



June 29, 2011

FOUNDATION INVESTIGATION AND DESIGN REPORT

**VARIABLE MESSAGE SIGN #18
HIGHWAY 11 SOUTHBOUND
APPROXIMATELY 12.6 KM SOUTH OF HIGHWAY 17
NORTH BAY, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5122-06-00**

Submitted to:

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REPORT



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PART A

FOUNDATION INVESTIGATION REPORT

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HIGHWAY 11 SOUTHBOUND

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GWP 5122-06-00



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by IBI Group (IBI) on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for a variable message sign (VMS #18) on Highway 11 southbound near North Bay, Ontario. The general location of the site is shown on the Key Plan on Drawing 1.

The terms of reference for the scope of work were outlined in Golder's Proposal P0-1191-0006, dated February 5, 2010 that formed part of IBI's agreement (Number 5009-E-0018) for this project. The work was carried out in accordance with Golder's Quality Control Plan for this project dated August 23, 2010. The site plan was provided to Golder by Trow Geomatics Inc. (Trow) on behalf of IBI in December 2010.

We understand that the proposed variable message sign will be an overhead truss extending over the southbound lane (SBL) embankment supported by a spread footing on the west side of the SBL roadway embankment and a caisson foundation on the east side of the embankment.

2.0 SITE DESCRIPTION

The site of the proposed VMS #18 is located on the SBL of Highway 11 approximately 13 km south of the intersection with Highway 17 at about Station 10+413 in the Township of East Ferris, south of North Bay, Ontario. This section of Highway 11 consists of a divided four-lane highway with two southbound lanes and two northbound lanes with fully paved shoulders, and 1 m wide and 1.5 m wide granular rounding to the embankment west and east side slopes, respectively. Drainage ditches parallel the highway on both sides of the SBL. The ground surface at the proposed structure location is about Elevation 232 m.

The topography of the site generally consists of low-lying areas separated by bedrock outcrops. A bedrock outcrop is located on the west side of the SBL ditch. Based on the existing topography, site drainage appears to be from south to north. The land use well beyond the highway right-of-way in the proposed sign location is generally residential. The ditch on the east side was noted to be lined with blast rock fill. Two photographs of the site are presented on Figure 1.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation work for the VMS #18 structure was carried out on October 28 and October 29, 2010 and on May 31, 2011, at which time two sampled boreholes, numbered BH-VMS#18W and BH-VMS#18E, one auger probe (AP-W) and two Dynamic Cone Penetration Tests (DCPT-W and DCPT-E), were drilled at the locations shown on Drawing 1. Borehole BH-VMS#18W was drilled on the west shoulder of the SBL, approximately 2.5 m from the edge of pavement. Probe AP-W and DCPT-W were located about 2.5 m north and 2.5 m south, respectively, of Borehole BH-VMS#18W. Borehole BH-VMS#18E was drilled on the east shoulder, approximately 1.3 m from the edge of the pavement and DCPT-E was advanced about 8.3 m east of Borehole BH-VMS#18E. Golder returned to site on March 11, 2011 to observe and confirm that bedrock is exposed on the west side of the existing SBL and on May 10, 2011, confirmed refusal at a depth of about 0.2 m depth by exposing bedrock at the centre of the SBL west ditch by hand shovel.

The foundation investigation was carried out using a truck-mounted CME-55 drill rig supplied and operated by Landcore Drilling of Chelmsford, Ontario. Boreholes BH-VMS#18W and BH-VMS#18E were advanced to depths of 6.9 m (including 3.4 m of bedrock coring) and 10.8 m below the existing ground surface (including



3.0 m of bedrock coring on May 31, 2011), respectively, using 108 mm inside diameter hollow stem augers, NW casing and NQ coring equipment. Probe AP-W, DCPT-W and DCPT-E were advanced to auger or DCPT refusal at depths of 1.3 m, 1.1 m and 2.9 m, respectively, below the existing ground surface. Soil samples were obtained, where possible, within the sampled boreholes at intervals of depth ranging from 0.75 m to 1.5 m, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586-08). The boreholes and auger probe were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 (as amended by O. Reg. 372).

The fieldwork was supervised throughout by a member of Golder's technical staff, who located the boreholes, auger probe and DCPT, arranged for the clearance of underground services and for traffic protection, supervised the drilling, sampling and in situ testing operations, logged the boreholes and drillhole, 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 Golder's Sudbury geotechnical laboratory, where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM standards, as appropriate. Classification testing (water contents, Atterberg limits and grain size distributions) was carried out on selected soil samples. An Unconfined Compressive Strength (UCS) test was carried out on a sample of the bedrock core.

The boreholes and DCPT were located in the field by Golder based upon the staked position by IBI. The as-drilled borehole and DCPT locations and the ground surface elevations (referenced to Geodetic datum) were subsequently surveyed by Trow and forwarded to Golder. The borehole locations are depicted on Drawing 1 and the borehole and DCPT coordinates and ground surface elevations are provided below.

Borehole Number	MTM NAD83 Zone 17 Northing (m)	MTM NAD83 Zone 17 Easting (m)	Ground Surface Elevation (m)	Drilled Depth (m)
BH-VMS#18W	5119523.1	316308.5	232.5	6.9
BH-VMS#18E	5119524.4	316320.0	232.1	10.8
AP-W	5119525.6	316308.6	232.4	1.3
DCPT-W	5119520.6	316308.6	232.5	1.1
DCPT-E	5119524.4	315328.0	230.0	2.9

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geology

Based on terrain mapping by the Ontario Geological Survey¹, the subsurface soils in the vicinity of the site consist organic terrain and outcropping bedrock, bordering areas with glaciolacustrine deposits comprised primarily of silts and sands. The bedrock in the vicinity of the site is characterized in Ontario Department of Mines and Northern Affairs² maps as either mafic intrusive of the Paleozoic eon or felsic plutonic consisting of gneiss of the Precambrian eon.

