedrock (measured from low side of sloping bedrock/pile interface)	*2,000 kN	n/a
324 mm Diameter Drilled Pile (tremie concrete filled, 13 mm thick steel pipe)	Socketed a minimum 0.6m into bedrock (measured from low side of sloping bedrock/pile interface)	*2,400 kN	n/a

*values depend on structural capacity of the pile and may need to be adjusted depending on final configuration, pipe steel grade, concrete strength, and reinforcing steel, if applicable.

For drilled caissons/piles founded in the gneissic bedrock, the resistance required to achieve 25 mm of settlement is greater than that given for ULS and therefore SLS conditions do not apply.

For larger diameter drilled caissons/piles (i.e. greater than about 324 mm diameter), an installation method similar to the system described previously would be required to achieve adequate socketing and in order to achieve larger axial resistance capacities. However, for larger diameter piles the sharply sloping bedrock becomes more difficult to excavate and requires more specialized equipment and construction techniques. As a result, larger diameter drilled piles/caissons may be uneconomical. If large diameter drilled piles/caissons are being considered, we can review the proposed installation method and provide axial resistance values upon request. Due to the variability of construction methods available and dependence of axial resistance design values on the method of large diameter pile/caisson installation, we cannot provide reliable values at this time.

5.5.2 Resistance to Lateral Loads

The resistance to lateral loading for the drilled piles/caissons should be in accordance with Section 5.4.2 for the pier location. For the north abutment location, if drilled caissons/piles are considered, the following table can be used in conjunction with the equations provided in Section 5.4.2.

Soil Unit	***n_h (MPa/m)***
Existing sand (fill) at north abutment location (above El. 256 m)	3 to 5
Existing rockfill with sand at north abutment location (above El. 254 m and below El. 256 m)	7 to 10

5.5.3 Frost Protection

At the north abutment location, pile caps should be provided with a minimum of 1.7 m of soil cover for frost protection. For the centre pier location, refer to Section 5.4.3.

5.6 Earthquake Consideration

For seismic design purposes, the Site Coefficient, S, for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.

5.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.7 m behind the back of the wall stem (see Case I in Figure C6.9.1(l)(i) of the *Commentary to the CHBDC*) or 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 (see Case II in Figure C6.9.1(l)(ii) of the *Commentary to the CHBDC)*.

- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM):

	SSM (sand fill)	**SSM (rock fill)**
Soil / rock unit weight:	20 kN/m^3	19 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.33	0.22
At rest, K_o	0.50	0.35

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	**Granular 'B' Type II**
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

    - Rotation of approximately 0.002 about the base of a vertical wall;

    - Horizontal translation of 0.001 times the height of the wall; or

    - A combination of both.

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.7 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio (A) for Gravenhurst is 0.05. Based on experience, for the thin overburden soils at the site and embankment heights of up to 2 m, a 10 to 20 per cent amplification of the ground motion may occur, resulting in an increase in the ground surface acceleration from 0.05g to between 0.055g and 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.09$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2.3k_h$, $k_v = 0$, and $k_v = -2/3$.

- The following seismic active pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.32	0.26	0.26
Non-yielding wall	0.37	0.30	0.30

Note : These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta=\phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio for this site.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to 15 mm at this site.

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$P = K\,\gamma'\,d + (K_{AE} - K)\,\gamma'\,H$$

Where:
K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
K_{AE} is the seismic active earth pressure coefficient;
γ' is the effective unit weight of the soil (kN/m^3)
  - taken as soil unit weights given above for fill materials
  - taken as 19 kN/m^3 for the native materials

d is the depth below the top of the wall (m); and
H is the height of the wall above the toe (m).

5.8 Approach Embankment Design and Construction

Based on the information provided on the General Arrangement Drawing for the site, the proposed existing and final top of grade for Highway 11 at the structure location ranges from about Elevation 258.0 m to 258.7 m. The existing ground surface at the proposed south and north abutment widening locations are at about Elevation 256.9 m and Elevation 257.4 m, respectively. As a result, the embankments will generally be less than 2 m high beyond the abutment foundation and wing wall footprint at each approach. However, the approach embankments at the wing wall and abutment stem backfill locations (i.e. beneath the approach slab) may be up to 6 m and 3 m thick based on the proposed design founding elevations (El. 253.1 m and El. 254.7 m) at the south and north abutments, respectively.

Based on the borehole results, the subsurface soils at the proposed approach embankment locations consist of a thin layer of loose sand with trace to some organics/roots and rock fill underlain by bedrock at shallow depth. All topsoil and organic matter should be stripped from below the approach embankment areas prior to fill placement.

The results of stability and settlement analysis for the new approach embankments are presented in the following sections.

5.8.1 Stability and Liquefaction

Based on the low embankment heights (i.e. typically less than 2 m) and shallow depth to bedrock, global stability of the approach embankments is not considered to be a concern at this site provided the recommended side slopes discussed in Sections 5.8.5 are used. At all areas, all soils containing organics (encountered at or below ground surface during field investigation operations) need to be removed prior to construction of the new embankments.

As described previously, a partially detached block of rock was noted at the crest of the slope in the area of the north abutment (refer to Figure B1). The partially detached block extends back (i.e. north) approximately 2 m from the crest of the exposed bedrock and about 2.5 m below the existing ground surface at the crest of the rock outcrop. Considering the design founding elevation for the north abutment ranges from about El. 253.8 m to El. 254.7 m for the different options, the base of the partially detached block is considered to be at or slightly above the design founding level; thus, stability of the partially detached rock wedge is not considered to be a concern with respect to the support of the foundation.

Considering the boreholes advanced at the abutment locations were dry upon completion of drilling and the Gull Lake water surface is at least 6 m below the base of the approach embankments, liquefaction within the existing sand fill below the approach embankments is not considered to be a concern (provided adequate drainage behind the retaining wall / wing walls is provided according to Section 5.7).

5.8.1.1 Embankment Fill Types

Based on the existing subsoil conditions, either earth fill or rock fill embankment options may be considered. The different fill alternatives (i.e. earth fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils / bedrock), construction cost and time, and ease of construction / availability.

It should be noted that the use of similar adjacent fill materials should be ensured to prevent problems caused by the migration of fines between dissimilarly graded fill types as well as potential variation in thermal effects related to different materials.

5.8.1.1.1 Earth Fill

The main advantage of using earth fill (i.e. granular fill) is the ease of construction and the lack of post-construction settlements within the fill embankment itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than rock fill slopes. For this project, acceptable earth fill is considered to be suitable locally available and/or imported, granular material.

5.8.1.1.2 Rock Fill

The main advantage of using rock fill is the ability to achieve steeper embankment side slopes. This is useful in areas with limited right-of-ways. In addition, rock fill will likely be available from any rock cuts proposed / required within the project limits; thus providing an advantage in cost. The disadvantage of using rock fill for the construction of embankments is that some post-construction settlement of the embankment fill itself will occur within about the first and second year of construction.

5.8.2 Settlement

Provided that the surficial topsoil and any sand fill containing organics is removed prior to the new embankment fill placement, settlements of the new approach embankments, due to compression of the thin foundation soils, are expected to be small. For new embankment fills constructed with rock fill, the majority of the settlement of the approach embankments is expected due to compression of the rock fill itself.

It is anticipated that the proposed foundation founding elevations will match existing footing founding elevations. As a result, new fill placed directly behind the abutments (i.e. beneath the approach slabs and within the proposed/existing rock cut) will be up to about 6 m and 3 m high at the SBL south and north abutment locations, respectively. As previously discussed, embankment heights beyond the approach slab are expected to be less than about 2 m.

The following sections describe the estimated settlement of the foundation soils and the estimated settlements of the embankment fill due to the loading imposed by the new approach embankments.

5.8.2.1 Settlement of Existing Fills

The subsoils at the abutment locations were placed over twenty years ago and consist of up to 1.5 m of sand fill and 1.7 m of rock fill. The settlement of the existing sand and rock fill is expected to be less than 25 mm, assuming that all topsoil/organic material has been removed. These settlements are expected to occur rapidly (i.e. during or shortly after construction).

5.8.2.2 Settlement of Rock Fill

If rock fill is used for the construction of the new embankments, in addition to the settlement due to compression of the foundation soils described above, there will be settlement due to compression of the rock fill itself. Settlement of the rock fill depends on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in the Special Provision 206S03 (dated January 2004) the settlement of the newly placed rock fill is expected to be relatively small. In general, it is estimated that for the granitic gneiss rock fill likely to be used at this site, for the up to 6 m high approach embankments, the settlement of the rock fill will be about 1% of the new effective height of rock fill. Estimated maximum total settlements within the approach embankments (directly behind the abutment) are anticipated to be in the order of 60 mm and 35 mm at the south and north abutments, respectively. It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction.

5.8.2.3 Settlement of Earth (Granular) Fill

Where earth fill (granular) is used for the construction of the embankments, the settlement of the approved new embankment fill itself is expected to be less than 25 mm. The majority of settlement will occur during construction.

It is noted that these modest amounts of settlement are conditional on the topsoil and organic soils being stripped and removed from the area of the embankment footprint prior to fill placement.

5.8.2.4 Mitigation of Approach Embankment Settlement

Based on the design drawings and conversations with the designer, the approach slabs will be supported directly on the approach embankment fill and the slabs cannot tolerate more than about 50 mm of settlement relative to the top of the abutment walls. As a result, it is recommended that earth (i.e. granular) fill be used beneath the plan limits of the approach slab to limit settlement of the embankment fill to less than 25 mm as described in the previous section. The granular fill can be tapered beyond the approach slab footprint (in the direction away from the abutment) to allow for transition to rock fill (similar to OPSD 3501.000). Generally rock fill placed above earth (granular) fill is preferred to prevent loss of finer material. However, if granular earth fill is placed above rock fill, the surface of the rock fill should be compacted and chinked prior to placing the granular material on top (as per SP206S03, January 2004, Sect. 206.07.08). Within the approach slab footprint and any settlement sensitive areas, the granular earth fill placed above rock fill should consist of Granular 'A' or Granular 'B' Type II material (OPSS 1010). Granular 'B' Type II material is preferred as more loss of material through the voids is expected if Granular 'A' material is used.

Although the use of earth (granular) fill mitigates settlement issues related to the approach slab, it creates stability problems due to the fact that exposed earth fill side-slopes must be maintained no steeper than 2H:1V and given the steeply sloping foreslope within the existing median. In order to design steeper side-slopes while maintaining earth (granular) fill below the approach slab, a detail similar to that shown in Figure 2A could be incorporated into the design. Figure 2A and 2B shows typical sections at the abutment approach slab location which uses temporary earth (granular) fill side-slopes at 1.5H:1V, covered with rock fill having permanent side-slopes at 1.25H:1V. As a result, the rock fill allows for steeper side-slopes which may be required for encroachment reasons or to match the existing steeply sloping foreslope.

5.8.3 Subgrade Preparation and Embankment Construction

The existing rockfill subsoils on bedrock are considered to be an appropriate subbase for the proposed approach embankments; however, prior to the placement of any fill, all surface and near surface layers of topsoil/organic deposits, sand fill, and any softened or loosened soils should be stripped from the plan limits of the proposed works and the remaining subgrade soils should be proof-rolled, where possible.

The following sections provide details on the recommendations for subgrade preparation and embankment construction.

5.8.4 Removal of Organics

Based on the information from the borings obtained during the field investigation, sandy soils containing organics can be expected near the surface in some areas of the new approach embankments. These sandy soils containing significant organics were typically less than 0.3 m thick, but up to 0.6 m thick, and should be stripped from the plan limits of the approach areas prior to fill placement.

5.8.5 Embankment Fill Placement

If earth fill (granular) is to be used for construction of the new embankments, placement of all granular fill material should be carried out in accordance with SP 206S03 (January 2004), in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the Standard Proctor maximum dry density. The final lift prior to placement of the granular sub-base or base course should be placed and compacted to current MTO requirements for pavements. Inspection and field density testing should be carried out by qualified geotechnical personnel during all earth fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. Side slopes for earth fill embankments should be no steeper than 2H:1V.

If rock fill is used for the construction of the new embankments, placement of all rock fill material should be carried out in accordance with the requirements as outlined in the Special Provision SP 206S03 (January 2004). The rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging shall be minimized by blading, dozing and ‘chinking’ the rock to form a dense, compact mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

Generally rock fill placed above earth (granular) fill is preferred to prevent loss of finer material. However, if earth (granular) fill is placed above rock fill, the surface of the rock fill should be compacted and chinked prior to placing the granular material on top (as per SP206S03, January 2004, Sect. 206.07.08).

According to Northern Region Engineering Directive NRE 98-200, a minimum platform widening of 1 m each side of the embankment should be provided. Although the platform widening is likely not needed for foundation/settlement reasons, it may be required for future overlays.

Vegetation cover should be established on all soil slopes to protect embankment fill against surficial erosion. Alternatively, if rock fill is used, no vegetation cover is required.

5.9 Design and Construction Considerations

5.9.1 Excavation

Excavations for construction of spread footings on bedrock at the abutment locations will typically extend through about 0.7 m to 3.2 m of loose to very dense sand and rock fills to expose the bedrock surface.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The sand and rock fill is classified as Type 3 soil according to OHSA. Excavations at the abutments will extend through relatively dry soils with only minor seepage expected near the bedrock surface in some areas. Temporary excavations (i.e. those that are open only for a relatively short period) greater than 1.2 m deep through the fill materials may be made with side slopes no steeper than about 1H:1V.

However, for excavations along side the existing Highway 11 SBL, temporary shoring may be required for roadway protection due to limited space for open-cut excavation and foundation construction. Where required, the temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision No. 105S19 (dated March 2005). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 105S19.

At the abutment locations, it is noted that the bedrock is classified as medium strong to very strong (i.e. estimated unconfined compressive strengths in the range of about 40 MPa to 170 MPa) and the level of fracturing in the upper portions of the rock is variable. This will make rock excavation potentially difficult, particularly in areas where only small depths and narrow zones of removal are needed. Bedrock excavation in the vicinity of the proposed abutment structure foundations should be carried out using line drilling and pre-shearing techniques (as discussed in Section 5.10). This method would provide better control over the configuration of the founding surface, and this procedure would be the preferred approach where deeper excavation into the bedrock is required for footing construction. Depending on the option adopted, excavation of the bedrock may be required in close proximity to the existing footing and measures will have to be specified for drilling/blasting to ensure there is no adverse impact on the existing bridges.

At the pier location, it is likely that a sheetpile cofferdam and dewatering will be required in order to construct the pile cap in the dry. It should be noted that obstructions (probable cobbles and boulders) were encountered within the existing sand to sand and gravel (probable fill) at and below Elevation 242 m, which may impede the installation of the sheetpiles. Considering the lakebed is at about Elevation 245 m and the subsoils consist of sand to sand and gravel (probable fill), with the lake water level at about El. 247.3 m, a tremied concrete seal will be required at the base of the excavation in order to allow for dewatering within the sheetpile cofferdam and placement of concrete for the pile cap in the dry. Referring to the design drawings, assuming a water level at El. 247.3 m, underside of footing to be placed in the dry at El. 245.7 m, the minimum thickness of tremied concrete (assuming a Factor of Safety against uplift equal to 1.3) is 1.8 m.

The Contractor will be responsible for determining the actual length of the sheetpiles for internal stability of the cofferdam. It should be noted that it may be difficult to install sheetpiles below El. 242 m in some areas (southwest portion of the footprint) due to obstructions (inferred cobbles/boulders) which were encountered during our drilling investigation.

It is assumed that the piles will be driven prior to placement of the tremie plug, after excavation to the base of the plug and that the piles will therefore be driven “in the wet”.

The Contractor’s sheetpile designer should check the tremie plug thickness against their design. A note should be included on the contract drawing to this effect.

5.9.2 Groundwater and Surface Water Control

At the abutments, depending on the time of year, minimal groundwater inflow into the excavations is anticipated during construction. However, the proposed abutment foundation footprints are located within the existing Highway 11 median storm water ditch; thus, storm water should be diverted away from excavations at all times. It is anticipated that groundwater or surface water, if encountered, can be adequately controlled by diverting the existing stormwater drainage path to promote run-off away from or around the proposed construction areas and/or by pumping from properly filtered sumps.

At the centre pier location, Gull Lake is about 2 m to 3 m deep. In order to construct the pile cap in the dry, the water must be adequately lowered within a sheetpile cofferdam and tremie plug as described in the previous section. Once the sheetpile cofferdam and tremie plug have been adequately sealed against water infiltration, the use of properly filtered sump pumps can be used to pump out the remaining water and control minor seepage to allow construction of the pile cap in the dry.

In all cases, a dry and stable excavation will be required to permit placement of mass concrete and construction of footings/pile caps.

5.9.3 Obstructions

At the south abutment location, rock fill was visible from the crest of the north facing foreslope down to the Gull Lake shoreline. At the north abutment location, the surficial sandy fill soils were underlain by about 1.5 m to 1.7 m of rockfill containing cobble and boulder sized pieces. Obstructions, likely cobbles and boulders, were also present at the centre pier location between depths of about 5.2 m and 6.1 m below ice surface (Elevation 242 m to 239 m) and cobbles / boulders were present directly above the bedrock surface (about 1 m thick) in one borehole (67-4) advanced during a previous investigation.

Conventional excavation equipment should be suitable for the majority of excavation through the on-site soils; however, the presence of rockfill with cobble and boulder sized pieces may interfere with or slow the progress of stripping and excavation at some locations. The presence of such obstructions may also affect the installation of sheet pile walls for construction of the centre pier and/or temporary roadway protection measures, if required. Ultimately, provision will have to be made in the Contract Specifications to ensure that the Contractor is equipped to handle such obstructions; an example of an NSSP is included in Appendix D.

5.9.4 Rock Hazards at Existing Bedrock Outcrops / Rock Cuts

Currently, rock cuts and/or bedrock in the vicinity of the Highway 11 SBL bridge widening north and south abutments are generally covered by the existing sand and rock fill. There is an exposed bedrock outcrop along the south facing slope at the north abutment as shown in Figure B1. Referring to Figure B1, at least one detached or partially detached block of rock was noted at the crest of the slope near the north abutment. It appears that the detached or partially detached block extends back approximately 2 m from the crest of the bedrock outcrop / rock cut and about 2.5 m below the ground surface. As a result, the proposed minimum footing offset / setback distance from the existing crest of the bedrock outcrop at the north abutment location is 4 m. To protect against potential rock hazards adjacent to the bridge, the new abutment foundations should be no closer to the crest than the existing abutment foundations, and the design founding elevations presented in this report should be used. It should be noted that the detached block of rock is considered stable unless excessive vibrations are caused from blasting and/or other construction activities. Rock blasting is not anticipated at the north abutment location (i.e. mass concrete placement is the preferred founding option in order to match the existing footing elevation), thus, the detached block is considered to be stable during and after construction. If rock blasting is required at the north abutment location, then the existing bridge foundations are be monitored against vibrations during blasting according to the recommendations provided in the NSSP in Appendix D. Considering the detached block is located within the median (i.e. between the existing bridge foundations to be monitored), the maximum peak particle velocity values not to be exceeded at the bridge foundation locations will provide protective measures against instability of the detached block.

5.9.5 Proposed Permanent Rock Cut Slopes

For the rock cuts which are planned to be excavated to create the required foundation footprints, the newly excavated rock faces are expected to be relatively less weathered and in better condition than the existing faces provided good blasting practises are implemented. For permanent cut slopes through the bedrock, the overall slope to the cut face may be formed vertical to near vertical (i.e. 0.25H:1V). The use of carefully controlled drill and blast excavation techniques will be required to ensure a neat excavation line and minimize face instabilities and long-term maintenance problems resulting from blast damage to the rock mass as discussed in the following sections.

5.10 Blasting Recommendations for Rock Excavations

5.10.1 Excavation Considerations

For excavations into the bedrock, the overall slope to the cut face may be formed vertical or at a steep near vertical slope (i.e. 0.25H:1V). The use of controlled blasting techniques (such as pre-shearing or cushion blasting) are recommended, particularly along footing areas, in order to provide a neat excavation line and minimize face instabilities resulting from damage to the rock mass.

5.10.2 Special Provisions

Blasting

Good blasting practices will be critical to maintaining the excavation lines and preserving the integrity of the rock mass in the area of the structure foundations and proposed rock cuts. The use of controlled blasting techniques is recommended for all of the bedrock excavation. It is recommended that the Contractor retain a blast engineer and submit proposed blast plans to the Contract Administrator at least 3 weeks in advance of rock excavation. It is recommended that a separate NSSP for the control of all blasting operations be prepared (refer to SP 299F06). The NSSP (see example in Appendix D) should include, but not be limited to, the following:

- Outlining the requirements, procedure and extent of a pre-blast survey. This would include all structures within a radius of about 100 m of the blasting operations, as well as notification to all individuals working or living within 500 m.
- Submission of a blast proposal by the blasting contractor or their blast consultant detailing the blast methodology, including drill hole patterns, hole size and depths, size of blasts, explosive and initiation product details, as well as all blast control procedures. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signalling and site clearing procedures, as well as procedures to deal with debris clean-up. This submission would be required prior to the commencement of any blasting operations.
- The requirement for trial blasts for all proposed production and wall control blast procedures.
- The requirements for ground and air vibration monitoring during the blasting operations. This would include details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings.

- At all locations where structures are located adjacent to rock cuts, the Contract Administrator should retain an independent rock engineering specialist (rather than the QVE) to inspect any new rock cut faces and provisions made for rock bolting, if necessary.

We recommend limiting ground vibration levels to 50 mm/s for adjacent bridges, services and buildings (refer to Table 1 in OPSS 120). Continuous monitoring of all blasting operations would dictate when changes to the blast procedures become necessary to meet these limits and how close to the blasting approaches the adjacent structures.

It is recommended that the specification for the blasting require a minimum of 80 percent half barrels (drill hole traces) visible on the cut face after scaling.

5.11 Closure

This report was prepared by Ms. Shannon Palmer, EIT and Mr. Kevin Bentley, P.Eng, a geotechnical engineer, and reviewed by Ms. Anne S. Poschmann, P.Eng., a Principal and senior geotechnical engineer. Mr. Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder conducted a quality control review of the report.