¹ Ontario Geological Survey, Ministry of Northern Development and Mines, and Northeast Science and Information Section, Ministry of Natural Resources 2005. Digital Northern Ontario Engineering Geology Terrain Study (NOEGTS); Ontario Geological Survey, Miscellaneous Release--Data 160.

² Ontario Department of Mines and Northern Affairs, North Bay Area, Map 2216.



4.2 Site Stratigraphy

Detailed descriptions of the subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected samples, are given on the attached Record of Borehole and Drillhole sheets in Appendix A. The stratigraphic boundaries shown on the Record of Borehole and Drillhole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs. These boundaries, therefore, represent transitions between material types rather than exact planes of geological change. Further, subsurface conditions will vary beyond the borehole locations.

In summary, the subsoil conditions in the west borehole consist of the existing pavement structure underlain by blast rock fill and bedrock. In the east borehole, the subsoil conditions consist of the existing pavement structure underlain by blast rock fill, in turn underlain by deposits of clayey silt to silty clay and silty sand. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt

A 0.1 m thick layer of asphalt was encountered from ground surface in Borehole BH-VMS#18W.

4.2.2 Fill

Underlying the asphalt or at ground surface, two distinct layers of fill were encountered at the two borehole locations, comprised of a 0.3 m thick layer of moist, brown to grey sand and gravel to sandy gravel containing trace to some silt underlain by a 0.4 m and 0.9 m thick layer of moist, brown sand to silty sand containing trace to some gravel. The sand to silty sand fill was encountered at Elevation 232.1 m and 231.8 m.

An SPT 'N'-value measured in the silty sand fill is 18 blows per 0.3 m of penetration, indicating a compact relative density.

Grain size distribution tests were performed on four samples of the granular fill and the test results are presented on Figure B-1.

The natural water content measured on three samples of the sand fill to sand and gravel fill range from 3 percent to 6 percent. The natural water content measured on one sample of the silty sand fill is 18 percent.

A layer of rock fill was encountered below the granular fill. The rock fill surface was encountered at Elevation 231.7 m and Elevation 230.9 m and the layer is 2.7 m and 2.8 m thick at Boreholes BH-VMS#18W and BH-VMS#18E, respectively. NQ coring was used to advance the boreholes through the rock fill layer and the core recoveries within the rock fill are 23 percent and 21 percent.

Auger probe and DCPT refusal was encountered in the rock fill at depths of 1.3 m and 1.1 m, respectively.

4.2.3 Clayey Silt to Silty Clay

A deposit of wet, brown to grey, clayey silt to silty clay, some sand and trace gravel, was encountered below the rock fill layer in Borehole BH-VMS#18E. The clayey silt to silty clay deposit was encountered at Elevation 228.1 m and is 2.2 m thick.



Two SPT 'N'-values measured in the clayey silt to silty clay are 7 blows and 11 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.

An Atterberg limits test was carried out on one sample of the cohesive stratum and test results are presented on Figure B-2. The liquid limit is 35 percent, the plastic limit is 15 percent and the plasticity index is 20 percent, which indicate that the deposit is classified as clayey silt to silty clay of low to medium plasticity. A grain size distribution test was carried out on one sample of the deposit and the results are presented on Figure B-3.

The natural water content measured on one sample of the clayey silt to silty clay deposit is 26 percent.

4.2.4 Silty Sand

A deposit of wet, brown, silty sand, some clay and trace gravel, was encountered below the clayey silt to silty clay stratum in Borehole BH-VMS#18E. The silty sand deposit was encountered at Elevation 225.9 m and is 1.6 m thick. Borehole BH-VMS#18E was terminated within this deposit due to casing refusal.

An SPT 'N'-value measured in the silty sand is 39 blows per 0.3 m of penetration, indicating a dense relative density.

The natural water content measured on the sample of the silty sand deposit is 21 percent.

An Atterberg limits test on the sample of silty sand indicates that this material is non-plastic. The results of a grain size distribution test on the sample recovered are presented on Figure B-4.

4.2.5 Bedrock/Refusal

In Boreholes BH-VMS#18W and BH-VMS#18E, bedrock was encountered at a depth of 3.5 m and 7.8 m below ground surface, respectively, corresponding to Elevations 229.0 m and 224.3 m, respectively and cored for a depth of 3.4 m and 3.0 m, respectively. Based on a review of the bedrock core samples, the bedrock generally consists of a coarse grained, slightly weathered, pinkish grey Felsic Gneiss.

Rock Quality Designation (RQD) values measured on the recovered bedrock core samples range from 55 percent to 100 percent, indicating the rock is of fair to excellent quality in accordance with Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006)³.

An Unconfined Compressive Strength (UCS) test was carried out on a representative sample of the rock core from each borehole and measured UCS of 151 MPa and 102 MPa indicating that the bedrock is very strong (R5 Grade) in accordance with Table 3.5 of the CFEM (2006).

Refusal to further cone penetration was encountered in DCPT-E at a depth of 2.9 m below ground surface, corresponding to Elevation 227.2 m; the DCPT rods were observed to be sliding to the west likely on bedrock or boulder in proximity to the bedrock surface.

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



During Golder's site visit on March 11, 2011, exposed bedrock was observed on the west side of the SBL west ditch. Further, excavation by hand shovel was attempted in the centre of the ditch and the hand shovel could not penetrate through the existing ground surface; however, on May 8, 2011, we re-visited the site and confirmed refusal on bedrock at a depth of 0.2 m.