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REFERENCES

Bowles, J.E. 1984. Physical and Geotechnical Properties of Soils, 2nd Edition. McGraw-Hill Book Company, New York.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-00. 2001. CSA Special Publication, S6.1-00. Canadian Standards Association.

Kulhawy, F.H. and Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design. EL-6800, Research Project 1493-6. Prepared for Electric Power Research Institute, Palo Alto, California.

Geology of Ontario. 1991. Ontario Geological Society, Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

Makdisi, F.I., and Seed, H.B. 1978. Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations. ASCE Journal of the Geotechnical Engineering Division, V. 104, GT7, pp 849-867.

United States Naval Facilities Engineering Command, 1971. Soil Mechanics. Design Manual DM-7.01, U.S. Government Printing Office, Washington, D.C.

United States Naval Facilities Engineering Command, 1986. Foundations and Earth Structures. Design Manual DM-7.02, U.S. Government Printing Office, Washington, D.C.

TABLES

TABLE 1

SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO.:04-1111-039B

TITLE: Gull Lake Narrows SBL Structure Widening, Gravenhurst

DATE: November 30, 2004

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core (2) Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm)** (MPa)	**Approx. (1) UCS** (MPa)
04-15	1	1.5	D	120.7	46.0		13217.7	12.78		6.048	5.824	134
	2	2.2	D	251.5	47.2		16161.9	15.63		7.003	6.826	157
	3	3.1	D	127.0	47.4		17520.2	16.94		7.551	7.369	169
	4a	3.5	D	292.1	47.4		17658.1	17.08		7.610	7.427	171
	4b	3.5	A	66.5	47.4	63.35	23001.7	22.24	5.542		6.165	142
	5	4.2	D	336.6	47.4		17340.9	16.77		7.465	7.288	168
04-16	1	1.1	D	195.6	47.2		17299.6	16.73		7.496	7.307	168
	2	1.6	D	292.1	47.3		13300.5	12.86		5.738	5.599	129
	3	1.9	D	238.0	47.2		15258.6	14.76		6.618	6.450	148
	4	2.7	D	254.0	47.2		18278.6	17.68		7.920	7.720	178
	5	3.3	D	226.8	47.3		15479.3	14.97		6.700	6.533	150

(1) Is_{50} x 23 (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

(2) Actual distance between point load cones at time of failure.

TABLE 1 (cont'd)

SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO.:04-1111-039

TITLE: Gull Lake Narrows SBL Structure Widening, Gravenhurst

DATE: November 30, 2004

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core (2) Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm)** (MPa)	**Approx.** (1) **UCS** (MPa)
04-17	1	3.9	D	203.2	47.5		10838.9	10.5		4.651	4.544	105
	2a	4.7	D	205.2	47.5		9301.4	9.0		3.991	3.899	90
	2b	4.7	A	98.1	47.5	77.02	8198.2	7.9	1.337		1.623	37
	3	4.8	D	98.9	47.5		5640.1	5.5		2.420	2.364	54
	4	6.4	D	381.0	47.5		11418.1	11.0		4.900	4.787	110
	5	7.2	D	381.0	47.4		9218.6	8.9		3.960	3.868	89
04-18	1	3.2	D	304.8	47.5		6398.6	6.2		2.746	2.682	62
	2a	3.9	D	234.6	47.4		13141.9	12.7		5.658	5.523	127
	2b	3.9	A	54.5	47.4	57.37	20781.5	20.1	6.107		6.497	149
	3	4.8	D	365.8	47.4		4302.5	4.2		1.852	1.808	42
	4a	4.8	D	101.6	47.4		15541.3	15.0		6.683	6.526	150
	4b	4.8	A	59.8	47.4	60.08	14838.0	14.3	3.975		4.318	99
	5	5.8	D	381	47.4		12500.6	12.1		5.382	5.254	121
	6	3.4	A	55.8	47.2	57.91	17478.8	16.9	5.041		5.385	124

(1) Is_{50} x 23 (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

(2) Actual distance between point load cones at time of failure

TABLE 1 (cont'd)

SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO.:04-1111-039

TITLE: Gull Lake Narrows SBL Structure Widening, Gravenhurst

DATE: April 5, 2005

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core (2) Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm)** (MPa)	**Approx. (1) UCS** (MPa)
04-19	1	16.0	D	150.9	47.0		12162.8	11.8		5.327	5.180	119
	2	16.4	A	69.9	47.2	64.82	22477.7	21.7	5.174		5.815	134
	3	16.7	D	100.8	47.2		15258.6	14.8		6.611	6.445	148
	4	16.7	A	65.8	47.2	62.91	19761.1	19.1	4.829		5.355	123
	5	17.8	D	87.4	47.5		13783.1	13.3		5.908	5.773	133
	6	18.2	A	68.1	47.2	63.99	20498.8	19.8	4.842		5.410	124
	7	18.3	D	103.4	47.2		16499.7	16.0		7.149	6.969	160

(1) Is_{50} x 23 (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

(2) Actual distance between point load cones at time of failure

TABLE 2
EVALUATION OF ABUTMENT FOUNDATION ALTERNATIVES
Gull Lake Bridge SBL Widening, Highway 11
G.W.P. 314-00-00

Footing Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Spread Footings on bedrock or mass concrete pad on bedrock	1	• Can minimize bedrock excavation depending on design footing level; • Relative ease of construction procedure.	• Variable bedrock surface will require bedrock and soil excavation with mass concrete placement to achieve level footing; • Bedrock will have to be blasted using controlled blasting techniques to minimize shattering and over-break; • Excavations up to about 3 m may be required at the both abutment locations. With limited construction space and adjacent to Hwy 11 wingwall, temporary shoring or roadway protection may be required.	• Lower relative costs than caisson installation; however, extent of bedrock excavation/blasting required may make costs more comparable.	• If bedrock is higher than anticipated, additional bedrock removal is required; • Variability in bedrock surface will impact mass concrete quantities and excavation depths.
Drilled Caissons / Tube Piles	2 / NF	• Can possibly be constructed at north abutment, where depth to competent bedrock is up to 3 m below existing ground surface. • Open cut excavation through existing sand and rock fill may be eliminated.	• Not practical at south abutment where bedrock surface is 0.7 m to 0.9 m below ground surface; • At north abutment location, drilling/augering through rockfill and into granite bedrock to achieve sufficient vertical and lateral capacity will be difficult; • Excavation to form trench or rock drilling will be required to achieve minimum required pile/caisson embedment lengths; • Access is limited at both abutment locations	• Higher costs for drilling/augering equipment.	• Equipment access may make this option not feasible; • Risk of encountering bedrock at higher elevations; difficult drilling through rockfill, sloping bedrock requires specified procedures to achieve sufficient socket lengths. • Depending on top of abutment wall elevation, this option may not be feasible due to shallow bedrock depth.
Spread Footings perched within embankment fill	NF	-	• Not practical at abutment locations where bedrock surface is so close to ground surface and final road grade; • Sloping bedrock (up to 1H:1V) at south abutment location makes design and constructability undesirable; • Partially detached block of rock at north abutment location will lead to stability / wedge failure concerns and mitigation.	-	-

Driven Piles	NF	-	• Typically less than 3 m distance between existing ground and bedrock surface; and proposed pile cap/road less than 2 m above existing ground surface (i.e. embedment depth less than 3 m in most areas); • Presence of rock fill requires subexcavation and replacement or pre-augering	-	-

NF: Indicates that the founding option is considered not feasible/practical.

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TABLE 3
EVALUATION OF PIER FOUNDATION ALTERNATIVES
Gull Lake Bridge SBL Widening, Highway 11
G.W.P. 314-00-00

Footing Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Steel H-Piles driven to bedrock	1	• Relative ease of construction; • Match existing Hwy 11 SBL and NBL pier foundation type which is over 35 years old; • Reduce disturbance to lakebed.	• Variable bedrock surface may lead to difficult driving conditions; however, the piles will need to be fitted with "Titus Rock Injector Design" rock points, Oslo points, or equivalent to account for sloping bedrock and presence of cobbles/boulders.	• Lower costs relative to drilled caisson/steel tube installation.	• Risk of sharply sloping bedrock and piles ability to "bite" into the bedrock surface; • Risk of piles being "hung up" in gravelly fill containing cobbles/boulders on lakebed and cobbles/boulders above bedrock surface.
Drilled Caissons / Tube Piles	2	• Can drill through potential obstructions within the sand to sand and gravel probable fill soils containing cobbles and boulders.	• Variable bedrock surface may lead to difficult drilling conditions and ability of caisson/pile to penetrate into bedrock may be compromised; • Increased disturbance to lakebed compared to driven piles.	• Higher costs for drilling/augering equipment (especially into bedrock) compared to driving piles.	• Risk of sharply sloping bedrock and caisson/pile ability to penetrate or "bite" into bedrock surface. As a result, risk of difficulties maintaining caisson/pile alignment.
Spread Footings	NF	-	• Not practical due to Gull Lake water depth of 2 m to 3 m at pier location and poor subsoil conditions.	-	-

NF: Indicates that the founding option is considered not feasible/practical.

RECORD OF BOREHOLE AND DYNAMIC CONE PENETRATION TEST SHEETS

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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

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) 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.

LIST OF SYMBOLS

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

I. General

π	3.1416
in x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

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
v	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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
l_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio = σ'_p/σ'_{vo}

(d) Shear Strength

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

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

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

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LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING 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 texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
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	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

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 varies 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 (W.R.T.) Core Axis

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

Description and Notes

An abbreviated 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

B - Bedding
FO - Foliation/Schistosity
CL - Cleavage
SH - Shear Plane/Zone
VN - Vein
F - Fault
CO - Contact
J - Joint
FR - Fracture
MF - Mechanical Fracture
II - Parallel To
⊥ - Perpendicular To

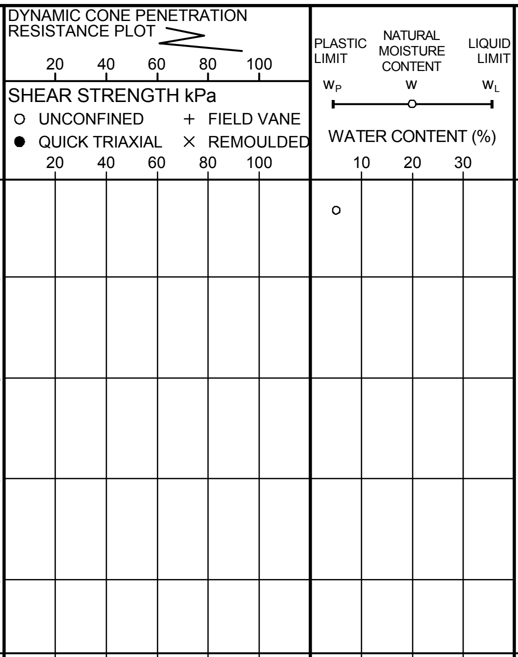
P - Polished
S - Slickensided
SM - Smooth
R - Ridged/Rough
ST - Stepped
PL - Planar
FL - Flexured
UE - Uneven
W - Wavy
C - Curved

RECORD OF BOREHOLE No BH04-15

1 OF 1 **METRIC**

PROJECT 04-1111-039			
W.P. 314-00-00	LOCATION N 4974592.6 ;E 316849.5		ORIGINATED BY SB
DIST	HWY 11	BOREHOLE TYPE 108 mm O.D. Solid Stem Power Auger	COMPILED BY JDR
DATUM Geodetic	DATE 22-Nov-04		CHECKED BY KB

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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.0	GROUND SURFACE										
0.0	Sand, some silt, trace gravel, trace to some organics/topsoil (FILL) Loose to very dense Brown Moist		1	SS	5				○		
256.0			2	SS	35/.08		256				
0.9	GRANITE GNEISS Fresh to weathered, very strong, fine to medium crystaline, foliated, dark grey/black and pink Bedrock cored from 0.9 m to 4.7 m For bedrock coring details refer to record of drillhole BH04-15						255 254 253				
252.3 4.7	End of Borehole Note: 1. Borehole dry during drilling operations.										



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

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH04-15

SHEET 2 OF 2

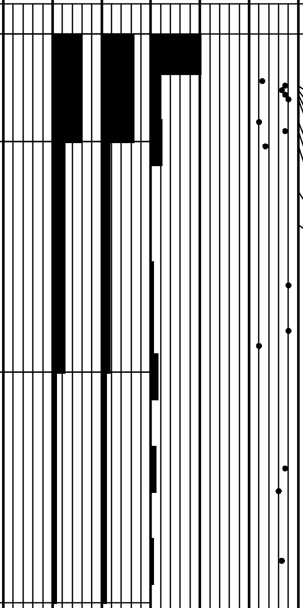
LOCATION: N 4974592.6 ;E 316849.5
INCLINATION: -90° AZIMUTH: ---

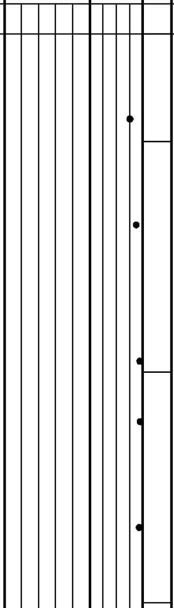
DRILLING DATE: 22-Nov-04
DRILL RIG: CME 55 Bombardier
DRILLING CONTRACTOR: Walker Drilling Ltd.

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	ELEV. / DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	256.03			
1	Rotary NQ	GRANITE GNEISS Fresh to weathered, very strong, fine to medium crystalline, foliated, dark grey/black and pink, with medium to coarse quartz and feldspar banding 1 mm infilling of weathered material along joints from 0.9 to 1.2 m Discontinuity at 3.7 m along veinlet	0.94	1	BR,, BR,Ro,IR FO, VN,IR,Ro FO,PL,Ro VN,IR,Ro	
2				2	FO,IR,Ro BR,IR,Ro FO,PL,Ro BR,IR,Ro BR,, BR,, JN, FO,PL,Ro JN, VN,IR,Ro VN,IR,Ro	
3				3	FO,PL,Ro FO,PL,Ro JN,FO,SM	Diametral 7.4 MPa Axial 6.2 MPa
4			252.27 4.70			
		End of Drillhole				
5						
6						
7						
8						
9						
10						





DEPTH SCALE
1 : 50

LOGGED: SB
CHECKED: KB

RECORD OF BOREHOLE No BH04-16

1 OF 1 **METRIC**

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4974595.5 ;E 316846.8
ORIGINATED BY SB
DIST
HWY 11
BOREHOLE TYPE 108 mm O.D. Solid Stem Power Auger
COMPILED BY JDR
DATUM Geodetic
DATE 22-Nov-04
CHECKED BY KB

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 (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE			kN/m³	GR SA SI CL
257.4	GROUND SURFACE										
0.0	Sand, some gravel, some silt, trace roots and organics (FILL) Compact Brown Moist		1	SS	14		257				
256.7 0.7	GRANITE GNEISS Fresh to slightly weathered, very strong, fine to medium crystaline, foliated, dark grey/black and pink Bedrock cored from 0.7 m to 3.7 m For bedrock coring details refer to record of drillhole BH04-16						256 255 254				
253.7 3.7	End of Borehole Note: 1. Borehole dry during drilling operations										

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

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

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH04-16

SHEET 2 OF 2

LOCATION: N 4974595.5 ;E 316846.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: 22-Nov-04

DRILL RIG: CME 55 Bombardier

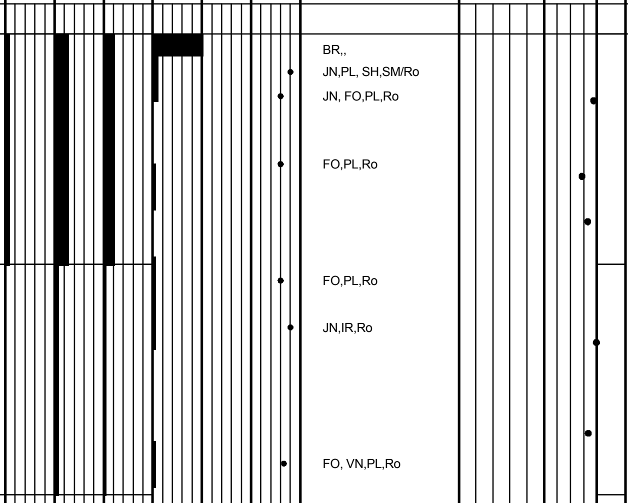
DRILLING CONTRACTOR: Walker Drilling Ltd.

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.	PENETRATION RATE (m/min)	COLOUR % RETURN FLUSH	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		256.73					
	Rotary NQ	GRANITE GNEISS Fresh to slightly weathered, very strong, fine to medium crystalline, foliated, dark grey/black and pink, with medium to coarse quartz and feldspar banding 1 mm infilling of weathered soil along joints to 0.8 m depth		0.66	1			BR,, JN,PL, SH,SM/Ro JN, FO,PL,Ro FO,PL,Ro	
					2			FO,PL,Ro JN,IR,Ro FO, VN,PL,Ro	
		End of Drillhole		253.68 3.71					



Depth scale: 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: KB

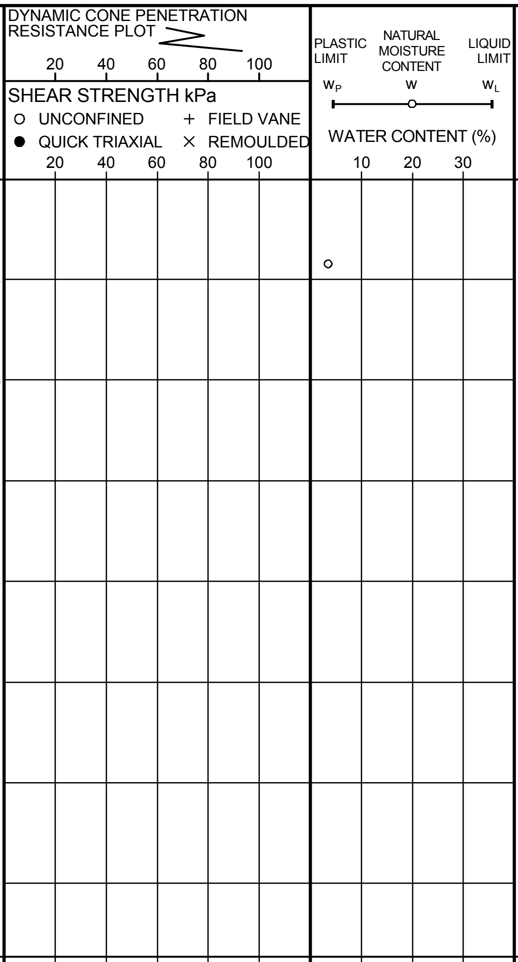
Golder Associates

RECORD OF BOREHOLE No BH04-17

1 OF 1 **METRIC**

PROJECT 04-1111-039
W.P. 314-00-00
DIST ______ HWY 11
DATUM Geodetic
LOCATION N 4974669.2 ;E 316919.4
BOREHOLE TYPE 108 mm O.D. Solid Stem Power Auger
DATE 22-Nov-04
ORIGINATED BY SB
COMPILED BY JDR
CHECKED BY KB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.0	GROUND SURFACE								
0.0	Sand, some silt, trace to some gravel, trace topsoil, roots and organics (FILL) Compact Brown Moist	▨	1 2	SS SS	13 10		256		
255.5 1.5	Rockfill with sand, contains cobbles and boulders (FILL) Cored from 1.5 m to 3.2 m	▨					255		
253.8 3.2	BIOTITE GNEISS Fresh to weathered, medium strong to very strong, fine to coarse crystaline, foliated, dark grey/black and white Bedrock cored from 3.2 m to 7.0 m						254 253 252 251		
250.0 7.0	GRANITE GNEISS Fresh, fine to medium crystaline, foliated, dark grey/black and pink						250		
249.3 7.7	Bedrock cored from 7.0 m to 7.7 m For bedrock coring details refer to record of drillhole BH04-17 End of Borehole Note: 1. Borehole dry during drilling operations.								



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

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH04-17

SHEET 2 OF 2

LOCATION: N 4974669.2 ;E 316919.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: 22-Nov-04

DRILL RIG: CME 55 Bombardier

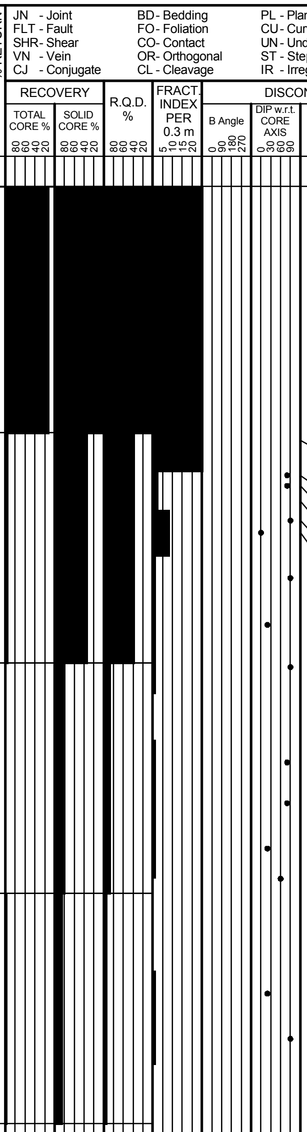
DRILLING CONTRACTOR: Walker Drilling Ltd.

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	

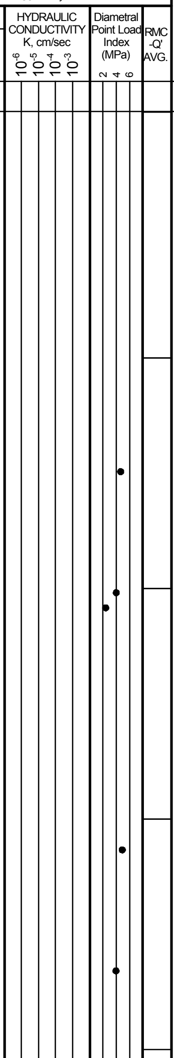
DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		255.47		
	Rotary NQ	Rockfill with sand, contains cobbles and boulders (FILL)	▩	1.52	1	
3		BIOTITE GNEISS Fresh to weathered, medium strong to very strong, fine to coarse crystalline, foliated, dark grey/black and white Infilling and staining along joints from 3.2 m to 3.4 m depth		253.84 3.15	2 3 4	Diametral 3.9 MPa Axial 1.6 MPa
7		GRANITE GNEISS Fresh, strong to very strong, fine to medium crystalline, foliated, dark grey/black and pink, with quartz and feldspar banding		249.99 7.00	4	
		End of Drillhole		249.27 7.72		

Depth scale marks: 2, 3, 4, 5, 6, 7, 8, 9, 10, 11



TYPE AND SURFACE DESCRIPTION

BR,,
BR,,
BR,,
,IR,Ro
,IR,Ro
BR,,
,,
BR,IR,Ro
,IR,Ro
JN,IR,Ro
FO,IR,Ro
FO, VN,IR,Ro
FO, VN,IR,Ro
JN,IR,VR
FO,IR,Ro
JN,IR,SM/Ro
JN, VN,IR,Ro



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: KB

PROJECT 04-1111-039

RECORD OF BOREHOLE No BH04-18

1 OF 1 **METRIC**

W.P. 314-00-00

LOCATION N 4974666.6 ;E 316922.5

ORIGINATED BY SB

DIST

HWY 11

BOREHOLE TYPE 108 mm O.D. Solid Stem Power Auger

COMPILED BY JDR

DATUM Geodetic

DATE 23-Nov-04

CHECKED BY KB

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 (%) 10 20 30

UNIT WEIGHT γ kN/m^3

SOIL PROFILE			SAMPLES						REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE		GR	SA	SI	CL
256.9	GROUND SURFACE											
0.0	Sand, some gravel, trace to some silt, trace roots/topsoil (FILL) Very loose Brown Moist		1	SS	3				14	77	8	1
256.1 0.8	Rockfill with sand, contains cobbles and boulders (FILL) Cored from 0.8 m to 2.3 m						256 255					
254.6 2.3	BIOTITE GNEISS Fresh, medium strong to very strong, fine to medium crystaline, foliated, dark grey/black and white Bedrock cored from 2.3 m to 6.4 m For bedrock coring details refer to record of drillhole BH04-18						254 253 252 251					
250.5 6.4	End of Borehole Note: 1. Borehole dry during drilling operations											

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

○ $^{3\%}$ STRAIN AT FAILURE

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH04-18

SHEET 2 OF 2

LOCATION: N 4974666.6 ;E 316922.5

INCLINATION: -90° AZIMUTH: ---

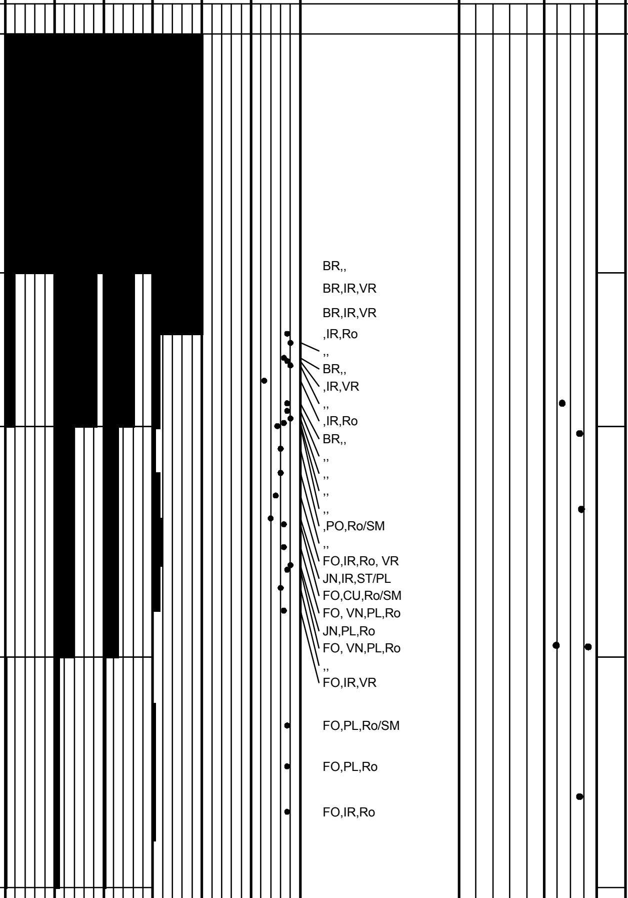
DRILLING DATE: 23-Nov-04
DRILL RIG: CME 55 Bombardier
DRILLING CONTRACTOR: Walker Drilling Ltd.

DATUM: Geodetic

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	
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	

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		256.13		
0.76–2.34	Rotary NQ	Rockfill with sand, contains cobbles and boulders (FILL)	▨	0.76	1	
2.34–6.40	Rotary NQ	BIOTITE GNEISS Fresh, medium strong to very strong, fine to coarse crystalline, foliated, dark grey/black and white with biotite banding		254.55 2.34	2 3 4	Axial 5.4 MPa Diametral 5.5 MPa Axial 6.5 MPa Axial 4.3 MPa
6.40		End of Drillhole		250.49 6.40		



TYPE AND SURFACE DESCRIPTION

BR,,
BR,IR,VR
BR,IR,VR
,IR,Ro
,,
BR,,
,IR,VR
,,
,IR,Ro
BR,,
,,
,,
,,
,,
,PO,Ro/SM
,,
FO,IR,Ro, VR
JN,IR,ST/PL
FO,CU,Ro/SM
FO, VN,PL,Ro
JN,PL,Ro
FO, VN,PL,Ro
,,
FO,IR,VR
FO,PL,Ro/SM
FO,PL,Ro
FO,IR,Ro

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: SB
CHECKED: KB

Golder Associates

RECORD OF BOREHOLE No BH04-19

1 OF 2 **METRIC**

PROJECT 04-1111-039

W.P. 314-00-00

DIST

HWY 11

DATUM Geodetic

LOCATION N 4974629.4 ;E 316887.6

BOREHOLE TYPE Power Augering Using 'N' Casing

DATE 17-Feb-05

ORIGINATED BY CR

COMPILED BY SLP

CHECKED BY KJB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
247.3	GROUND SURFACE								
0.0	Ice								
246.9 0.5	Water								
244.9 2.4	Sand to Sand and Gravel, trace silt, trace organics Very loose to compact Brown (Probable Fill)	1	SS	10					
		2	SS	7					
		3	SS	4		46	53	1	0
242.0 5.3	Cobbles/Boulder encountered between 5.3 m and 5.8 m								
241.5 5.8	Sand, trace to some silt, trace gravel, contains cobbles/boulders Compact Greyish brown (Probable Fill)	4	SS	21					
		5	WS						
239.0 8.4	Sand, some silt, trace gravel and clay Compact Grey	6	SS	14					
		7	SS	12		0	84	14	2
		8	SS	25					
234.1 13.3	Clayey Silt, trace sand, contains sand seams Stiff Grey	9	SS	15					

STRAT PLOT

ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT

20 40 60 80 100

SHEAR STRENGTH kPa

○ UNCONFINED + FIELD VANE

● QUICK TRIAXIAL × REMOULDED

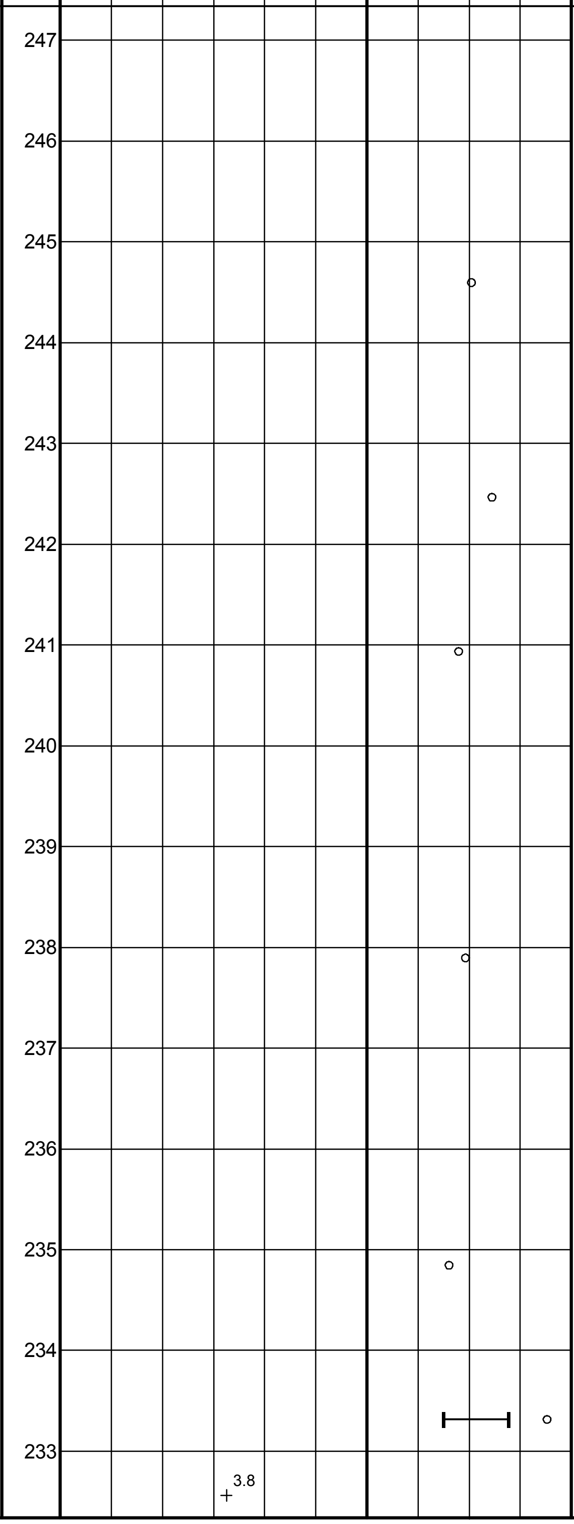
20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%)

10 20 30

UNIT WEIGHT γ kN/m³



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+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

MIS-MTO 001 04111039AAMTO.GPJ GAL-MISS.GDT 3/8/06

RECORD OF BOREHOLE No BH04-19

2 OF 2 **METRIC**

PROJECT 04-1111-039

W.P. 314-00-00

LOCATION N 4974629.4 ;E 316887.6

ORIGINATED BY CR

DIST

HWY 11

BOREHOLE TYPE Power Augering Using 'N' Casing

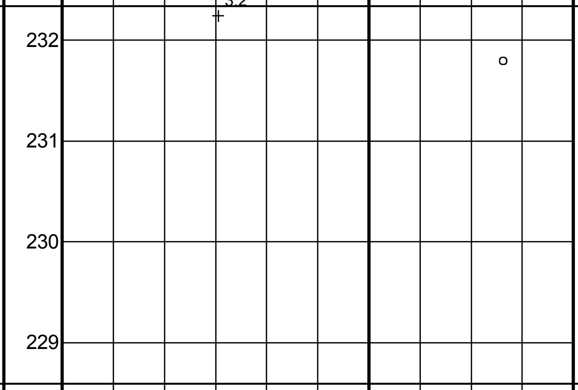
COMPILED BY SLP

DATUM Geodetic

DATE 17-Feb-05

CHECKED BY KJB

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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	NUMBER	TYPE	"N" VALUES						
232.1										
231.8	Sandy Silt, trace clay Grey	10	TO							
231.4 15.9	Sand, trace silt Very dense Grey	~~11~~	~~SS~~	~~100/.07~~						
	GRANITE GNEISS Fresh to slightly weathered, foliated, coarse quartz banding, fine to medium grained, black and pink Bedrock cored from 15.9 m to 18.8 m For bedrock coring details refer to record of drillhole BH04-19									
228.6 18.8	End of Borehole Note: 1. Frozen ice surface (i.e. lake level) at Elevation 247.3 m.									



+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH04-19

SHEET 3 OF 3

LOCATION: N 4974629.4 ;E 316887.6

DRILLING DATE: 22-Feb-05

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

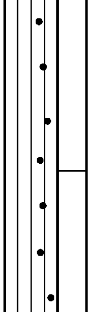
DRILL RIG: Skid Mounted D-25

DRILLING CONTRACTOR: Walker Drilling Ltd.

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.	PENETRATION RATE (m/min)	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 PER 0.3 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -																
16		**GRANITE GNEISS** Fresh, strong to very strong, fine to medium crystalline, foliated, dark grey/black and pink, contains quartz and feldspar banding		15.93	1													
17					2													Axial Axial
18																		
		End of Drillhole		18.75														
19																		
20																		
21																		
22																		
23																		
24																		
25																		



DEPTH SCALE

1 : 50

LOGGED: SB

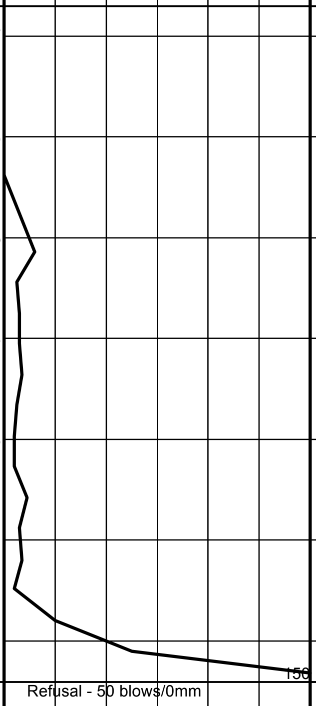
CHECKED: KB

RECORD OF BOREHOLE No DCPT05-1

1 OF 1 **METRIC**

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4974634.7 ;E 316884.0
ORIGINATED BY CR
DIST
HWY 11
BOREHOLE TYPE Skid Mounted Tripod
COMPILED BY SLP
DATUM Geodetic
DATE February 20, 2005
CHECKED BY KB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O 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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
247.3	GROUND SURFACE										
0.0	Ice						247				
246.9 0.4	Water						246				
245.6 1.7	Start of Dynamic Cone Penetration Test (DCPT) at 1.7 m depth						245 244 243 242 241				
240.6 6.7	End of DCPT							Refusal - 50 blows/0mm			



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

Golder Associates

PROJECT 04-1111-039

RECORD OF BOREHOLE No DCPT05-2

1 OF 2 **METRIC**

W.P. 314-00-00

LOCATION N 4974635.9 ;E 316891.1

ORIGINATED BY CR

DIST HWY 11

BOREHOLE TYPE Skid Mounted Tripod

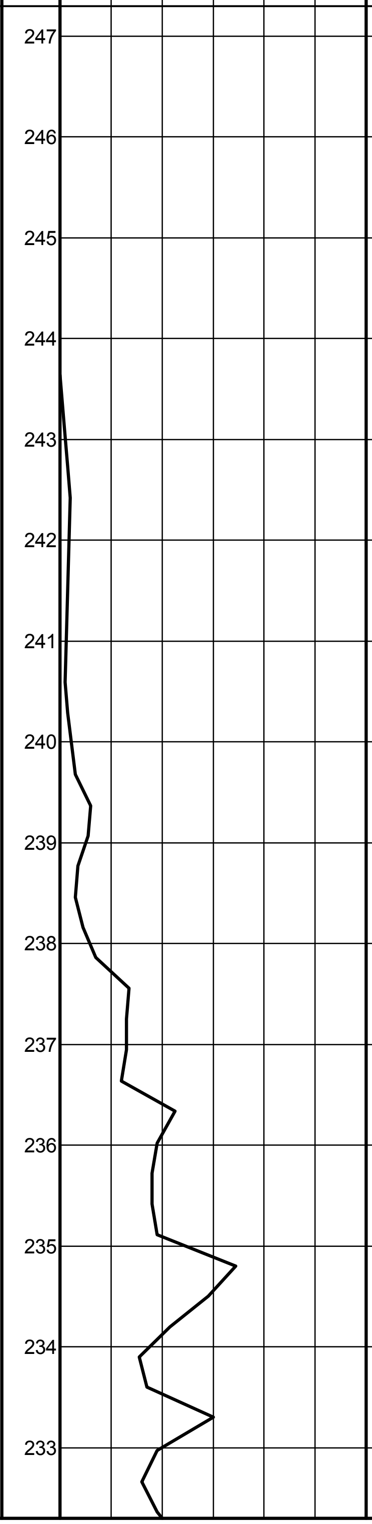
COMPILED BY SLP

DATUM Geodetic

DATE February 20, 2005

CHECKED BY KB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O 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 (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
247.3	GROUND SURFACE										
0.0	Ice										
246.9 0.4	Water										
244.0 3.4	Start of Dynamic Cone Penetration Test (DCPT) at 3.4 m depth										



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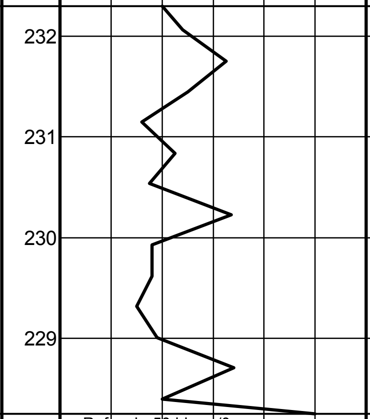
$+^3$, $\times^3$: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No DCPT05-2

2 OF 2 **METRIC**

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4974635.9 ;E 316891.1
ORIGINATED BY CR
DIST
HWY 11
BOREHOLE TYPE Skid Mounted Tripod
COMPILED BY SLP
DATUM Geodetic
DATE February 20, 2005
CHECKED BY KB

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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Start of Dynamic Cone Penetration Test (DCPT) at 3.4 m depth										
228.3 19.1	End of DCPT Cone refusal on infered bedrock							Refusal - 50 blows/0 mm			

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

Golder Associates

RECORD OF BOREHOLE No DCPT05-3

1 OF 2 **METRIC**

PROJECT 04-1111-039

W.P. 314-00-00

LOCATION N 4974630.2 ;E 316890.5

ORIGINATED BY CR

DIST

HWY 11

BOREHOLE TYPE Skid Mounted Tripod

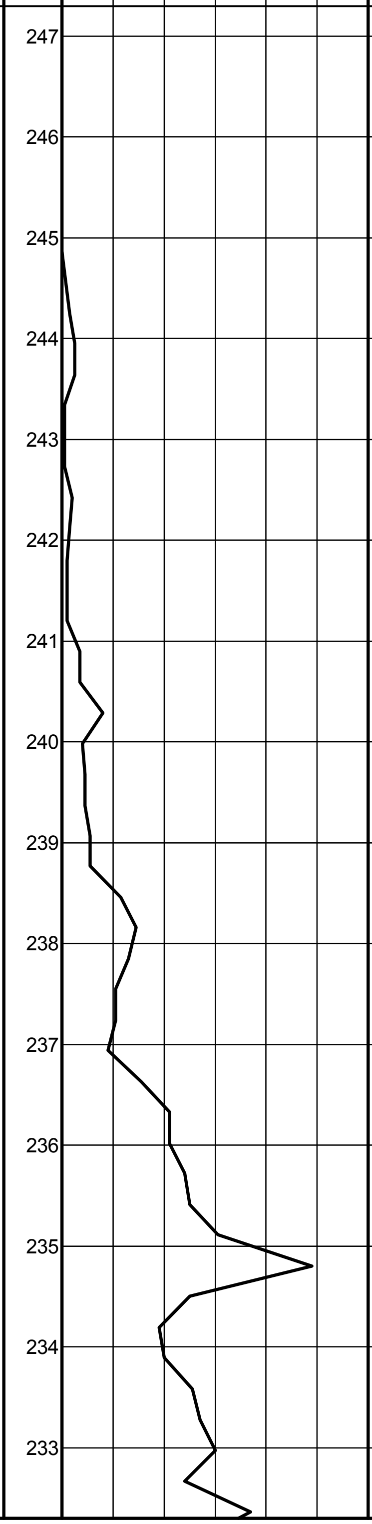
COMPILED BY SLP

DATUM Geodetic

DATE February 20, 2005

CHECKED BY KB

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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
247.3	GROUND SURFACE										
0.0 246.9	Ice										
0.4 244.9	Water										
2.4	Start of Dynamic Cone Penetration Test (DCPT) at 2.4 m depth										



Continued Next Page

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

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

Golder Associates

RECORD OF BOREHOLE No DCPT05-3

2 OF 2 **METRIC**

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4974630.2 ;E 316890.5
ORIGINATED BY CR
DIST
HWY 11
BOREHOLE TYPE Skid Mounted Tripod
COMPILED BY SLP
DATUM Geodetic
DATE February 20, 2005
CHECKED BY KB

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Start of Dynamic Cone Penetration Test (DCPT) at 2.4 m depth						232 231 230				
229.8 17.5	End of DCPT Cone refusal on infered bedrock							Refusal - 50 blows/0 mm			

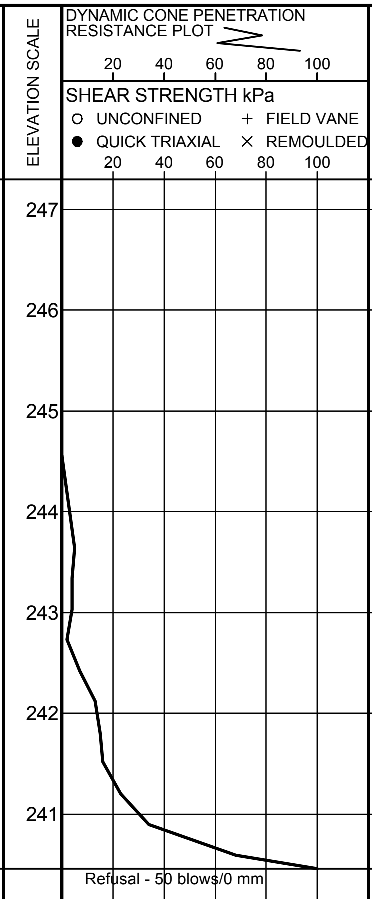
$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No DCPT05-4

1 OF 1 **METRIC**

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4974625.7 ;E 316890.3
ORIGINATED BY CR
DIST HWY 11
BOREHOLE TYPE Skid Mounted Tripod
COMPILED BY SLP
DATUM Geodetic
DATE February 21, 2005
CHECKED BY KB

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
247.3	GROUND SURFACE							
0.0	Ice							
246.8 0.5	Water							
244.6 2.7	Start of Dynamic Cone Penetration Test (DCPT) at 2.7 m depth							
240.5 6.8	End of DCPT							



PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30

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

RECORD OF BOREHOLE No DCPT05-5

1 OF 1 **METRIC**

PROJECT 04-1111-039

W.P. 314-00-00

LOCATION N 4974630.2 ;E 316882.1

ORIGINATED BY CR

DIST HWY 11

BOREHOLE TYPE Skid Mounted Tripod

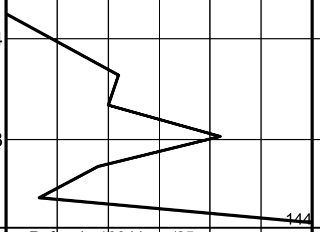
COMPILED BY SLP

DATUM Geodetic

DATE February 20, 2005

CHECKED BY KB

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 (%) 10 20 30	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				
247.3	GROUND SURFACE										
0.0 246.8	Ice	⋮⋮⋮					247				
0.5	Water	▤					246 245				
244.3 3.1	Start of Dynamic Cone Penetration Test (DCPT) at 3.1 m depth						244 243				
242.1 5.2	End of DCPT							Refusal - 100 blows/25 mm			



+³, ×³: Numbers refer to Sensitivity

○ ³% STRAIN AT FAILURE

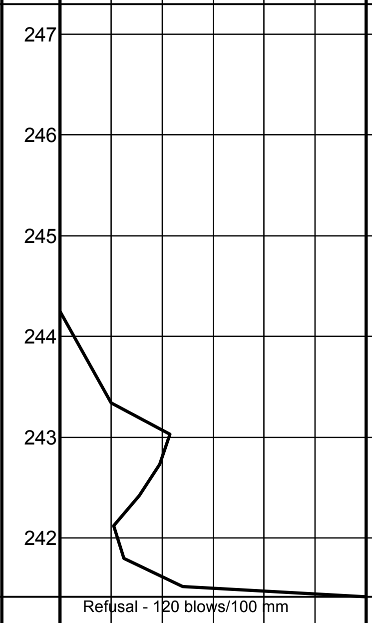
Golder Associates

RECORD OF BOREHOLE No DCPT05-6

1 OF 1 **METRIC**

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4974629.4 ;E 316884.9
ORIGINATED BY CR
DIST
HWY 11
BOREHOLE TYPE Skid Mounted Tripod
COMPILED BY SLP
DATUM Geodetic
DATE February 22, 2005
CHECKED BY KB

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 (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
247.3	GROUND SURFACE										
0.0	Ice						247				
246.8 0.5	Water						246 245				
244.3 3.1	Start of Dynamic Cone Penetration Test (DCPT) at 3.1 m depth						244 243 242				
241.4 5.9	End of DCPT							Refusal - 120 blows/100 mm			



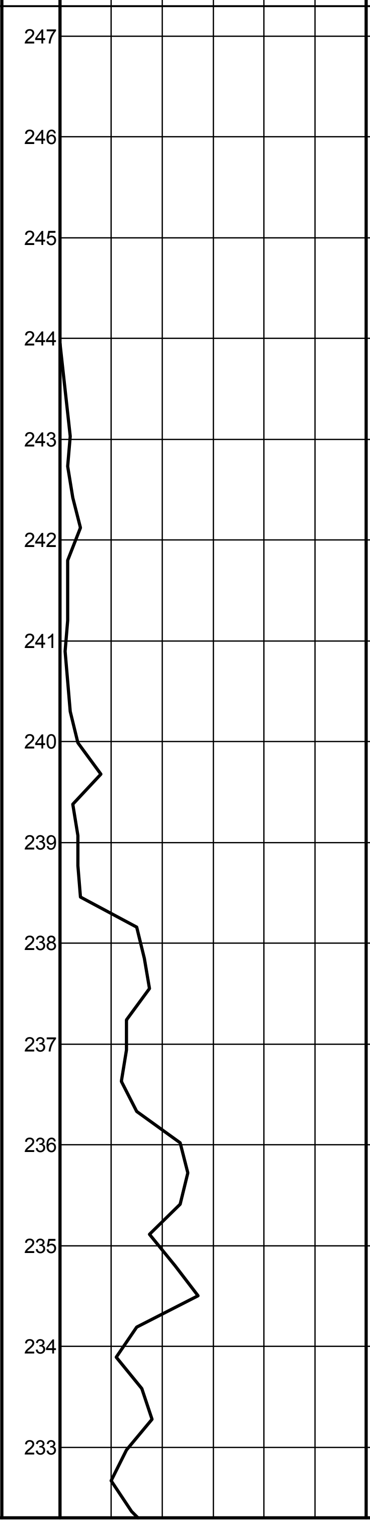
$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No DCPT05-7

1 OF 2 **METRIC**

PROJECT 04-1111-039
W.P. 314-00-00
DIST ______ HWY 11
DATUM Geodetic
LOCATION N 4974631.2 ;E 316894.0
BOREHOLE TYPE Skid Mounted Tripod
DATE February 22, 2005
ORIGINATED BY CR
COMPILED BY SLP
CHECKED BY KB

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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
247.3	GROUND SURFACE										
0.0 246.8	Ice										
0.5 244.0	Water										
3.4	Start of Dynamic Cone Penetration Test (DCPT) at 3.4 m depth										



Continued Next Page

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

RECORD OF BOREHOLE No DCPT05-7

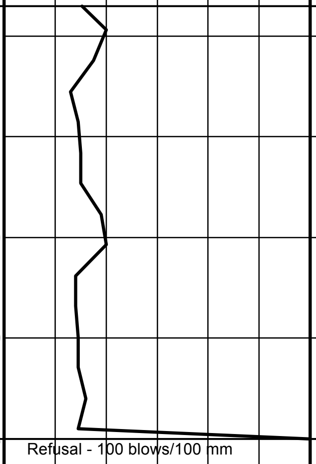
2 OF 2 **METRIC**

PROJECT 04-1111-039
W.P. 314-00-00
DIST HWY 11
DATUM Geodetic

LOCATION N 4974631.2 ;E 316894.0
BOREHOLE TYPE Skid Mounted Tripod
DATE February 22, 2005

ORIGINATED BY CR
COMPILED BY SLP
CHECKED BY KB

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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Start of Dynamic Cone Penetration Test (DCPT) at 3.4 m depth						232 231 230 229				
228.0 19.3	End of DCPT Cone refusal on infered bedrock						228	Refusal - 100 blows/100 mm			



+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO

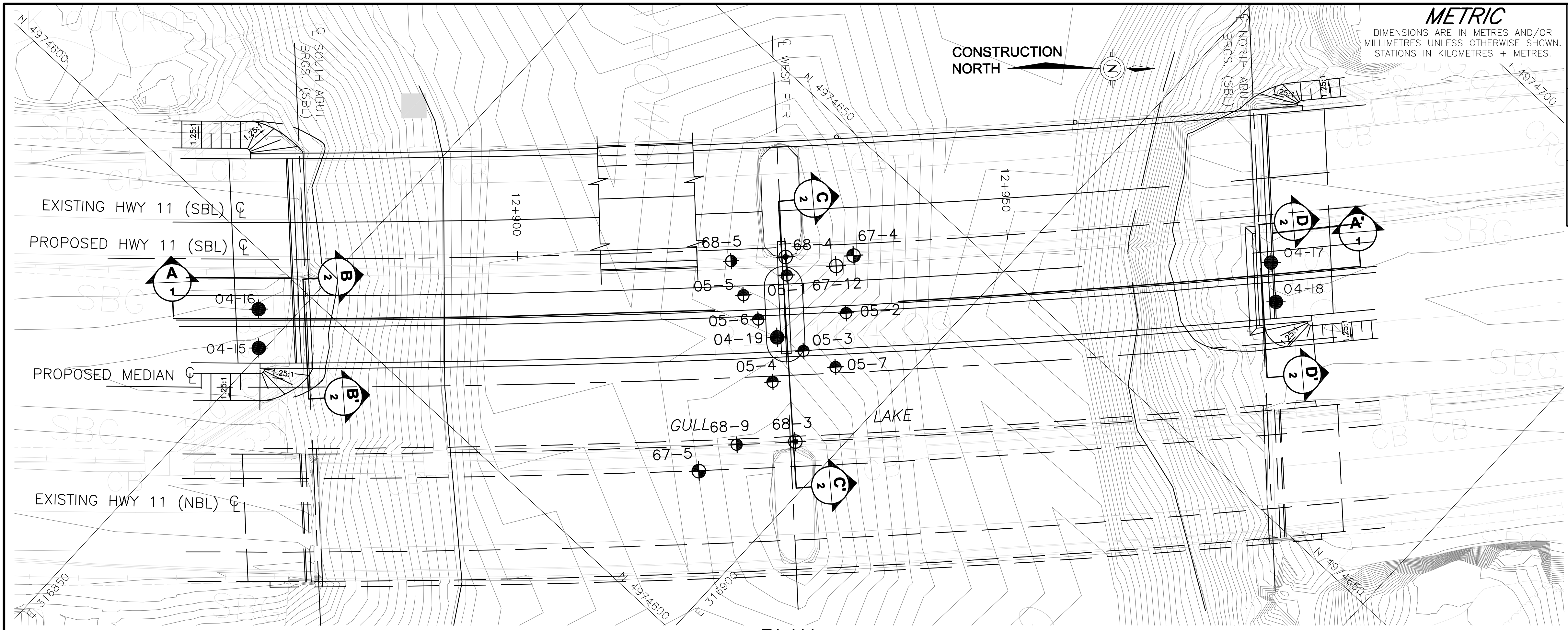
CONT No.
WP No.314-00-00

SHEET

HIGHWAY 11
GULL LAKE BRIDGE SOUTHBOUND LANE WIDENING
BOREHOLE LOCATIONS AND
SOIL STRATA

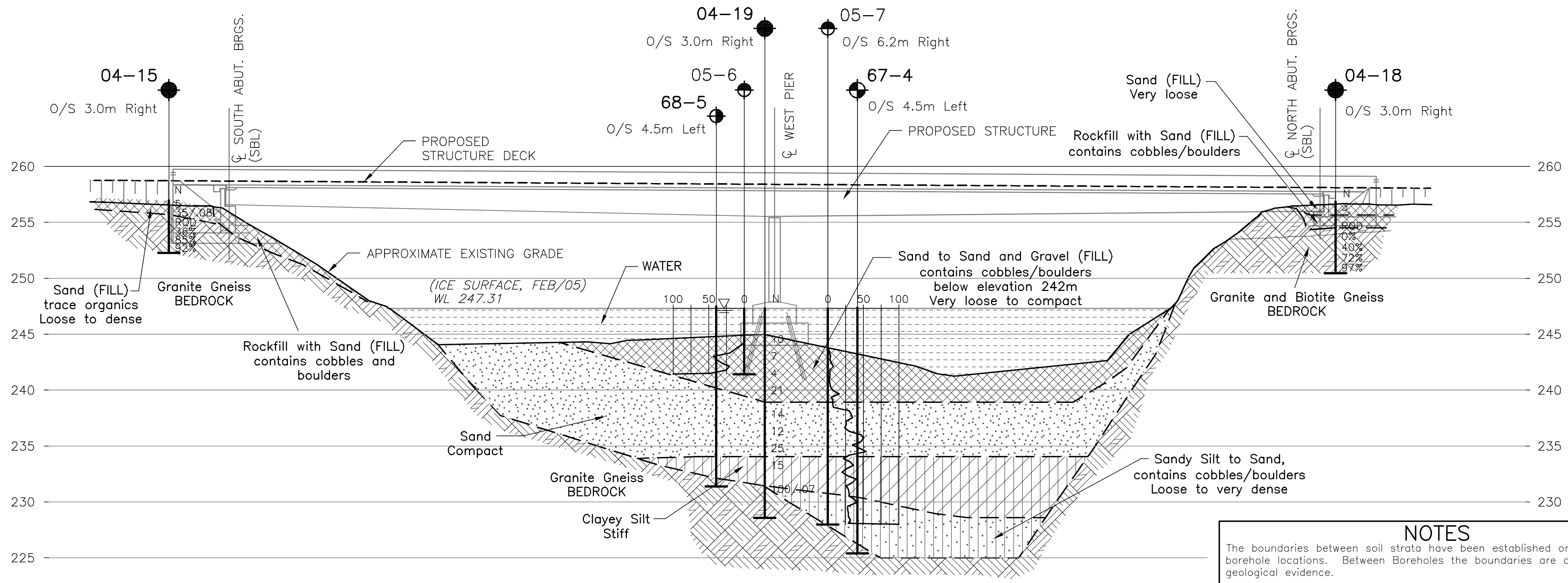
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

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



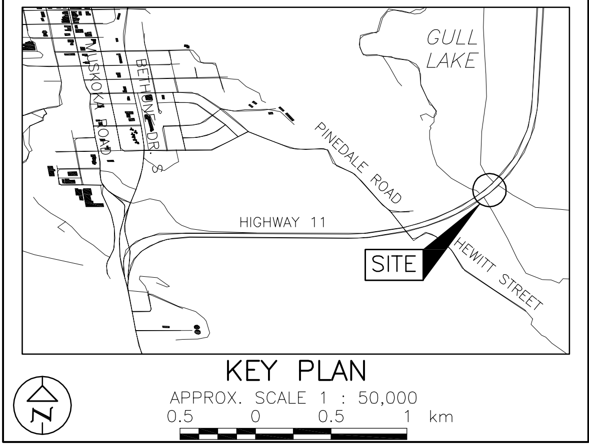
PLAN

SCALE
5 0 5 10 m



A-A' 1 ℄ PROFILE ALONG HIGHWAY 11

SCALE
5 0 5 10 m



KEY PLAN
APPROX. SCALE 1 : 50,000
0.5 0 0.5 1 km

LEGEND

●	Borehole – Current Investigation
⊕	Dynamic Cone Penetration Test – Current Investigation
⊕	Borehole – Trow (1967)
⊕	Probehole – D.H.O. (1968)
⊕	Dynamic Cone Penetration Test–Trow (1967)
⊕	Borehole D.H.O. (1968)
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▽	WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
04-15	257.0	4974592.6	316849.5
04-16	257.4	4974595.5	316846.8
04-17	257.0	4974669.2	316919.4
04-18	256.9	4974666.6	316922.5
04-19	247.3	4974629.4	316887.6
05-1	247.3	4974634.7	316884.0
05-2	247.3	4974635.9	316891.1
05-3	247.3	4974630.2	316890.5
05-4	247.3	4974625.7	316890.3
05-5	247.3	4974630.2	316882.1
05-6	247.3	4974629.4	316884.9
05-7	247.3	4974631.2	316894.0
67-4	247.4	4974640.8	316887.6
67-5	247.4	4974613.9	316891.0
67-12	247.4	4974638.8	316887.1
68-3	247.5	4974622.8	316896.2
68-4	247.5	4974636.0	316882.6
68-5	247.5	4974632.0	316879.0
68-9	247.5	4974618.5	316892.0

NOTES

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.

For subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents

REFERENCE

Base plans and General Arrangement dwg provided in digital format by McCORMICK RANKIN CORPORATION, drawing file nos. 5799-align-May31-05.dwg, 05799_XB1.dwg and General Arrangement dwg file no. 5799-301-001_SBL, received July 18, 2005.

NO.	DATE	BY	REVISION

Geocres No. 31D-414

HWY. 11		PROJECT NO. 04-1111-039B	DIST.
SUBM'D. KJB	CHKD. ASP	DATE: SEPT 2005	SITE: 42-141
DRAWN: JDR/MSM	CHKD. KJB	APPD. FJH	DWG. 1

PLOT DATE: August 03, 2006
FILENAME: T:\Projects\2004\04-1111-039 (MRC, Gravenhurst)\-AB-(GULL LAKE)\04111039AB001.dwg

CONT No.
WP No.314–00–00

SHEET

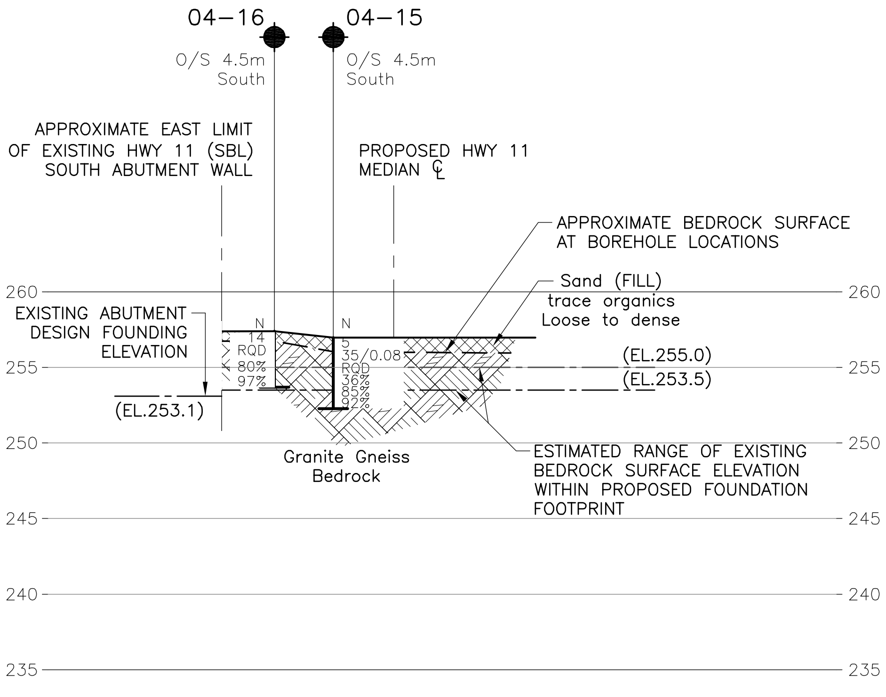
HIGHWAY 11

GULL LAKE BRIDGE SOUTHBOUND LANE WIDENING

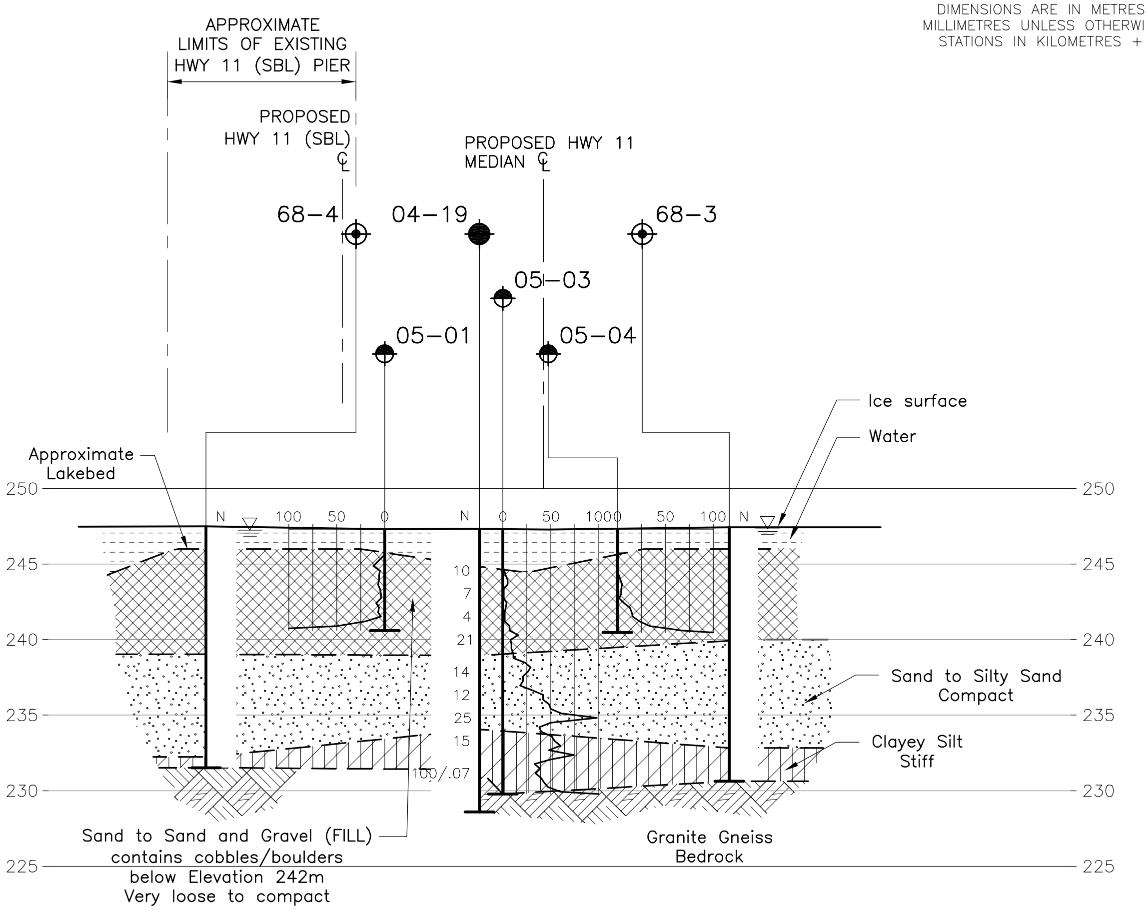
BOREHOLE SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

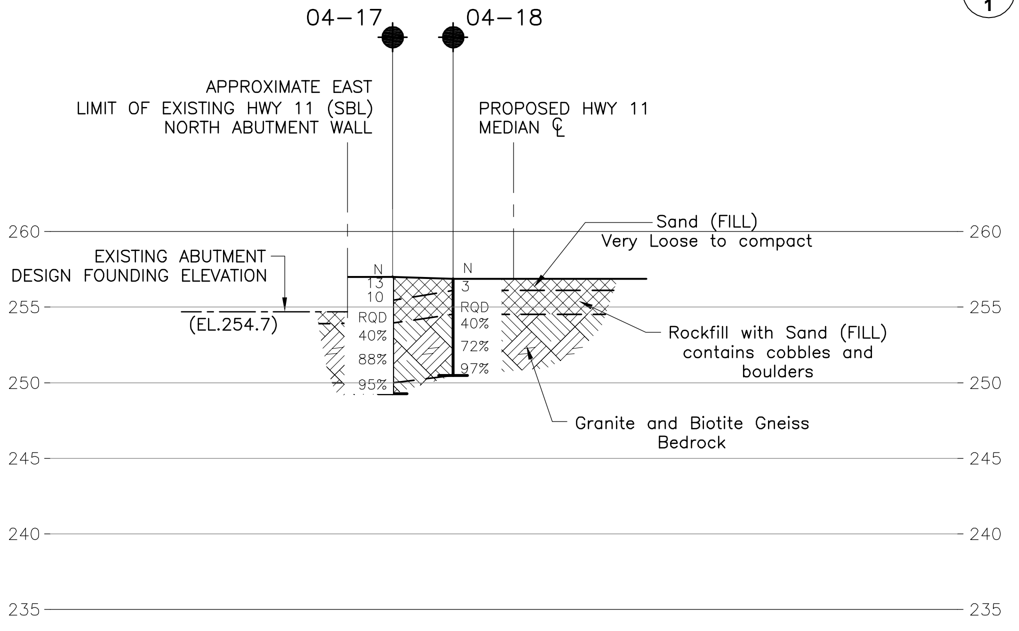
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