4.2.6 Groundwater Conditions

Details of the groundwater conditions and water levels observed at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. In Borehole BH-VMS#18W, the groundwater level was observed in the open borehole at a depth of 3.7 m below the existing ground surface upon completion of drilling, corresponding to Elevation 228.8 m. In Borehole BH-VMS#18E, the groundwater level was observed in the casing at a depth of 2.9 m below the existing ground surface upon completion of drilling, corresponding to Elevation 229.2 m. It should be noted that these water levels do not represent the stabilized water level and that the groundwater elevation will fluctuate seasonally depending on precipitation and local soil and fill material permeability and should be expected to rise during wet periods of the year.

5.0 CLOSURE

The fieldwork for this project was carried out by Mr. Ed Savard from our Sudbury office under the coordination of Mr. David Muldowney, EIT. This report was prepared by Mr. David Muldowney, EIT, and the technical aspects were reviewed by Mr. André Bom, P.Eng., and Mr. Jorge M.A. Costa, P.Eng., a Principal with Golder. Mr. Costa, also a Designated MTO Contact for Golder, conducted a quality control review of the report.



Report Signature Page

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PART B

**FOUNDATION DESIGN REPORT
VARIABLE MESSAGE SIGN #18
HIGHWAY 11 SOUTHBOUND
APPROXIMATELY 12.6 KM SOUTH OF HIGHWAY 17
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed variable message sign (VMS #18). The recommendations are based on interpretation of the factual data obtained from the boreholes, auger probe and DCPT advanced during the subsurface investigation at this site and from site observations. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess feasible foundation design alternatives and to design the proposed sign foundation. As such, where comments are made on construction, they are provided only in order to highlight those aspects which could affect the planning 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.

6.2 Sign Foundation

We understand that the proposed overhead truss sign will be supported on foundations located on both the west and east sides of the Highway 11 SBL, at about Station 10+413 facing the southbound traffic. We understand that the location of the originally proposed west and east sign support foundations (4.5 m from the west and east edges of pavement) have been revised to 6.5 m and 9.5 m from the west and east edges of pavement, respectively.

Borehole BH-VMS#18W, which was advanced at Station 10+412 approximately 2 m to the east of the centre of the originally proposed west foundation (i.e. approximately 2.5 m from the edge of the pavement), encountered 0.8 m of granular fill underlain by 2.7 m of rock fill extending to the bedrock surface at a depth of 3.5 m below the existing ground surface. The AP and DCPT advanced 2.5 m to the north and 2.5 m to the south of Borehole BH-VMS#18W, on the shoulder of Highway 11, encountered refusal within the rock fill at depths of 1.3 m and 1.1 m, respectively, below ground surface. Borehole BH-VMS#18E, which was advanced at Station 10+413 approximately 3.2 m to the west of the centre of the originally proposed east foundation (i.e. approximately 1.3 m from the edge of the pavement), encountered 1.2 m of granular fill underlain by 2.8 m of rock fill, a clayey silt to silty clay stratum extending to a depth of 6.2 m and silty sand to a depth of 7.8 m underlain by bedrock. DCPT-E, advanced about 8.3 m east of Borehole BH-VMS#18E in the median, encountered refusal at a depth of 2.9 m below existing ground surface and the rods were noted to be sliding to the west indicative of a likely sloping refusal surface. The unstabilized groundwater levels in Boreholes BH-VMS#18W and BH-VMS#18E upon completion of drilling are 3.7 m and 2.9 m, respectively, below ground surface, corresponding to Elevation 228.8 m and 229.2 m.

Overhead sign supports are typically designed with a standard caisson foundation in accordance with the requirements in MTO's *Sign Support Manual* (2007). Alternatively, the sign can be supported on a spread footing.

Based on discussions with IBI and the structural designer, given the existing subsurface conditions at the location of the proposed sign support foundations, the preferred foundation alternative for the overhead sign is a spread footing founded on bedrock on the west side of the SBL and a caisson socketted into bedrock on the east side of the SBL (i.e. median). Table 1 (attached) summarizes the advantages, disadvantages, relative costs and risks/consequences of the foundation alternatives. Recommendations for the two foundation options are provided in Sections 6.2.1 and 6.2.2.



It should be noted that the presently proposed locations of the sign foundations will be about 4.0 m west of Borehole BH-VMS#18W on the west side of the SBL and about 8.2 m east of BH-VMS#18E on the east side of the SBL.

6.2.1 Caisson Foundation

Caisson foundations for overhead sign supports are typically designed in accordance with the requirements in MTO's *Sign Support Manual* for standard sign sizes. However, a site-specific design is required at this site given the existing subsurface conditions and the size of the sign compared with the standard size specified in the sign manual.

It is recommended that the east foundation be constructed using conventional caissons with a diameter of 0.6 m to 0.9 m. Because the west foundation will be a spread footing founded on bedrock, in order to minimize differential settlement between the west and east foundations, we recommend that the caisson for the east foundation be socketted into bedrock. We understand that IBI's structural engineer requires that the caisson be socketted a minimum of 2 m into bedrock.

To facilitate caisson installation and to provide granular backfill around the caisson to improve lateral resistance, we recommend that the relatively loose material in the upper 1.2 m of the overburden at the DCPT-E location, as inferred from the driving records (i.e. 'N'-values of 5 blows or less per 0.3 m of penetration) and the existing rock fill that may be present at the caisson location be removed and replaced with an inverted cone of MTO's Special Provision (SP) SP110S13 (Aggregates) Granular 'B' Type II. The new granular fill should be placed in maximum 300 mm thick lifts and compacted to not less than 95 percent of the standard Proctor maximum dry density (SPMDD) of the material. Below the groundwater level, the Granular 'B' Type II may be placed in maximum 0.5 m thick loose lifts nominally compacted with the backhoe/excavator bucket (i.e. 90 percent of the SPMDD). The inverted cone of granular fill should be placed such that the bottom of the granular fill extends at least one caisson diameter around the caisson. An open excavation is likely feasible for excavating and backfilling adjacent to the existing embankment. Further discussion regarding excavations is provided in Section 6.3.