B-B' 1 SOUTH ABUTMENT SECTION

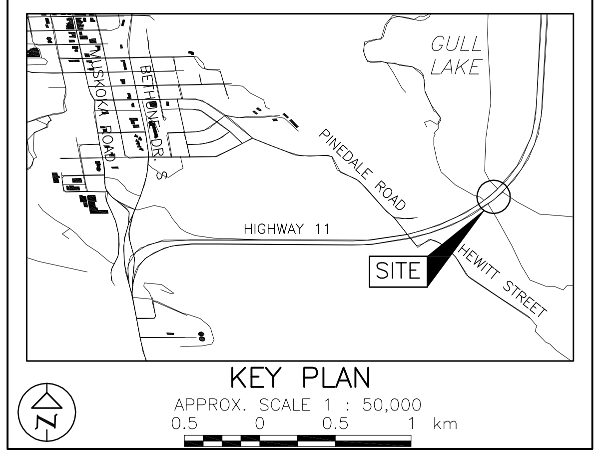


C-C' 1 CENTRE PIER SECTION



D-D' 1 NORTH ABUTMENT SECTION

SCALE
5 0 5 10 m



KEY PLAN
APPROX. SCALE 1 : 50,000
0.5 0 0.5 1 km

LEGEND

●	Borehole – Current Investigation
⊕	Dynamic Cone Penetration Test – Current Investigation
⊕	Borehole – Trow (1967)
⊕	Probehole – D.H.O. (1968)
⊕	Dynamic Cone Penetration Test–Trow (1967)
⊕	Borehole D.H.O. (1968)
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▽	WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
04–15	257.0	4974592.6	316849.5
04–16	257.4	4974595.5	316846.8
04–17	257.0	4974669.2	316919.4
04–18	256.9	4974666.6	316922.5
04–19	247.3	4974629.4	316887.6
05–1	247.3	4974634.7	316884.0
05–2	247.3	4974635.9	316891.1
05–3	247.3	4974630.2	316890.5
05–4	247.3	4974625.7	316890.3
05–5	247.3	4974630.2	316882.1
05–6	247.3	4974629.4	316884.9
05–7	247.3	4974631.2	316894.0
67–4	247.4	4974640.8	316887.6
67–5	247.4	4974613.9	316891.0
67–12	247.4	4974638.8	316887.1
68–3	247.5	4974622.8	316896.2
68–4	247.5	4974636.0	316882.6
68–5	247.5	4974632.0	316879.0
68–9	247.5	4974618.5	316892.0

NOTES

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For subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents

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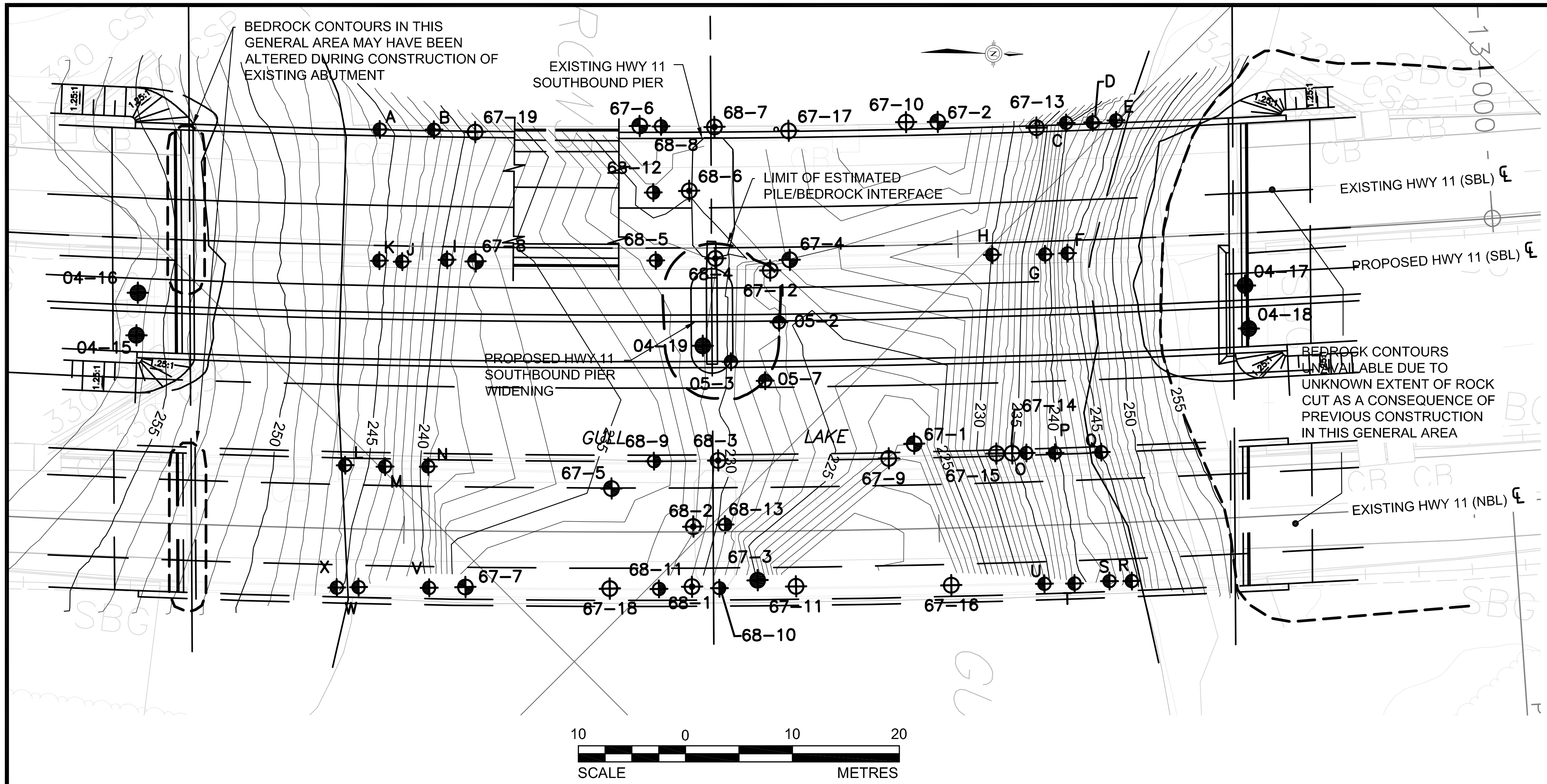
NO.	DATE	BY	REVISION

Geocres No. 31D–414

HWY. 11		PROJECT NO. 04–1111–039B	DIST.
SUBM'D. KJB	CHKD. ASP	DATE: SEPT 2005	SITE: 42–141
DRAWN: JDR/MSM	CHKD. KJB	APPD. FJH	DWG. 2

PLOT DATE: August 03, 2006
FILENAME: T:\Projects\2004\04–1111–039 (MRC, Gravenhurst)\–AB–(GULL LAKE)\0411111039AB002.dwg

FIGURES



LEGEND:

Estimated bedrock surface contour elevation (m)

● Borehole - Current Investigation

⊕ Dynamic Cone Penetration Test - Current Investigation

⊕ Borehole - Trow (1967)

⊕ Probehole - D.H.O. (1968)

⊕ Dynamic Cone Penetration Test-Trow (1967)

⊕ Borehole D.H.O. (1968)

⊕ Probehole - Trow (1967)

Approximate Limit of unknown extent of rock cut

NOTES:

1. DATUM IS GEODETIC

2. LIMIT OF ESTIMATED PILE/BEDROCK INTERFACE IS BASED ON PROPOSED PILE BATTER OF 1H:3.75V AND PROPOSED PIER WIDENING PILE CAP LOCATION PROVIDED BY MRC.

REFERENCES:

MAPPING BASED ON:

1.) BEDROCK CONTOURS AND PREVIOUS BOREHOLES FROM ORIGINAL DESIGN DRAWING NOS. D6107-2 AND D6107-3, TITLED "SOUTH BOUND LANE" AND "NORTH BOUND LANE", GULL LAKE, BOREHOLE LOCATIONS AND SOIL STRATA, PREPARED BY WILLIAM TROW ASSOCIATES LIMITED, DATED MARCH 1967, REVISED MARCH 1968.

2.) BOREHOLE AND DCPT's FROM CURRENT INVESTIGATION.

3.) BEDROCK MAPPING DURING CURRENT INVESTIGATION.

Borehole/DCPT Probehole No.	ESTIMATED BEDROCK ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
A	246.0	4974622.3	316852.0
B	242.8	4974625.8	316855.6
C	239.9	4974668.1	316896.9
D	241.9	4974669.9	316898.6
E	245.1	4974671.6	316900.0
F	244.7	4974659.6	316905.6
G	238.8	4974658.0	316904.1
H	230.2	4974654.5	316900.7
I	239.5	4974618.1	316865.0
J	243.6	4974615.0	316862.2
K	245.7	4974613.6	316860.6
L	246.7	4974597.8	316871.9
M	243.9	4974600.4	316874.6
N	239.2	4974603.2	316877.5
O	236.5	4974643.7	316916.1
P	239.8	4974645.6	316917.9
Q	244.5	4974648.7	316920.9
R	244.1	4974642.2	316931.5
S	241.0	4974640.7	316930.0
T	239.5	4974638.2	316927.9
U	234.0	4974636.2	316925.9
V	239.7	4974595.3	316885.5
W	243.4	4974590.7	316880.8
X	245.9	4974589.2	316879.4
67–1	221.3	4974636.9	316908.0
67–2	222.5	4974659.7	316888.3
67–3	225.2	4974617.5	316906.7
67–4	228.2	4974640.8	316887.6
67–5	234.6	4974613.9	316891.0
67–6	225.4	4974639.7	316868.9
67–7	232.5	4974597.7	316887.9
67–8	236.7	4974619.9	316867.0
67–10	224.3	4974657.6	316886.2
67–11	215.4	4974619.6	316909.7
67–12	224.9	4974638.8	316887.1
67–13	232.2	4974665.8	316895.2
67–14	235.0	4974642.7	316915.1
67–16	217.7	4974630.0	316919.8
67–19	237.7	4974628.4	316858.4

Borehole/DCPT Probehole No.	ESTIMATED BEDROCK ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
68–1	230.8	4974612.7	316902.8
68–2	232.2	4974616.8	316898.9
68–3	230.8	4974622.8	316896.2
68–4	231.6	4974636.0	316882.6
68–5	231.5	4974632.0	316879.0
68–6	229.8	4974638.7	316876.5
68–7	229.4	4974644.6	316873.9
68–8	228.5	4974641.1	316870.3
68–9	235.2	4974618.5	316892.0
68–10	228.3	4974614.4	316904.7
68–11	232.5	4974610.4	316900.8
68–12	229.1	4974636.2	316874.2
68–13	229.3	4974619.0	316900.9
04–15	256.0	4974592.6	316849.5
04–16	256.7	4974595.5	316846.8
04–17	253.8	4974669.2	316919.4
04–18	254.6	4974666.6	316922.5
04–19	231.4	4974629.4	316887.6
05–2	228.3	4974635.9	316891.1
05–3	229.8	4974630.2	316890.5
05–7	228.0	4974631.2	316894.0

PLOT DATE: August 03, 2006
FILENAME: T:\Projects\2004\04-1111-039 (MRC, Gravenhurst)\-AB-(GULL LAKE)\RCK_041111039AB001.dwg

Golder Associates
Mississauga, Ontario, Canada

FILE No. RCK_041111039AB001.dwg
PROJECT No. 04-1111-039
REV.

SCALE AS SHOWN
DATE FEB., 2006
DESIGN
CAD JFC/MSM
CHECK KJB
REVIEW ASP

TITLE

ESTIMATED BEDROCK SURFACE CONTOURS

HIGHWAY 11
GULL LAKE BRIDGE SOUTHBOUND
LANE WIDENING

FIGURE
1

PLOT DATE: August 03, 2006
FILENAME: T:\Projects\2004\04-1111-039 (MRC, Gravenhurst)\-FA- (FIGURES)\041111039FA002.dwg

SCHEMATIC OF TYPICAL EMBANKMENT FILL DETAILS AT ABUTMENTS
GULL LAKE BRIDGE SBL WIDENING, HIGHWAY 11
G.W.P. 314-00-00

FIGURE 2

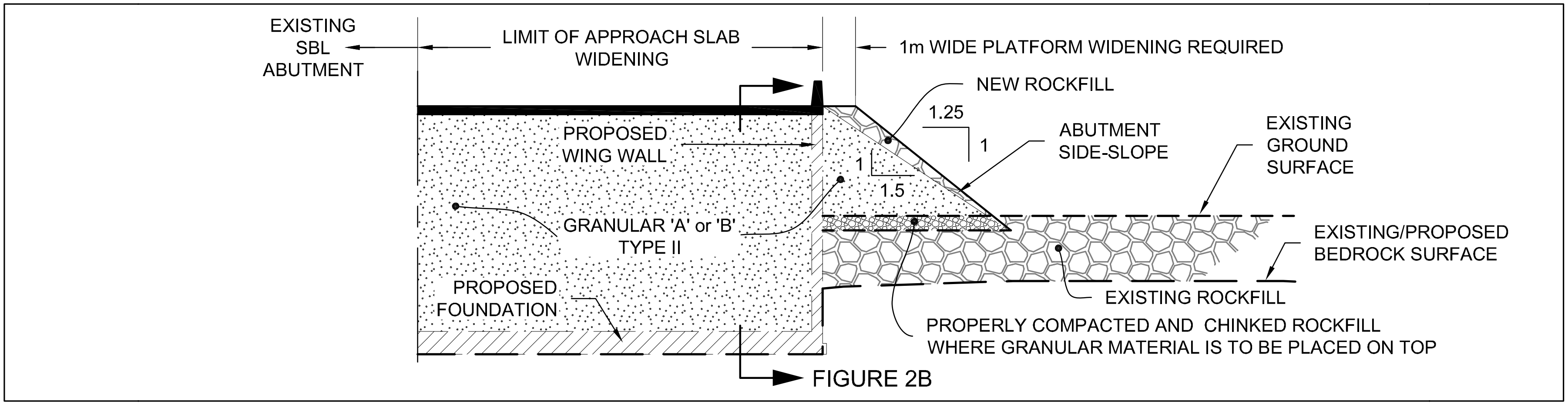
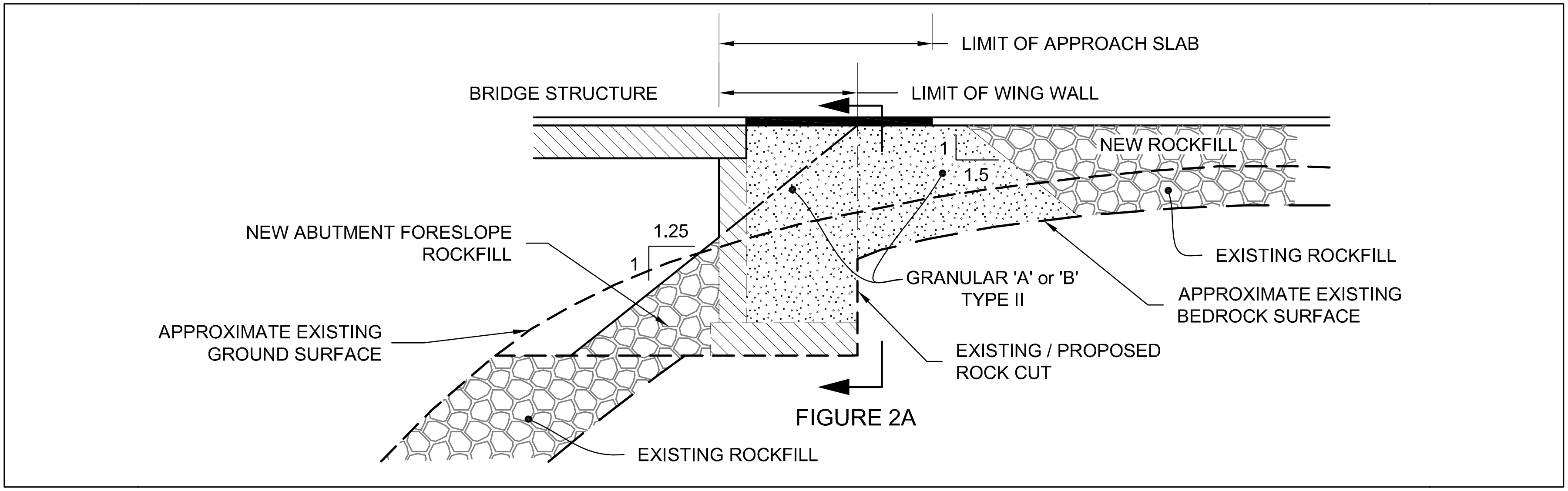


FIGURE 2A - TYPICAL SECTION ACROSS APPROACH SLAB



ALL DRAWINGS NOT TO SCALE AND NOT FOR CONSTRUCTION

FIGURE 2B - TYPICAL PROFILE SECTION AT ABUTMENT

DATE: MARCH 2006
PROJECT: 04-1111-039B

Golder Associates

CAD: MSM
CHK: KJB

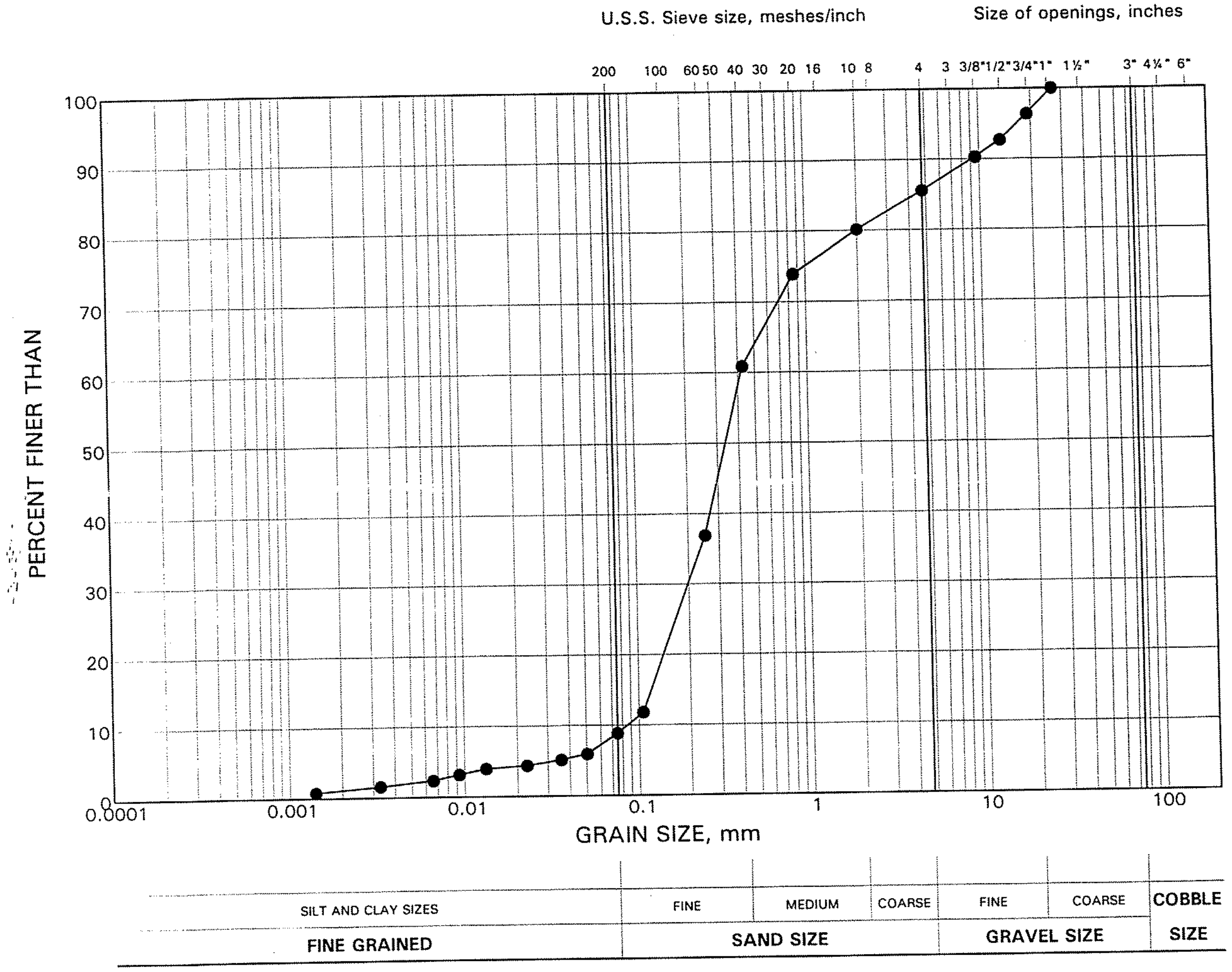
APPENDIX A

LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION

Sand (Fill)