Based on the subsurface conditions encountered at the proposed location of the east foundation (i.e. firm to stiff cohesive soils underlain by dense cohesionless soils and relatively shallow groundwater level), the installation of the caisson should be carried out with a temporary steel liner. Further, full balanced head conditions should be maintained at all times during construction to reduce the potential for base heave in the bottom of the caisson and ground loss. If the liner is generally dry following augering/excavation, cast-in-place concrete may be used; however, tremie concrete will be required if dewatering inside the liner is not possible.

The stratigraphy encountered in the Borehole BH-VMS#18E and the design parameters applicable for the design of the originally proposed east caisson are given in Table 2. Because the caisson foundation has been relocated to the toe of the SBL embankment, about 8.2 m east of Borehole BH-VMS#18E, site-specific soil design parameters cannot be provided at the proposed caisson location. However, based on discussion with IBI's structural engineer, provided that the upper 1.2 m of relatively loose material is sub-excavated and replaced with Granular 'B' Type II, as recommended above, then the minimum design parameters values specified in MTO's *Sign Support Manual* for caissons is applicable to the east foundation.



For cohesionless soils, the unfactored passive lateral earth pressure, P_p (kPa), distributed along the depth of the caisson foundation, may be calculated using the following equations:

$$P_p = K_p \gamma d_w \text{ above the groundwater table; and}$$

$$P_p = K_p \gamma d_w + K_p \gamma' (d - d_w) \text{ below the groundwater table}$$

where K_p is the passive earth pressure coefficient;

γ is the bulk unit weight of the soil (kN/m³);

γ' is the effective unit weight of the soil below the groundwater level (kN/m³);

d is the depth below the ground surface (m); and

d_w is the depth to the groundwater level (m).

For cohesive soils, P_p may be calculated using the following equation:

$$P_p = \sigma_z + 2 c_u$$

where σ_z is the total vertical stress (kPa);

c_u is the undrained shear strength (kPa).

As shown on the depth of frost penetration isopleths for Northern Ontario in Ontario Provincial Standard Drawings (OPSD) 3090.100 (Foundation Frost Depths), the depth of frost penetration for the North Bay area is approximately 1.9 m. The pile cap for the caisson should be provided with a minimum of 1.9 m of conventional soil cover for frost protection.

In the design of the foundations, the passive resistance within the upper 1.9 m below ground surface should be neglected to account for frost action. In addition, for foundation design, full passive resistance will be mobilized only where the ground surface in front of and behind the caissons is level. The K_p values are provided in Table 2. Where sloping ground is present adjacent to the caisson foundation or where the foundation will be installed at the crest of an embankment, the K_p values used in the calculation of the passive resistance should be adjusted to account for the presence of the sloping ground. Based on a review of the proposed final grading shown on the Contract Drawings provided by IBI for the 90 percent complete design, the sloping ground condition does not apply at this site.

6.2.2 Spread Footings

As noted above, we recommend that the proposed west foundation consist of a spread footing on bedrock. The advantage of founding the footing on bedrock is the greater geotechnical resistances achieved for the footing and the elimination of differential settlement between the west and east footings. However, the disadvantage of constructing the west footing on bedrock is the potential additional costs associated with bedrock excavation or the need for mass concrete to achieve a level surface.

Consideration could also be given to founding both the west and east footings on a structural fill pad upon removal of the existing rock fill. However, the excavations and placement of the structural fill will likely extend below the groundwater level, depending on the season of construction, for the east footing. The structural fill will likely be placed uncompacted or nominally compacted below the water level. The lower geotechnical resistance



associated with nominally compacted structural fill will result in a much larger footing size requiring a much larger excavation, increasing the impact to the existing embankment unless the footing is shifted to the east towards the median centreline.

We do not recommend founding the west footing on bedrock and the east footing on a structural fill pad over the native cohesive soil due to potential for differential settlement occurring between the two spread footings on each side of the embankment.

Spread footings constructed on the existing rock fill are not recommended. The existing rock fill was likely “dumped” below the groundwater level, depending on the water level at the time of construction, following removal of organics and/or soft material that may have been present at the site at the time of highway construction. Based on the encountered depth of rock fill, there was likely little to no control of the rock fill placement at the time of construction and loss of fines from the rock fill could be expected as a result of degradation of point-to-point contact, weathering and continued rearrangement of the rock fill from adjacent traffic loadings and freeze-thaw cycles.

Founded on Bedrock (West Side)

We recommend that the west footing be founded on the bedrock surface. Bedrock is exposed to the west of the SBL ditch and slopes down to the east. At Borehole BH#VMS-18W advanced within the paved shoulder, the bedrock was encountered at a depth of 3.5 m below existing ground surface, corresponding to Elevation 229.0 m.

Due to the sloping nature of the bedrock at this location and based on the centre of the west sign support being positioned about 6.5 m west of the west edge-of-pavement, as shown on cross-section drawings provided by IBI, variation in the bedrock surface elevation should be anticipated and excavation of the bedrock may be required in order to provide a level surface for the footing subgrade. Bedrock excavation within the footing footprint (assumed to be 1.5 m wide by 4 m long) can likely be achieved by the use of hoe-ramming or similar excavation techniques. The bedrock at the founding depth will be of good quality but nevertheless the founding surface should be properly prepared (i.e. removing loose shattered rock fragments). A Non Standard Special Provision (NSSP) for levelling the bedrock surface should be included in the Contract Documents; an example NSSP is provided in Appendix C. Depending on the final bedrock surface slope, doweling of the footing to bedrock would increase the sliding resistance; however, we understand from IBI's structural engineer that doweling will not be required at the west footing.

For spread footings founded directly on the bedrock or on mass concrete over bedrock, frost susceptibility is not an issue.

Founded on Structural Fill (Both Sides)

As an alternative to founding the west footing on bedrock, consideration could be given to founding the footing on a structural fill pad following removal of the existing rock fill. Due to the shallow depth to bedrock and sloping bedrock surface at the proposed footing location, the structural fill thickness below the footing will be variable. Based on a minimum soil cover of 1.9 m for frost protection, the footing could be founded at Elevation 230.5 m, depending on final ground surface elevation. Alternatively, insulation could be used for frost protection to raise the footing subgrade elevation.



At the east footing location, sub-excavation of the existing rock fill would be required to a depth of about 4.0 m below existing grade to expose the native cohesive soils. The footing could be founded at about Elevation 230.0 m, assuming 1.9 m of soil cover for frost protection and depending on the final grade at the sign location. Based on the depth to the bottom of the rock fill layer, the thickness of the structural fill pad below the footing would be about 2.0 m. Given the proximity of the east footing to the existing edge of pavement and the anticipated size of footing and the extent of structural pad below the footing, the excavation will extend up the slope towards the centreline of the two southbound lanes, requiring lane closures of the respective impacted lane during excavation, unless the sign foundations can be relocated further from the roadway shoulder. We understand from IBI that impacting and reinstating the existing travelled lanes is to be avoided due to maintenance issues following construction associated with sub-excavation and backfilling within the existing roadway.

Below the groundwater level, the structural pad should be constructed of SP110S13 (Aggregates) Granular 'B' Type II placed in maximum 0.5 m thick loose lifts nominally compacted with the backhoe/excavator bucket (i.e. 90 percent of the SPMDD). Above the groundwater level, the structural pad should be constructed of SP110S13 (Aggregates) Granular 'A' or Granular 'B' Type II placed in maximum 150 mm thick lifts and compacted to not less than 100 percent of the standard Proctor maximum dry density (SPMDD) of the material. The top surface of the structural pads should extend at least 1 m beyond the plan limits of the footing and the side slopes of the pad should be no steeper than 1H:1V.

The spread footing constructed on the structural fill should be provided with a minimum of 1.9 m of soil cover below final ground surface for frost protection.

Inspection and approval of the foundation area by the Quality Verification Engineer prior to structural pad and footing construction should be required in accordance with OPSS 902 (Excavating and Backfilling), to ensure that all rock fill has been removed from the foundation areas, review and approve the placement and compaction of the structural pad below the footing and that the foundation base has been properly prepared for the placement of concrete.

6.2.3 Geotechnical Resistance

For the west footing bearing directly on the bedrock surface, a factored geotechnical resistance at Ultimate Limit States (ULS) of 1 MPa may be used for design. Serviceability Limit States (SLS) for 25 mm settlement conditions do not apply for footings founded on bedrock or on mass concrete.

Spread footings constructed on a structural pad consisting of Granular 'B' Type II placed in wet conditions may be designed based on a factored geotechnical axial resistance of 150 kPa at ULS for a footing rectangular in shape up to 4 m long by 1.5 m wide. For the same spread footing dimension indicated above, the geotechnical axial resistance value of 75 kPa for SLS (for 25 mm settlement) design may be assumed.

Spread footings of the dimensions noted above constructed on a Granular 'A' or Granular 'B' Type II pad, placed and compacted to greater than 100 percent of the SPMDD in-the-dry, may be designed based on a factored geotechnical axial resistance at ULS of 700 kPa and a geotechnical axial resistance at SLS of 350 kPa.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing dimensions or founding depth differs from those given above.



The geotechnical resistances provided above are given for loadings 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 Clauses 6.7.4 and C6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC, 2006)* and the related commentary.

6.2.4 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the cast-in-place concrete footings and the prepared bedrock surface should be calculated in accordance with Section 6.7.5 of the *CHBDC* using a coefficient of friction, $\tan \phi'$, equal to 0.70. For a cast-in-place footing constructed on a granular pad, a coefficient of friction, $\tan \phi'$, equal to 0.58 should be used in assessing lateral forces/sliding resistance between the concrete footings and the granular pad and for a precast concrete footing constructed on a granular pad, a coefficient of friction, $\tan \delta$, equal to 0.45 should be used.

For footings on bedrock, the sliding/lateral resistance between the concrete footing/mass concrete and the bedrock may be supplemented by dowelling/anchoring into the bedrock, if necessary. As discussed in Section 6.2.2, we understand from IBI's structural engineer that because the bedrock surface will be levelled, dowels will not be required.

6.3 Construction Considerations

Care must be taken by the Contractor during the excavation operations adjacent to the highway to minimize impact to the existing roadway. Open cut excavations of short duration through the granular fill and rock fill are considered feasible for the proposed foundation. Excavations for the proposed foundations should be carried out in accordance with the latest Occupational Health and Safety Act (OHSA) for Construction Projects. When referencing OHSA, the existing granular fill and rock fill should be considered as "Type 3 Soil" and temporary excavation side slopes should be made no steeper than 1 horizontal (H) to 1 vertical (V) from the existing ground surface. Depending on the size of the rock fragments, localized flatter side slopes may form naturally.

If side slopes flatter than 1H:1V are required during construction, provision for protection of the existing pavement structure will be required in accordance with MTO's Ontario Provincial Standard Specification (OPSS) 539 (Temporary Protection Systems), designed to meet Performance Level 2. However, installing shoring in/through rock fill may prove extremely difficult and standard sheet-pile or soldier pile and lagging walls will not be appropriate for this site. A trench box may be the most practical alternative support system. However, since trench boxes will not be in intimate contact with the surrounding rock fill, the use of trench boxes will not provide significant benefits in terms of reducing the horizontal extent of the excavation, but would provide protection from sloughing of the rock fill. Lane closures will likely be required for the areas of the sign foundation excavations depending on the size of the excavations.

During construction, stockpiles should be placed well away from the edge of the excavation, and their height should be controlled so they do not surcharge the sides of the excavation and/or overall slope. Generally, for this site, the distance between the crest of the excavation and the toe of the stockpile should be greater than the diameter of the base of the stockpile.



The excavation around and above the spread footing may be backfilled using an approved granular material such as SP110S13 (Aggregates) Granular 'B' (Type I or Type II) placed in 0.3 mm thick loose lifts and uniformly compacted to greater than 95 percent of the SPMDD.

The existing sand and gravel and sandy gravel fill meets the specifications for a Granular 'B' Type I material and may be re-used for the roadway shoulder. The existing fill should not be used above and around the spread footings due to the potential for migration of the material into the rock fill. The existing silty sand fill does not meet the gradation requirements for a Granular 'A' or Granular 'B' (Type I or Type II) material and, as such, should not be re-used for the roadway shoulder.

The SBL pavement structure (i.e. asphalt, base and subbase) should be re-instated to match the existing pavement structure.

The final grade surrounding the sign should be sloped to promote surface water drainage and pavement structure drainage away from the pavement and sign, to the adjacent ditch, and surfaced with a 300 mm thick layer of R-10 Rip-Rap meeting the requirements in OPSS 1004 (Aggregates – Miscellaneous), to reduce the potential for erosion of the slope locally.

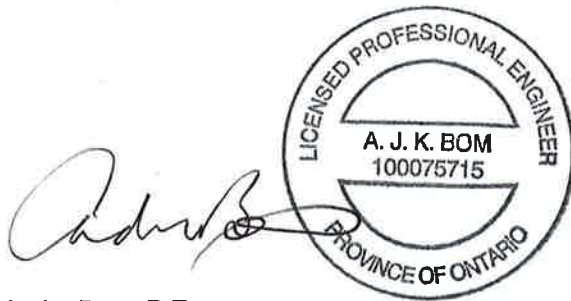
7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng., and the technical aspects were reviewed by Mr. Jorge M.A. Costa, P.Eng., a Principal with Golder. Mr. Costa, also a Designated MTO Contact for Golder, conducted a quality control review of the report.



Report Signature Page

GOLDER ASSOCIATES LTD.



Andre Bom, P.Eng.
Geotechnical Engineer



Jorge M.A. Costa, P.Eng.
Designated MTO Contact, Principal

AB/JMAC/lb

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N:\Active\2010\1190 Sudbury\1191\10-1191-0006 IBI Various VMS\Reporting\VMS 18 North Bay\Final Report\10-1191-0006 RPT 11June29 IBI VMS 18 North Bay FIDR.Docx



REFERENCES

- Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, Fourth Edition.
- Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-06, 2006. CSA Special Publication, S6.1-06. Canadian Standard Association.
- Ministry of Transportation, Ontario, 2007. Sign Support Manual. Policy, Planning & Standards Division, Engineering Standards Branch, Bridge Office
- Occupational Health and Safety Act and Regulation for Construction Projects, January 2006.

STANDARDS

ASTM International

- | | |
|---------------|---------------------------------------------------------------------------------------------|
| ASTM D1586-08 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
|---------------|---------------------------------------------------------------------------------------------|

Ministry of Transportation Ontario Special Provisions

- | | |
|-----------|----------------------------------------------------------------------------------------------|
| SP 110S13 | Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |
|-----------|----------------------------------------------------------------------------------------------|

Ontario Provincial Standard Drawings

- | | |
|---------------|----------------------------------------------|
| OPSD 3090.100 | Foundation Frost Depths for Northern Ontario |
|---------------|----------------------------------------------|

Ontario Provincial Standard Specifications

- | | |
|-----------|------------------------------------------------------------------------|
| OPSS 539 | Construction Specification for Temporary Protection Systems |
| OPSS 902 | Construction Specification for Excavating and Backfilling – Structures |
| OPSS 1004 | Material Specification for Aggregates – Miscellaneous |

Ontario Water Resources Act

- | | |
|---------------------------|-------------------------------------|
| Ontario Regulation 372/07 | Amendment to Ontario Regulation 903 |
| Ontario Regulation 903/90 | Wells |



Table 1: Evaluation of Foundation Alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footing Founded on Bedrock Surface (West Footing)	1	<ul style="list-style-type: none"> ■ Conventional construction. ■ Better known condition/quality subgrade and high geotechnical resistances available. ■ Frost cover protection not required for footings on bedrock. 	<ul style="list-style-type: none"> ■ Requires excavation of bedrock to achieve level surface or levelling concrete pad and/or anchors/dowels. 	<ul style="list-style-type: none"> ■ Lower overall cost compared to caissons and footings on granular pad. ■ Additional costs associated with bedrock excavation and/or levelling concrete pad and/or dowels. 	<ul style="list-style-type: none"> ■ Risk of difficulties associated with bedrock excavation in the footing area. ■ Risk of larger volumes of mass concrete for levelling the footing subgrade and/or requirement for anchors/dowels for lateral sliding resistance due to sloping bedrock.
Caisson Socketted into Bedrock (East Footing)	2	<ul style="list-style-type: none"> ■ High bearing resistances available within the very strong bedrock. ■ Standard design. 	<ul style="list-style-type: none"> ■ Requires removal of loose soil at ground surface to 1.2 m depth and replacement with structural granular pad (Granular 'B' Type II), partially placed and uncompacted below ground water. ■ May require a levelling pad (fill or excavation of slope) to accommodate the caisson drilling equipment. ■ Permanent steel liner likely required due to encountered groundwater depth; full balanced head and tremie concrete may be required. 	<ul style="list-style-type: none"> ■ Typically \$1,000/m for a 1 m diameter caisson in soil with a liner; much higher costs per metre for socketting into bedrock (about \$10,000 per m). ■ Additional costs associated with rock fill removal and replacement with Granular 'B' Type II. 	<ul style="list-style-type: none"> ■ Lower risk of rock fill excavations negatively impacting existing lanes of traffic compared with spread footing alternative.



FOUNDATION REPORT, VMS #18 HIGHWAY 11 SOUTHBOUND, GWP 5122-06-00

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on Granular Pad (East and West Footings)	3	<ul style="list-style-type: none">■ Conventional construction.■ West: excavation of bedrock not required.	<ul style="list-style-type: none">■ Removal of rock fill required and replaced with structural granular pad, likely placed uncompacted below ground water.■ Frost protection cover required for footing on granular pad; frost protection cover thickness to be measured perpendicular to slope, resulting in footing having to be located further into the embankment, to achieve an adequate thickness of soil/granular fill on the embankment slope.■ At least a 4 m deep excavation required on each footing location and replacement of rock fill with Granular 'B' Type II material.	<ul style="list-style-type: none">■ Costs associated with rock fill removal and replacement with Granular 'B' Type II.■ Lower excavation costs than spread footings on bedrock but cost will likely increase due to the need for remedial works.	<ul style="list-style-type: none">■ Risk of rock fill excavations negatively impacting existing lanes of traffic; both footings would require relocating further from embankment to minimize risk of impacting existing lanes.

Note: This table should be read in conjunction with the Foundation Investigation and Design Report.

Compiled by: AB
Reviewed by: JMAC



Table 2: Soil Parameters for Sign Foundation Design

Borehole Number	Stratum ¹	Elevation (m)	c_u (kPa)	ϕ' (Degrees)	K_p	γ (kN/m ³)	γ' (kN/m ³)	Design Groundwater Level Elevation (m)
BH-VMS#18E	Granular 'B' Type II (removal of Rock Fill)	Depth of Frost Penetration (1.9 m) to Elevation 228.1	-	35	3.0	21	11	229.2
	Clayey Silt to Silty Clay	228.1 to 225.9	40	-	NA	18	8	
	Silty Sand	225.9 to 224.3	-	35	3.0	20	10	

- Notes: 1. Stratum is applicable in vicinity of borehole location, does not apply at actual caisson location about 8.2 m east of borehole.
2. This table should be read in conjunction with the Foundation Investigation and Design Report.

c_u = Undrained shear strength of soil (kPa)

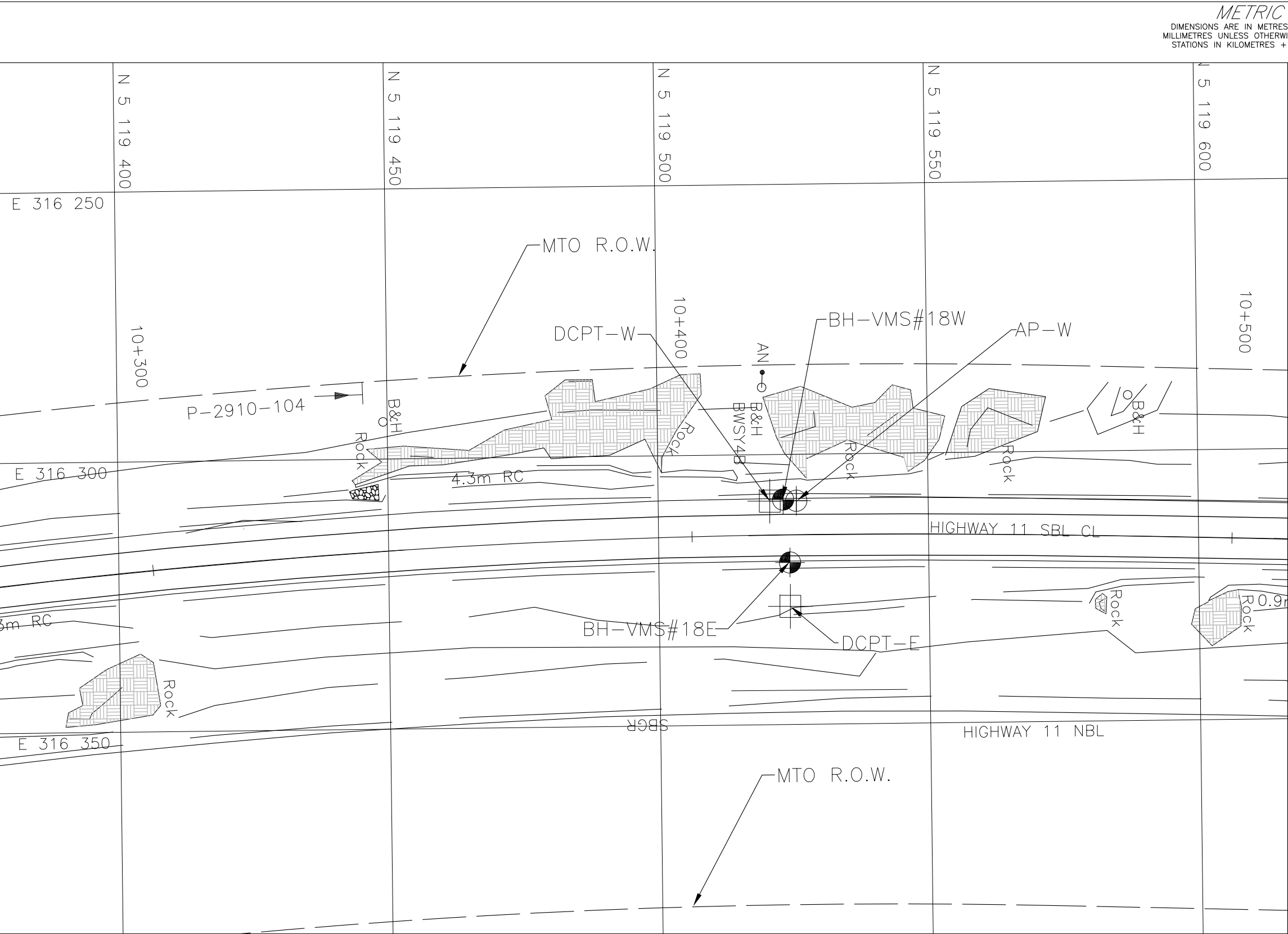
ϕ' = Effective angle of friction in soil (degrees)

K_p = Coefficient of passive earth pressure

γ = Bulk unit weight of soil (kN/m³)

γ' = Bulk unit weight of soil below the groundwater level (kN/m³)

Compiled by: AB
Reviewed by: JMAC



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

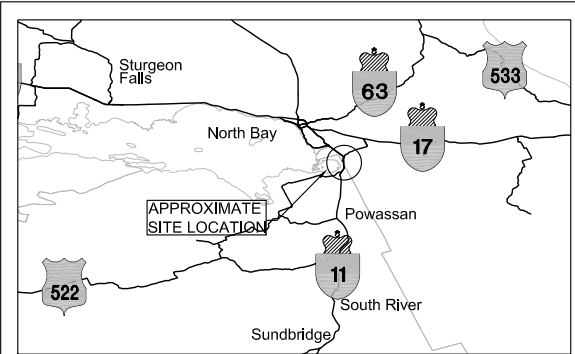
CONT No.
WP No.5122-06-00

VARIABLE MESSAGE SIGN #18
HIGHWAY 11 SOUTHBOUND, NORTH BAY
BOREHOLE, AUGER PROBE AND
DCPT LOCATIONS

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN
N.T.S

LEGEND

- BH-VMS#18: Approximate Borehole Location
- AP: Approximate Auger Probe Location
- DCPT: Approximate Dynamic Cone Penetration Test Location

No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
BH-VMS#18W	232.5	5119523.1	316308.5
BH-VMS#18E	232.1	5119524.4	316320.0
AP-W	232.4	5119525.6	316308.6
DCPT-W	232.5	5119520.6	316308.6
DCPT-E	230.0	5119524.4	316328.0

NOTES

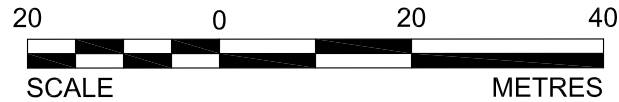
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

REFERENCE

Key plan provided in digital format by IBI, drawing file no.TM-TWG-keyplan.dwg, received December 7, 2010

Base plan provided in digital format by TROW Geomatics inc, drawing file no.NTB-01007030 HWY 11 VMS.dwg, received December 7, 2010



NO.	DATE	BY	REVISION
Geocres No.31L-143			
HWY. 11	PROJECT NO.10-1191-0006		DIST.
SUBM'D. DAM	CHKD. AB	DATE: JUNE 2011	SITE:
DRAWN: PL	CHKD.	APPD. JMAC	DWG. 1



Figure 1 – Photographs of VMS #18 Site, North Bay

Photograph 1: East of SBL, looking south (October 2010)



Photograph 2: West side of SBL, looking north (May 2011)





APPENDIX A

Record of Borehole and Drillhole



LIST OF SYMBOLS

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

1. GENERAL

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	Factor of Safety
V	volume
W	weight

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_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 $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
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 cm² 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	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



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

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

<u>Terms</u>	<u>Size*</u>
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 (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD 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 separation) 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 separation such as fractures, bedding planes and foliation planes or mechanically induced fractures 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	⊥ - Perpendicular To
FO - Foliation / Schistosity	- Parallel To
CL - Cleavage	P - Polished
SH - Shear Plane / Zone	K - Slickensided
VN - Vein	SM - Smooth
F - Fault	R - Rough
CO - Contact	ST - Stepped
J - Joint	PL - Planar
FR - Fracture	U - Undulating
MF - Mechanical Fracture	C - Curved

PROJECT		10-1191-0006		RECORD OF BOREHOLE No BH-VMS#18E				1 OF 1 METRIC										
W.P.		5122-06-00		LOCATION		N 5119524.4; E 316320.0		ORIGINATED BY										
DIST		HWY 11		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing		COMPILED BY										
DATUM		GEODETIC		DATE		October 28 and 29, 2010 and May 31, 2011		CHECKED BY										
								AB										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
232.1	GROUND SURFACE							20	40	60	80	100						
0.0	Sandy gravel, trace silt (FILL)		1	AS	-		232										68 27 (5)	
231.8	Brown Moist		2	AS	-		231											