FIGURE A1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	18	1	256.6

Date August, 2005

Project 04-1111-039B

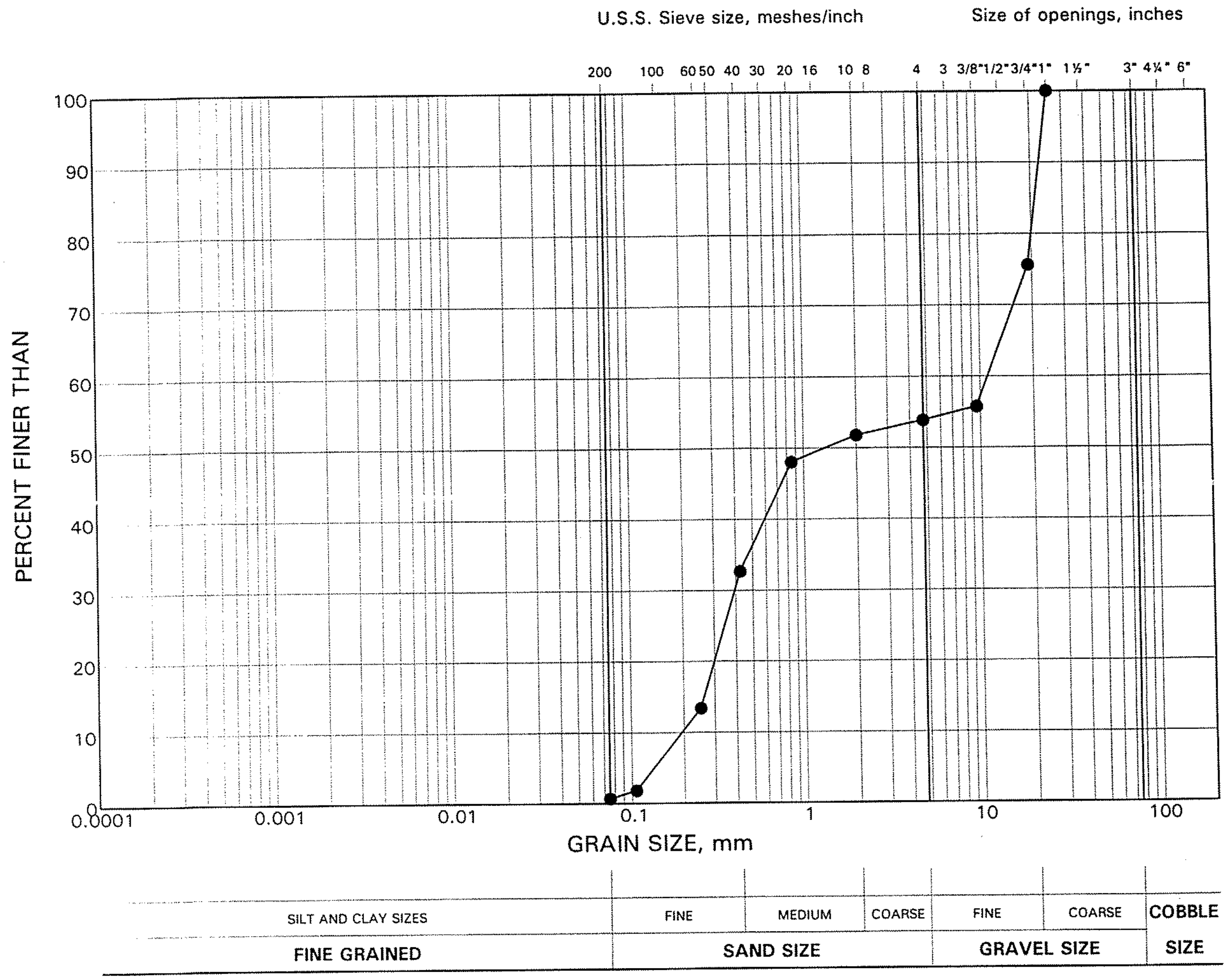
Golder Associates

Prepared by LG

Checked by KJB

GRAIN SIZE DISTRIBUTION
Sand and Gravel (Probable Fill)

FIGURE A2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	04-19	3	242.4

Date August, 2005

Project 04-1111-039B

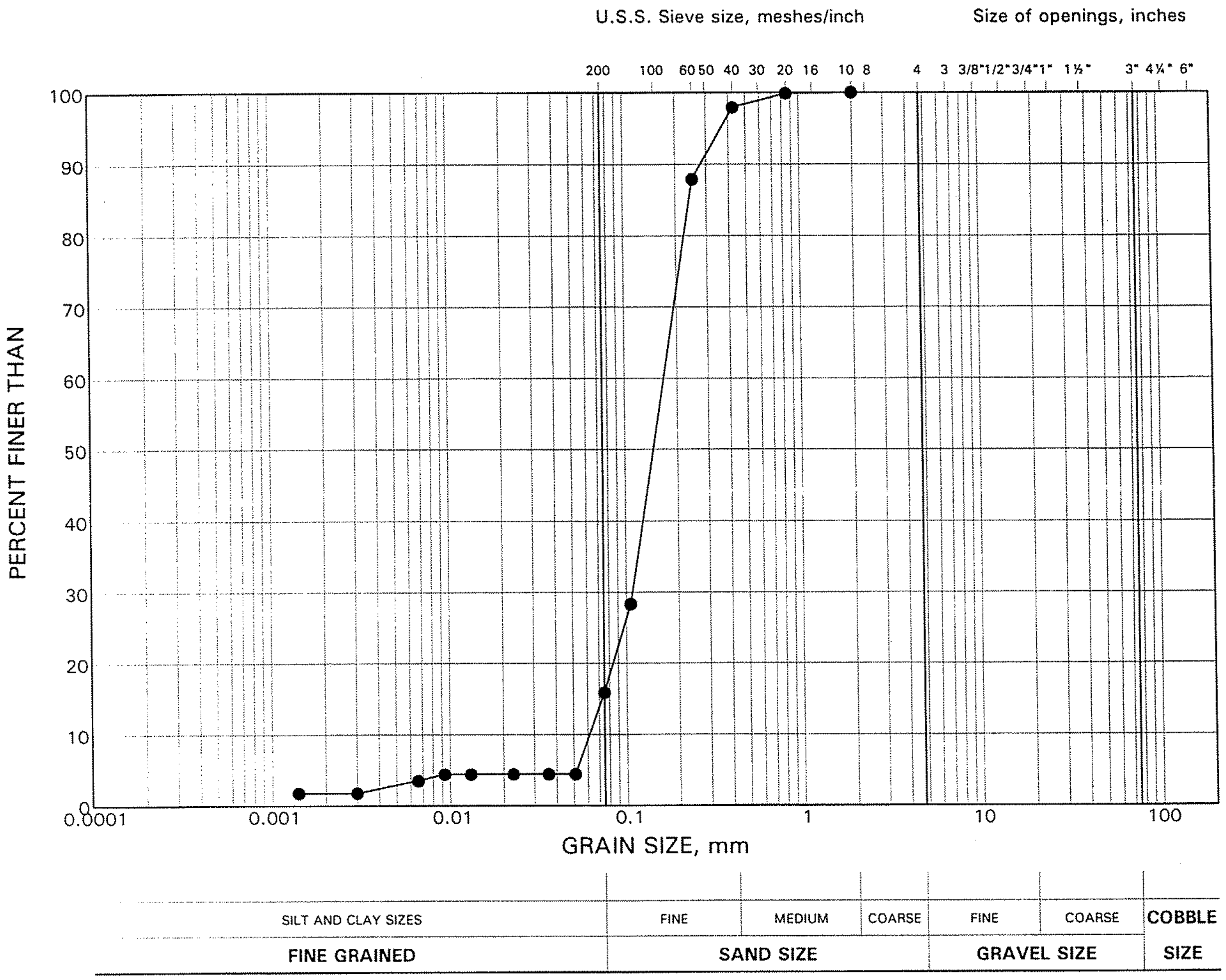
Golder Associates

Prepared by LG

Checked by KJP

GRAIN SIZE DISTRIBUTION
Sand

FIGURE A3



LEGEND

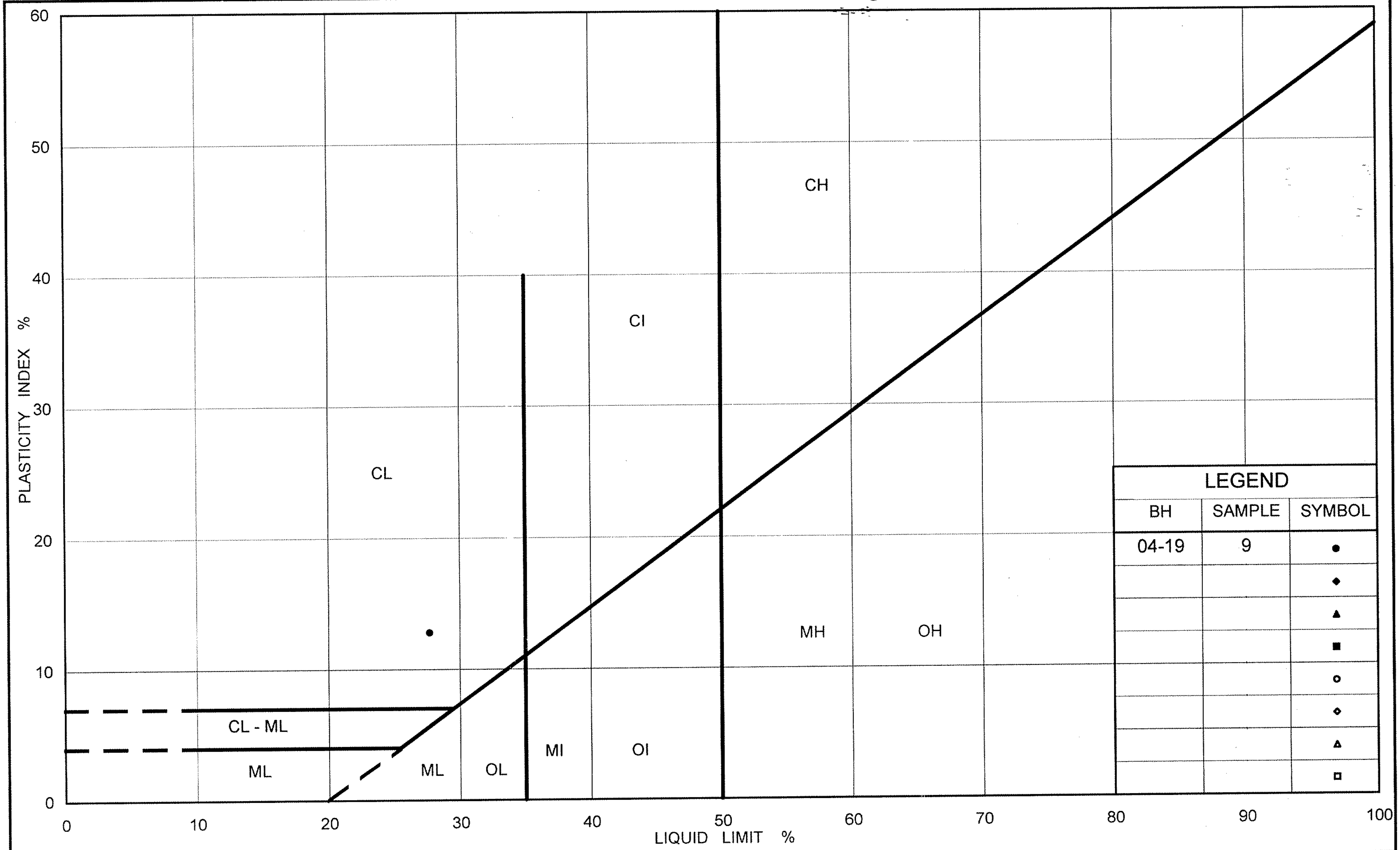
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	04-19	7	236.3

Date August, 2005
Project 04-1111-039B

Golder Associates

Prepared by LG
Checked by KJB

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
04-19	9	●
		◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt

FIG No. A4

Project No. 04-1111-039B

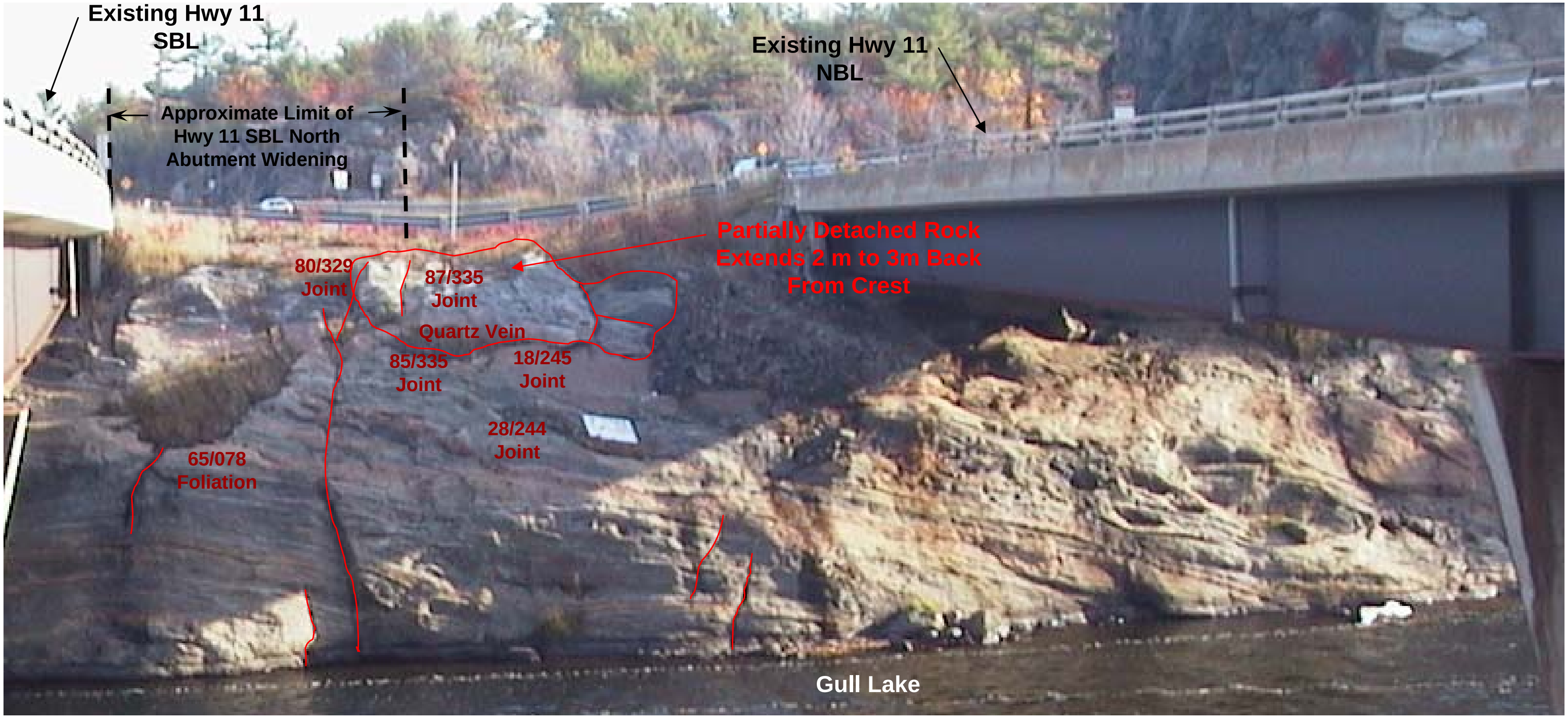
APPENDIX B

ROCK HAZARD ASSESSMENT PHOTOGRAPHS

Proposed North Abutment Location

Gull Lake Narrows SBL Widening, Highway 11

FIGURE B1



North Abutment Location

Date: March 11, 2005
Project: 041111039B

Golder Associates

Drawn: MJT
Chkd: KJB

APPENDIX C

SUBSURFACE INFORMATION FROM PREVIOUS INVESTIGATIONS

Additional Boreholes carried out by the Foundation Section
At Gull Lake and Hwy. (400) - 11 Line 'D' for Centre Pier
Locations of the Northbound and Southbound Lane Structures.

No.	Line 'D', Hwy. #11	Subsoil Conditions			Datum Elev. (Ice)	Rock Elev.
68-1 ⌖	102+77 O/S 70.5' Rt.	0	- 15"	Ice	811.98	756.98
		15"	- 24'7"	Water		
		24'7"	- 45'	Silty Fine Sand		
		45'	- 55'	Clayey Silt with Sand		
		55'	-	Sound Bedrock - Banded Paragneiss with pink granitic injections.		
68-2 ⌖	102+77 O/S 51' Rt.	0	- 15"	Ice	811.96	761.66
		15"	- 24'	Water		
		24'	- 43'	Silty Fine Sand		
		43'	- 50'4"	Clayey Silt with Sand		
		50'4"	-	Sound Bedrock - Banded Paragneiss with pink granitic injections.		
68-3 ⌖	102+85 O/S 31.5' Rt.	0	- 15"	Ice	811.98	756.98
		15"	- 24'8"	Water		
		24'8"	- 48'	Silty Fine Sand		
		48'	- 55'	Clayey Silt with Sand		
		55'	-	Sound Bedrock - Banded Paragneiss with pink granitic injections.		
68-4 ⌖	102+85 O/S 30' Lt.	0	- 15"	Ice	811.98	759.68
		15"	- 27'5"	Water		
		27'5"	- 50'	Silty Fine Sand		
		50'	- 52'3"	Clayey Silt with Sand		
		52'3"	-	Sound Bedrock - Banded Paragneiss with pink granitic injections.		
68-5 ◎	102+69 O/S 30' Lt.	0	- 15"	Ice	812.04	759.24
		15"	- 24'	Water		
		24'	- 52'	Silty Fine Sand		
		52'	- 52'10"	Clayey Silt with Sand		
		52'10"	-	Hard contact		

cont'd. /2 ...

Additional Boreholes carried out by the Foundation Section At Gull Lake and Hwy. (400) - 11 Line 'D' for Centre Pier Locations of the Northbound and Southbound Lane Structures.

No.	Line 'D', Hwy. #11	Subsoil Conditions	Datum Elev. (Ice)	Rock Elev.
68-6 ⌖	102+77 O/S 51' Lt.	0 - 15" Ice 15" - 26'2" Water 26'2" - 37' Silty Fine Sand 37' - 58' Clayey Silt with Sand 58' - Sound Bedrock - Banded Paragneiss with pink granitic injections.	811.89	753.89
68-7 ⌖	102+85 O/S 70.5' Lt.	0 - 15" Ice 15" - 29' Water 29' - 40' Silty Fine Sand 40' - 59'6" Probably same as above. 59'6" - Sound Bedrock - Banded Paragneiss with pink granitic injections.	811.85	752.35
68-8 ◎	102+68 O/S 70.5' Lt.	0 - 15" Ice 15" - 26' Water 26' - 40' Silty Fine Sand 40' - 62'6" (Wash ahead) 62'6" - Refusal - probable Bedrock.	811.85	749.35
68-9 ◎	102+67 O/S 31.5' Rt.	0 - 15" Ice 15" - 22'4" Water 22'4" - 40'8" Silty Fine Sand 40'8" - Refusal, probably Bedrock.	811.98	771.28
68-10 ◎	102+85 O/S 70.5' Rt.	0 - 15" Ice 15" - 24'9" Water 24'9" - 40' Silty Fine Sand 40' - 63'2" (Wash ahead of Casing) 63'2" - Refusal, probable Bedrock.	811.98	748.78

cont'd. /3 ...

Additional Boreholes carried out by the Foundation Section At Gull Lake and Hwy. (400) - 11 Line 'D' for Centre Pier Locations of the Northbound and Southbound Lane Structures.

No.	Line 'D', Hwy. #11	Subsoil Conditions			Datum Elev. (Ice)	Rock Elev.
68-11 ◎	102+67 O/S 70.5' Rt.	0 15" 20'3" 40' 49'4"	- 15" - 20'3" - 40' - 49'4" -	Ice Water Silty Fine Sand (Wash ahead of Casing) Refusal, probable Bedrock.	811.89	762.59
68-12 ◎	102+67 O/S 51' Lt.	0 15" 23'10" 40' 60'6"	- 15" - 23'10" - 40' - 60'6" -	Ice Water Silty Fine Sand (Wash ahead of Casing) Refusal, probable Bedrock.	811.89	751.39
68-13 ◎	102+87 O/S 51' Rt.	0 15" 26'4" 40' 59'10"	- 15" - 26'4" - 40' - 59'10" -	Ice Water Silty Fine Sand (Wash ahead of Casing) Refusal, probable Bedrock.	811.96	752.16

⌖ Borehole carried out during February 1968 - Rock samples obtained with AXT Core Barrel.

◎ Probe Hole carried out during February 1968 - Rock contac established by Wash boring techniques.

WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO 1
PROJECT NO J3485

LEGEND

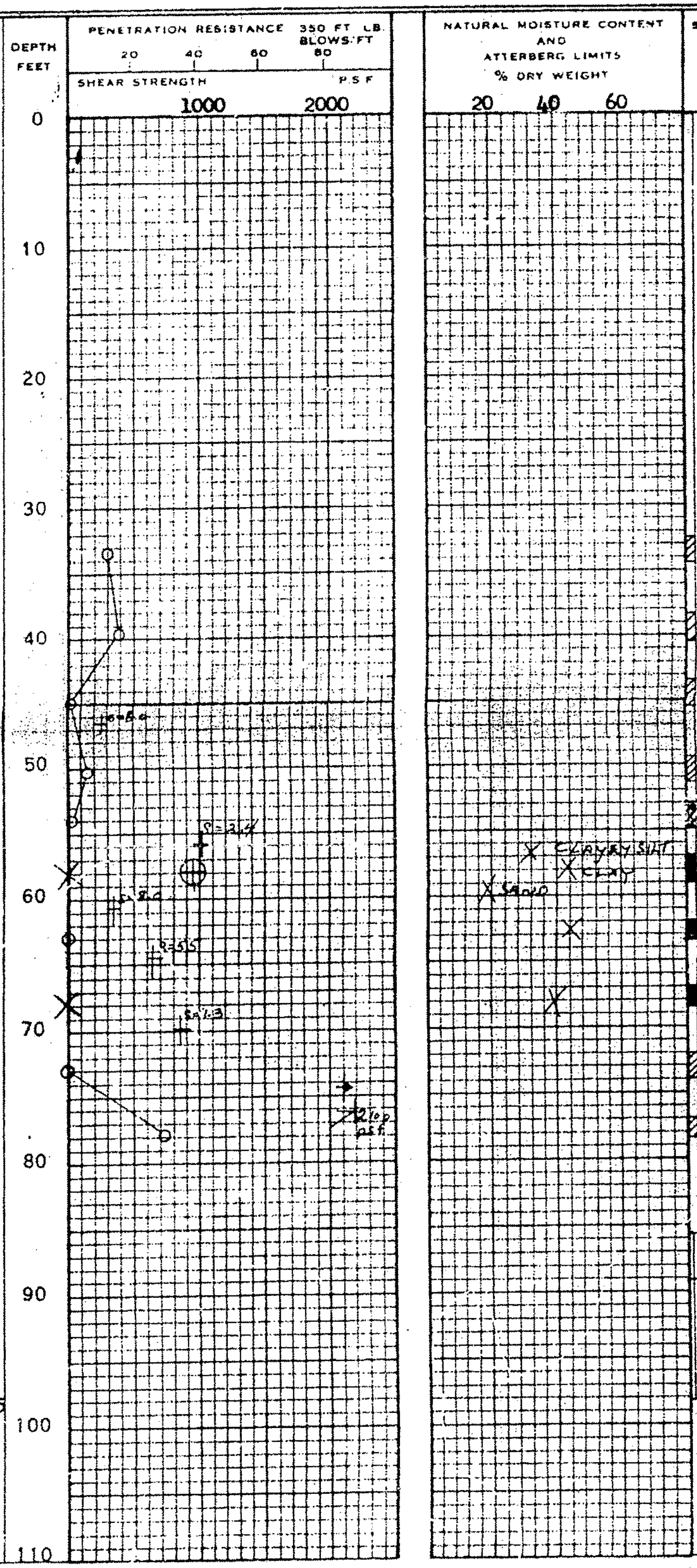
PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —x—x—x—
2" DIA. CONE ———
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) +S

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX
ATTERBERG LIMITS
LIQUID LIMIT ---o
PLASTIC LIMIT ├—
SAMPLE TYPE
2" O.D. SPLIT TUBE ▨
2" I.D. SHELBY TUBE ■ P = Pushed
3" O.D. SHELBY TUBE ▩ L = Levered

BOREHOLE NO. 67-1
PROJECT Proposed Gull Lake Crossing,
LOCATION Gravenhurst By-Pass, Hwy. No. 11 (400)
HOLE LOCATION Sta. 103+45; 26'R
HOLE ELEVATION 811.74 ft.
DATUM G.B.M. DCCXIII
Publication 19 - Gravenhurst

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET
		811.7	0
	WATER		
	SAND- fine with a little medium, silty, grey, compact; becomes finer and more silty with depth.	783.8	
	SILT and CLAY LAYERS with numerous fine sand seams throughout, predominantly silt, soft to firm, grey, wet.	769.7	
	CLAY- silty, firm, grey; becomes more silty below about 65 feet depth, few clayey silt layers and fine sand seams.	749.7	
	SILT- sandy, very stiff, wet, few thin clay seams, grey. -few small boulders at bedrock surface	737.7	
	BEDROCK- Banded paragneiss with pink granitic injections, sound.	726.0	
	End of Borehole	713.5	

SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
1	
2	
3	
4	
5	
6	114
7 P	110
8 L	
9 P	113
10	
89%	Recovery
100%	Recovery



Notes: 1) Hole cased with flush joint casing and advanced by conventional wash boring methods to 85.7 ft. depth and then by conventional diamond drilling methods; to 98.2 ft. depth. Bedrock core recovered in AXT size.

WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 2
PROJECT NO. J3485

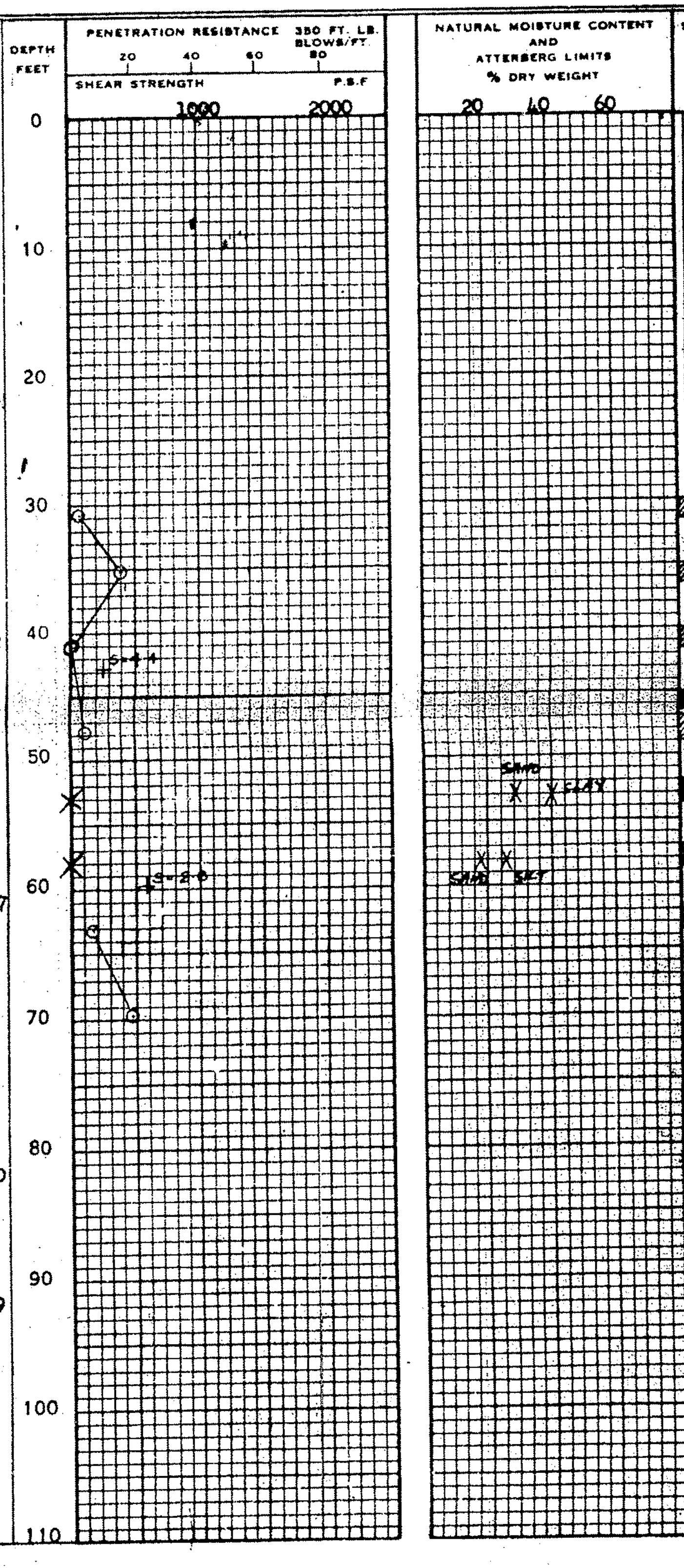
LEGEND

PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X^{LI}
2" O.D. SPLIT TUBE	—o—o—o—	ATTERBERG LIMITS	
2" I.D. SHELBY TUBE	—x—x—x—x—	LIQUID LIMIT	—o
2" DIA CONE	———	PLASTIC LIMIT	⊢—
SHEAR STRENGTH		SAMPLE TYPE	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	⊕	2" O.D. SPLIT TUBE	▨
UNCONFINED COMPRESSION	⊖	2" I.D. SHELBY TUBE	■ P = Pushed
VANE TEST AND SENSITIVITY (S)	$+^{S}$	3" O.D. SHELBY TUBE	■ L = Levered

BOREHOLE NO. 67-2
PROJECT Proposed Gull Lake Crossing,
LOCATION Gravenhurst By-Pass, Hwy. No. 11 (400)
HOLE LOCATION Sta. 103+54; 72'L
HOLE ELEVATION 811.74 ft.
DATUM G.B.M. DCCXIII
Publication 19 - Gravenhurst

SYMBOL	SOIL DESCRIPTION	ELEV FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
		811.7		
	WATER			
⁞	SAND- loose to compact, fine, few medium pockets, slightly silty to about 37 ft.depth then very silty, brown, becoming grey below about 33 ft.depth, few clayey pockets below about 37 ft.depth.	784.5	▨ 1 ▨ 2	
╱	SILT and CLAY LAYERS- soft to firm, grey, numerous fine sand seams throughout, predominantly silt, wet.	771.2	▨ 3 P ■ 4 P ▨ 5 ■ 6 L ■ 7 L	 128.2 126.1
⁞	SAND- grey, fine, silty, numerous cobbles and boulders below about 72 ft.depth.	750.7	▨ 8 ■ 9 10 [illegible]	
	BEDROCK- banded paragneiss with pink granitic injections, sound.	730.0	100% Recovery	
	End of Borehole	719.9		

Notes: 1) Hole cased with flush joint casing and advanced using conventional wash boring methods to 81.7 ft.depth, and then by conventional diamond drilling methods to 91.8 ft.depth. Bedrock core recovered in AXT size.



WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO 3
PROJECT NO J3485

BOREHOLE NO 67-3
PROJECT Proposed Gull Lake Crossing,
LOCATION Gravenhurst By-Pass, Hwy. No. 11 (400)
HOLE LOCATION Sta. 102+98; 68'R
HOLE ELEVATION 811.74 ft.
DATUM G.B.M. DCCXIII
Publication 19 - Gravenhurst

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —✱—✱—✱—✱—
2" DIA. CONE ————

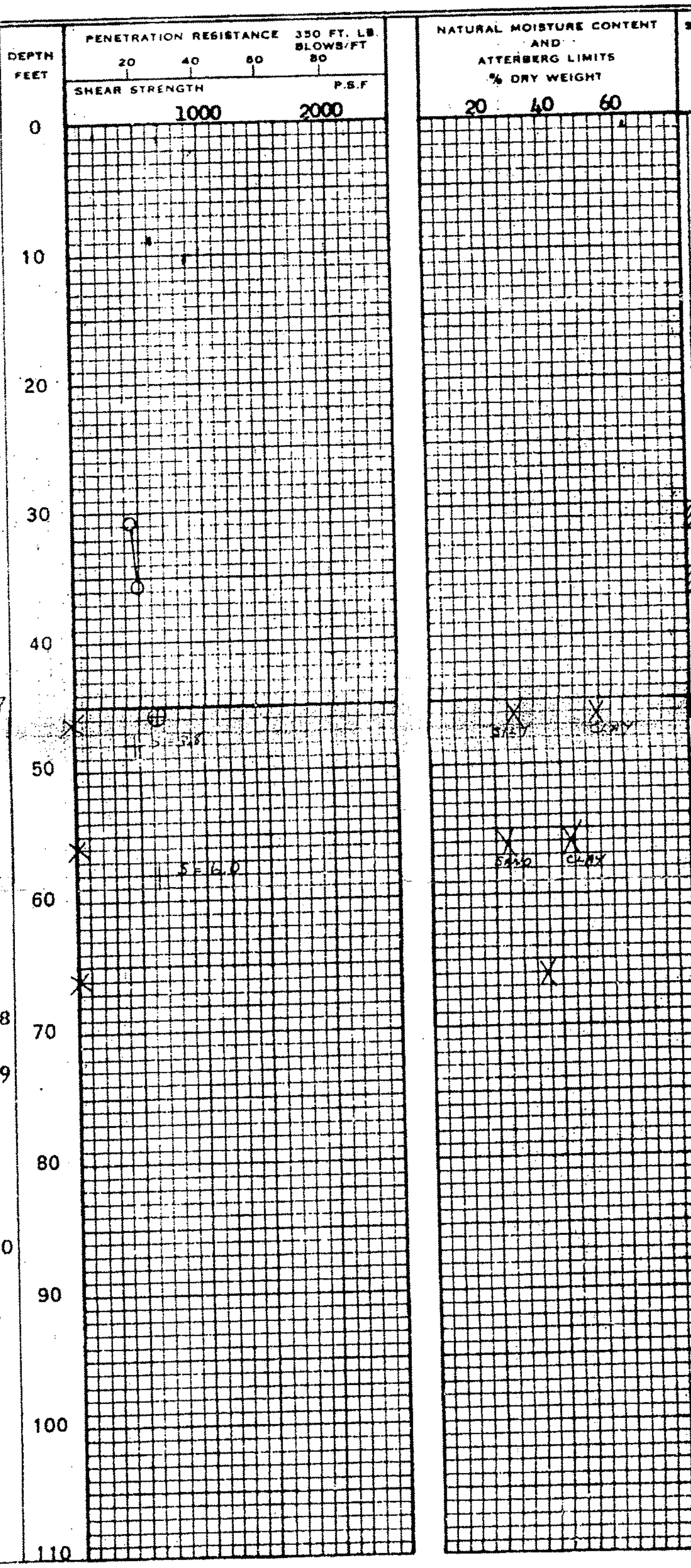
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊖
VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT ⊢—

SAMPLE TYPE
2" O.D. SPLIT TUBE ▨
2" I.D. SHELBY TUBE ■ P = Pushed
3" O.D SHELBY TUBE ▥

SOIL DESCRIPTION	ELEV. FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	811.7		
WATER			
SAND- fine with a little medium, compact, grey, slightly silty, somewhat layered.	786.3	1	
		2	
-few silty layers below about 38 ft. depth.			
SILT and CLAY LAYERS- soft to firm, grey, very silty above about 53 ft. depth; becoming less silty with depth, few fine sand seams throughout, wet.	767.7	3 P	111
		4 P	111.8
		5 P	112.7
SAND and GRAVEL- grey, few cobbles and small boulders.	742.8		
BEDROCK- banded paragneiss with pink granitic injections, sound.	738.9	96.6% Recovery	
		96.4% Recovery	
		100% Recovery	
End of Borehole	726.0		



Notes: 1) Hole cased to 72 ft.depth with flush joint casing and advanced to 72.8 ft.depth using conventional wash boring methods. Bedrock core recovered below 72.8 ft.depth in AXT size by conventional diamond drilling methods.

WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 4
PROJECT No. J3485

LEGEND

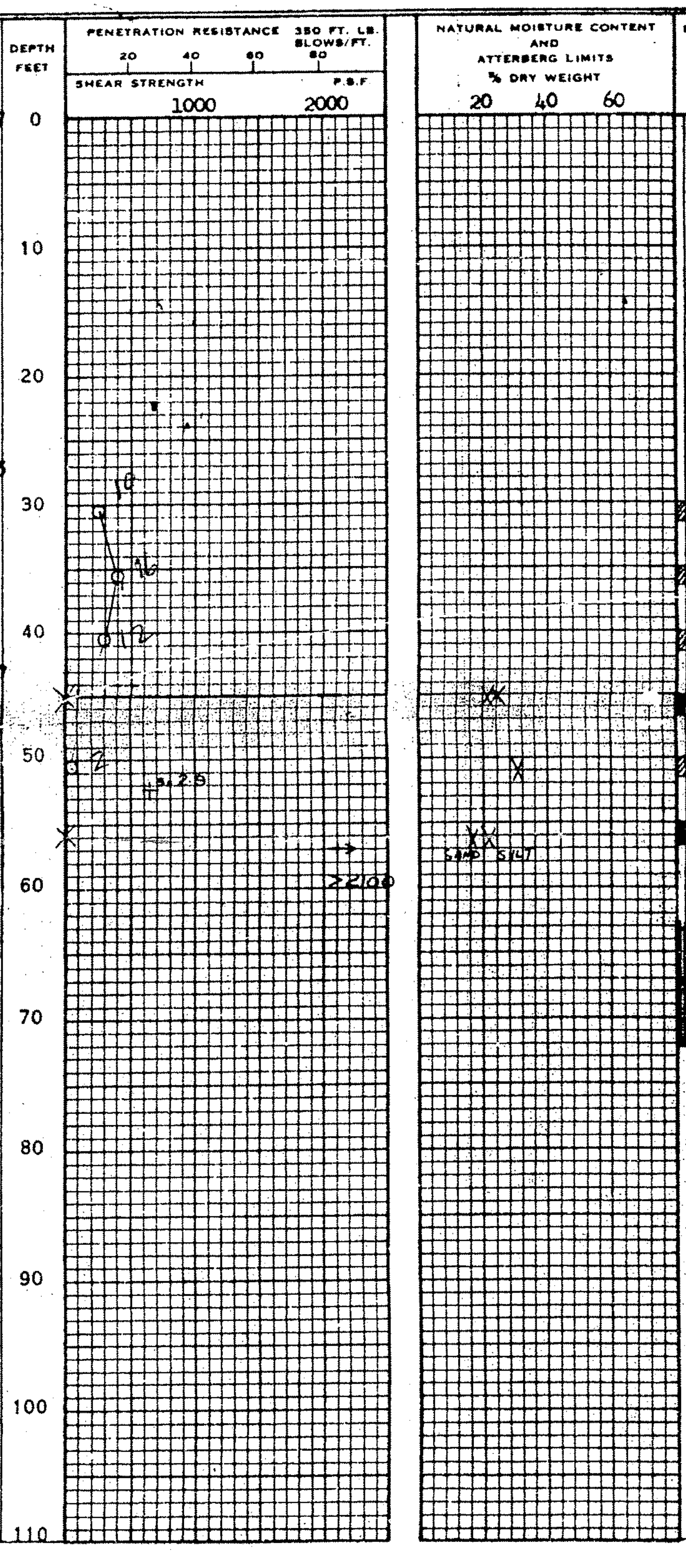
BOREHOLE No. 67-4
PROJECT Proposed Gull Lake Crossing,
LOCATION Gravenhurst By-Pass, Hwy. No. 11 (400)
HOLE LOCATION Sta. 103+08; 30'L
HOLE ELEVATION 811.74 ft.
DATUM G.B.M. DCCXIII
Publication 19 - Gravenhurst

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —x—x—x—
2" DIA. CONE ————
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊖
VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}
ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT ⊢—
SAMPLE TYPE
2" O.D. SPLIT TUBE ▨
2" I.D. SHELBY TUBE ■ L = Levere
3" O.D. SHELBY TUBE ▩

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
		811.7	0		
	WATER				
│ ¦ │	SAND- compact, fine, slightly silty, few medium and coarse sizes throughout, brown to about 34 ft.depth then grey.	784.5		1 2 3	
╱ ╱	CLAY and SILT LAYERS- firm, grey, wet, numerous fine sand seams throughout, predominantly silt, wet.	768.7		4 L 5	126
o o	-predominantly ~~sandy~~ silt and fine sand below about 55 ft.depth, -numerous cobbles and small boulders below about 59 ft.depth.	756.7		6 L	127
	BEDROCK- banded paragneiss with pink granitic injections, sound.	748.7		100% Recovery	
	End of Borehole	739.5			

Notes: 1) Hole cased to 63 ft.depth with flush jointed casing and advanced by conventional wash boring methods. Bedrock core recovered below 63 ft. depth in AXT size by conventional diamond drilling methods.



WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 5
PROJECT No. J3485

BOREHOLE No. 67-5
PROJECT Proposed Gull Lake Crossing,
LOCATION Gravenhurst By-pass, Hwy. 11 (400)
HOLE LOCATION Sta. 102+53; 40'R
HOLE ELEVATION 811.74 feet
DATUM G.B.M. DCCXIII
Publication 19 - Gravenhurst

LEGEND

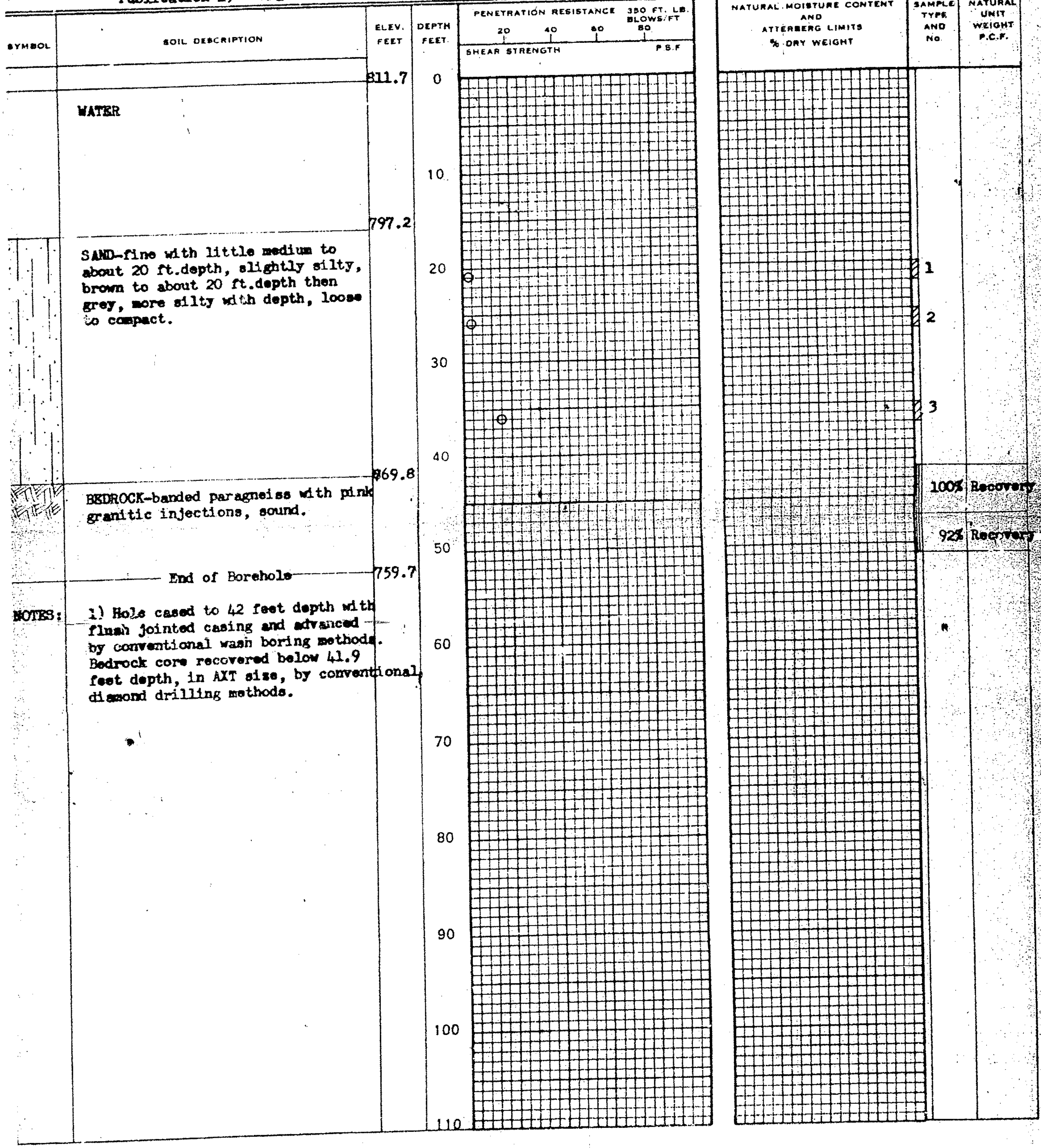
PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —x—x—x—x—
2" DIA. CONE ————

SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT ⊢—

SAMPLE TYPE
2" O.D. SPLIT TUBE ▨
2" I.D. SHELBY TUBE ■
3" O.D. SHELBY TUBE ▧



SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
		811.7	0		
	WATER				
		797.2			
	SAND-fine with little medium to about 20 ft.depth, slightly silty, brown to about 20 ft.depth then grey, more silty with depth, loose to compact.			1 2 3	
		769.8			
	BEDROCK-banded paragneiss with pink granitic injections, sound.			100% Recovery 92% Recovery	
	End of Borehole	759.7			

Depth scale: 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110

NOTES: 1) Hole cased to 42 feet depth with flush jointed casing and advanced by conventional wash boring methods. Bedrock core recovered below 41.9 feet depth, in AXT size, by conventional diamond drilling methods.

WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 6
PROJECT No. J3485

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —+—+—+—
2" DIA. CONE ————

SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) $+^{s}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

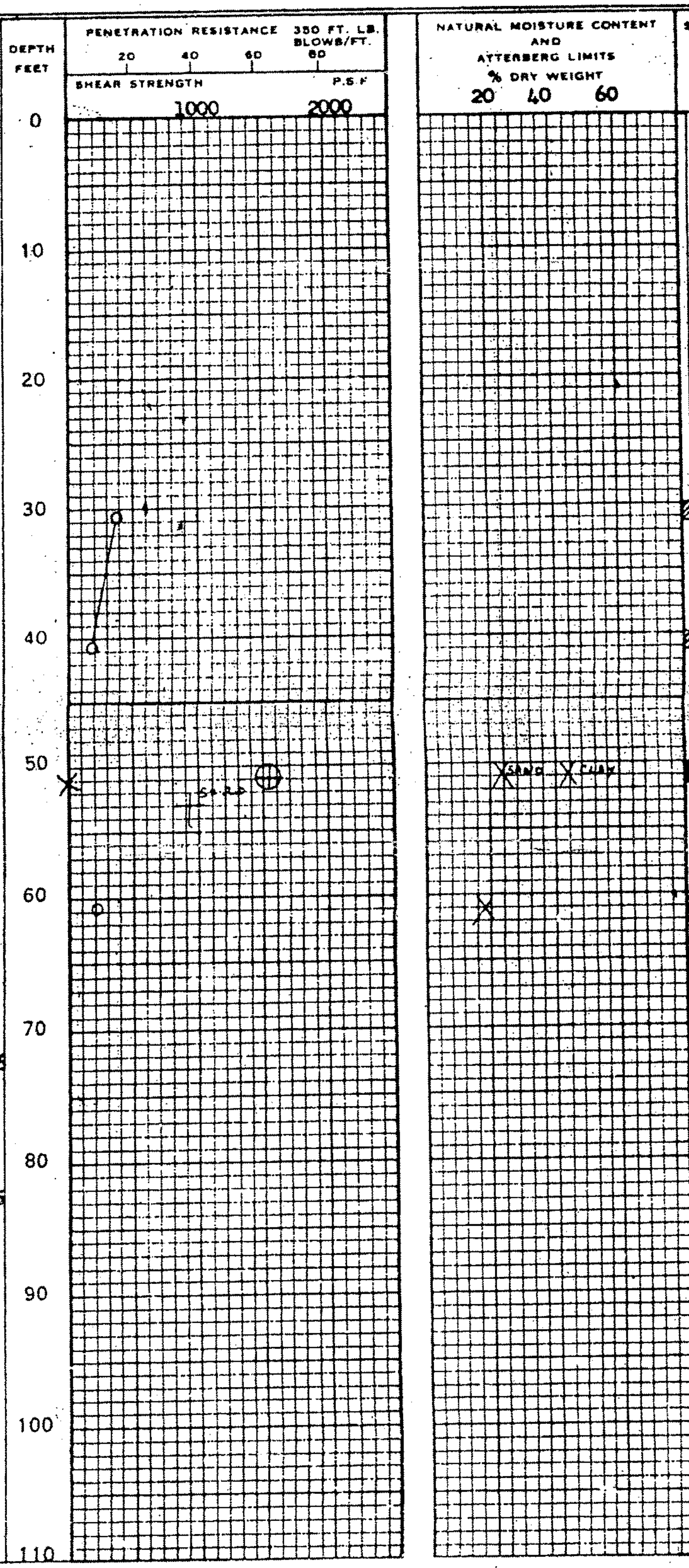
ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT ⊢—

SAMPLE TYPE
2" O.D. SPLIT TUBE ▨
2" I.D. SHELBY TUBE ■ L = Layered
3" O.D. SHELBY TUBE ▩

BOREHOLE No. 67-6
PROJECT Proposed Gull Lake Crossing,
LOCATION Gravenhurst By-Pass, Hwy. No. 11 (400)
HOLE LOCATION Sta. 102+62; 72'L
HOLE ELEVATION 811.74 ft.
DATUM G.B.M. DCCXIII
Publication 19 - Gravenhurst

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
		811.7		
	WATER	788.3		
	SAND- fine, loose to compact, slightly silty, brown to about 28 ft.depth, then grey, scattered medium to coarse gravel size.	766.7	1 2	
	SILT and CLAY LAYERS- firm to stiff, grey, predominantly silt, wet. -few fine sand seams below about 50 ft.depth.	751.7	3 L	117
	SILT- grey, firm, layered, few sand and clay layers, -numerous gravel sizes below about 62 ft.depth, -few small boulders below about 66 ft. depth.	739.6	4	
	BEDROCK- banded paragneiss with pink granitic injections, sound.	729.5	100% Recovery	
	End of Borehole			

Notes: 1) Hole cased to 73 ft.depth with flush jointed casing and advanced by conventional wash boring methods. Bedrock core recovered below 72.1 ft. depth, in AXT size, by conventional diamond drilling methods.



WILLIAM TROW ASSOCIATES LTD.

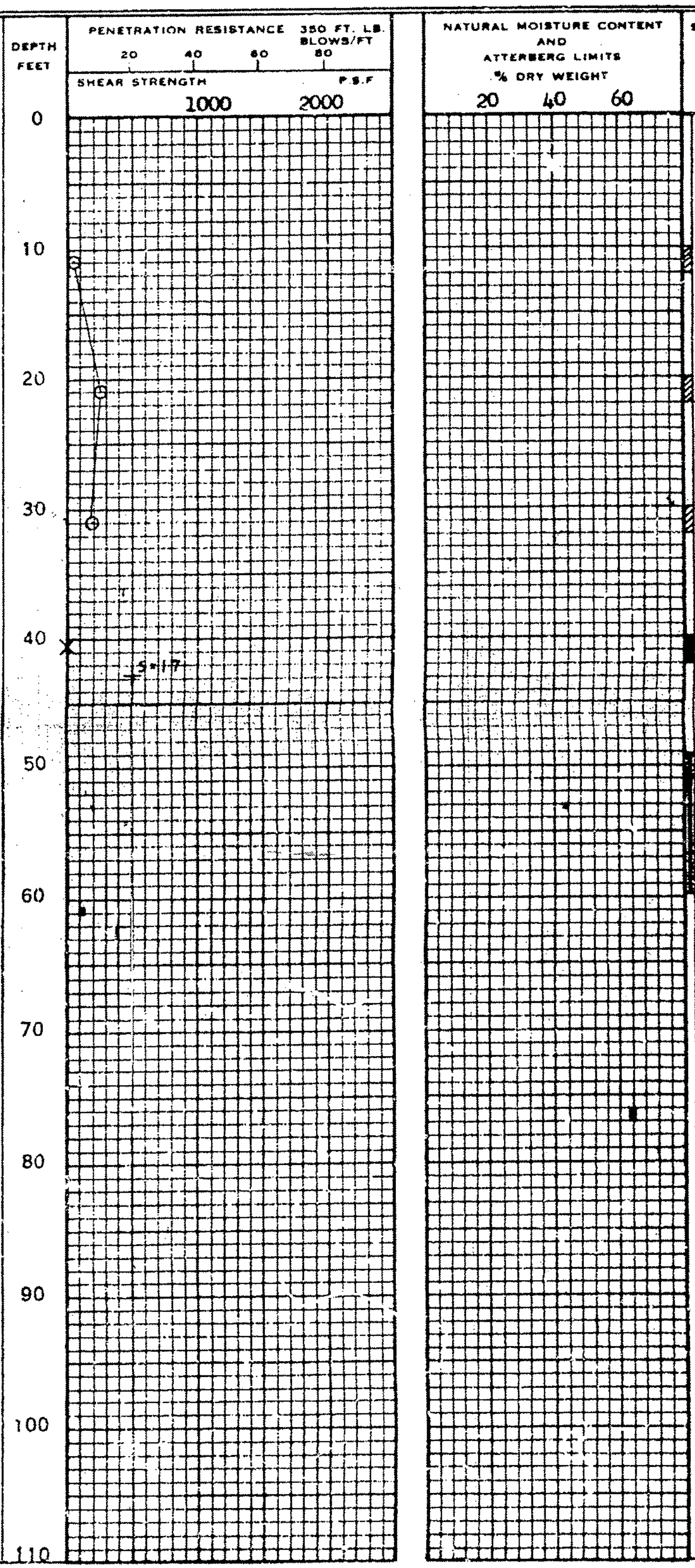
SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 7
PROJECT NO. J3483

BOREHOLE NO. 67-7
PROJECT Proposed Gull Lake Crossing,
LOCATION Gravenhurst By-Pass, Hwy. No. 11 (400)
HOLE LOCATION Sta. 102+10; 70'R
HOLE ELEVATION 811.74 ft.
DATUM G.B.M. DCCXIII
Publication 19 - Gravenhurst

LEGEND

PENETRATION RESISTANCE	
2" O.D. SPLIT TUBE	—o—o—o—
2" I.D. SHELBY TUBE	—x—x—x—
2" DIA. CONE	———
SHEAR STRENGTH	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	⊕
UNCONFINED COMPRESSION	⊗
VANE TEST AND SENSITIVITY (S)	$+^{S}$
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X^{LI}
ATTERBERG LIMITS	
LIQUID LIMIT	—o
PLASTIC LIMIT	⊢—
SAMPLE TYPE	
2" O.D. SPLIT TUBE	▨
2" I.D. SHELBY TUBE	■ L = Levered
3" O.D. SHELBY TUBE	■



SYMBOL	SOIL DESCRIPTION	ELEV. FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
		811.7		
	WATER	808.0		
	SAND- fine and medium, slightly silty, loose, brown to about 8 ft. depth then grey, few silt layers and an occasional thin clay seam throughout.	776.7	1 2 3	
	SILT and CLAY LAYERS- numerous fine sand seams throughout, predominantly silt, soft to firm, grey, wet, -numerous gravel sizes below about 45 ft.depth.	762.6	4 L	
	BEDROCK- banded paragneiss with pink granitic injections, relatively sound to sound.		54.4% Recovery 80.4% Recovery 100% Recovery	
	End of Bore'ole	751.8		

Notes: 1)Hole cased to 50 ft.depth with flush jointed casing and advanced by conventional wash boring methods. Bedrock core recovered below 49.1 ft.depth in AXT size by conventional diamond drilling methods.

WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

DRAWING No. 8
PROJECT No. 13495

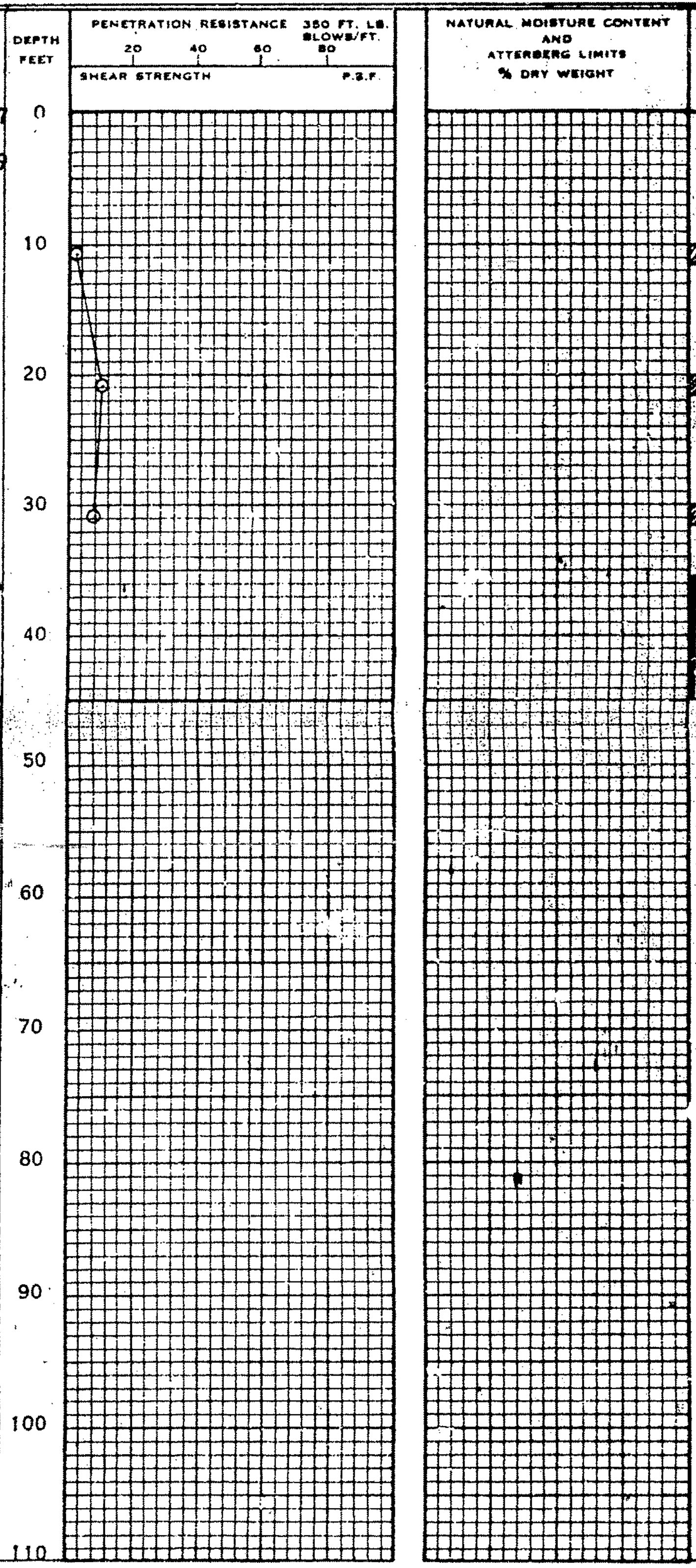
LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —x—x—x—x—
2" DIA. CONE ————
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}
ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT ⊢—
SAMPLE TYPE
2" O.D. SPLIT TUBE ▮
2" I.D. SHELBY TUBE ▮
3" O.D. SHELBY TUBE ▮

BOREHOLE No. 67-8
PROJECT Proposed Gull Lake Crossing,
LOCATION Gravenhurst By-Pass, Hwy. No. 11 (400)
HOLE LOCATION Sta. 102+12; 30'L
HOLE ELEVATION 811.74 ft.
DATUM G.B.M. DCCXIII
Publication 19 - Gravenhurst

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
		811.7		
	WATER			
	SAND- fine with a little medium, slightly silty, loose, brown to about 7 ft.depth then grey, few thin silt seams throughout. -small boulder below about 34 ft. depth.	807.9	1 2 3	
	BEDROCK- banded paragneiss with pink granitic injections,sound.	776.4	100% Recovery	
	End of Borehole	766.7		



Notes: 1)Hole cased to 35.3 ft.depth with flush jointed casing and advanced by conventional wash boring methods. Bedrock core recovered below 35.3 ft.depth in AXT size by conventional diamond drilling methods.

APPENDIX D

NON-STANDARD SPECIAL PROVISIONS

MASS CONCRETE – Item No.

Special Provision

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the North and South abutment footings.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904.

Basis of Payment

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

DOWELS Into Rock – Item No.

Special Provision

Scope of Work

Work under this item is for the placement and field testing of dowels into rock.

Materials and Installation

Dowels into rock shall be constructed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS 1440 (dowel bars conforming to CSA Standard CSAG30.18, Grade 400).

Where dowels are to be placed in rock, holes shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete (or at least 25 MPa at 28 days).

If the hole contains water, the contractor shall remove the water otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D 3689-90 and ASTM D 114381 (Re-approved 1994). Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

The following table summarizes the number of rock dowels where performance testing shall be carried out to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Bridge	**Foundation**	**Number of Dowels for Performance Testing**
Gull Lake Narrows SBL Bridge Widening	North Abutment	2
	South Abutment	2

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25

DOWELS Into Rock – Item No.

Special Provision

Cycle-Step	3-1	3-2	3-3	3-4	3-5
% Design Load	50	75	100	110	25

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, 3 additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-tensioning Institute (1985) as follows:

The dowels are acceptable if the total elastic movement is greater than 80% of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50% of the bond length.

Basis of Payment

Payment at the Contract Price for the above tender items shall include full compensation for all labour, equipment and material to do work.

ROCK POINTS - Item No.

Non-Standard Special Provision

Scope

As part of the work under the above tender item, the Contractor shall supply Titus “Rock Injector Design” Pile Points on HP 310 x 110 Piles. Piles will be driven to bedrock.

References

OPSS 906 – Structural Steel
SP903S01

Materials

The pile points shall be of the following:

Product	Manufacturer
HPP-R-12	Titus Steel Company Ltd. 6767 Invader Cr. Mississauga, ON Tel (905) 564-2446

(Or approved equivalent which includes Oslo Points as per OPSD 3000.201)

Basis of Payment

Payment at the Contract Price for the above tender items shall be full compensation for all labour, equipment and material to do the work.

CONTROLLED BLASTING, TRIM BLASTING and VIBRATION MONITORING at Foundation Locations and Permanent Rock Cuts – Item No.

Special Provision

Scope of Work

Work under this item is for the complete removal of rock using appropriate controlled drilling and blasting techniques at locations indicated in the contract and disposal of rock material. This includes trim blasting and all rock removal required at the proposed foundation abutment locations, which are located directly beside the existing abutment foundation locations.

Construction

The use of explosives shall follow the general specifications outlined in the latest version of OPSS 120.

Controlled Blasting: Drilling equipment shall consist of the following:

> A hydraulic track drill or equivalent capable of drilling the required controlled blasting holes accurately and uniformly across the top of a rock cut, or other suitable equipment, given the site conditions.

Trim blasting shall be performed at the proposed abutment locations. The Contractor should submit the trim blast design to the Contract Administrator according the requirements provided in the next section.

Removal shall be carried out in such a manner to minimize disturbance to any surrounding rock / structures beyond the excavation limits.

All material resulting from the operation shall be managed in accordance with OPSS 180 specified elsewhere in the contract.

All costs associated with the management of materials are deemed to be included in the contract unit price.

Trial blasting will be required for all proposed production and wall control blast procedures.

Monitoring and Reporting

Ground and air vibration monitoring is required during the blasting operations. Ground vibration levels should be limited to the maximum peak particle velocity values provided in Table 1 in

OPSS 120 for adjacent services, bridges and buildings (i.e. 50 mm/s for frequencies greater than 40 Hz).

The Contractor shall submit the following information to the Contract Administrator at least 3 weeks in advance of rock excavation.

- Blast Contractor: contractor must be fully qualified, experienced and capable of working at heights with approved Ministry of Labour safety full arrest devices. A statement of experience is required;

- An outline of the requirements, procedure, and extent of the pre-blast survey required;

- Proposal prepared by blast contractor or blast consultant detailing the blast methodology, including drill hole patterns, hole size and depths, size of blasts, explosive and initiation product details, as well as all blast control procedures. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signalling and site clearing procedures, as well as procedures to deal with debris clean-up; and

- Details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings. As a minimum, vibration monitoring should be provided at the following locations:

    - SBL bridge, south abutment stem/foundation (on the east/median side closest to the blasting operation);
    - NBL bridge, south abutment stem/foundation (on the east/median side closest to the blasting operation).

Instrumentation for monitoring ground and air vibration effects from the blasting should be set up in accordance with the International Society of Explosives Engineers field practice guidelines (1999).

At all locations where structures (existing and proposed) are located adjacent to or within rock cuts, the new or existing rock cut faces and/or structure founding surfaces should be inspected by an independent rock engineering specialist and provisions made for rock bolting/dowelling, if necessary.

A minimum of 80 percent half barrels (drill hole traces) visible on the cut face after scaling is required.

Measurement of Payment

The measurement for payment shall be by Plan Quantity, as may be revised by Adjusted Plan Quantity of the lineal vertical metre of drilling required to trim blast.

Basis of Payment

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

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ROCKFILL/COBBLES/BOULDERS DURING EXCAVATION, DRILLING, PILE INSTALLATION, ETC. - Item No.

Special Provision

The overburden soils at the abutment and approach embankment locations contain rockfill with cobble and boulder sized pieces.

The fill soils at the pier location contained obstructions (cobbles/boulders) between about Elevation 242 m and 239 m. The sandy silt to sand soil directly above the bedrock at the pier location also contained cobbles and boulders.

The water-bearing sandy soils will be susceptible to cave-in, sloughing and boiling.

Appropriate equipment and procedures will be required to penetrate/remove rockfill/cobbles/boulders that are encountered during excavation, augering/drilling, pile driving and/or sheet pile installation, etc.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION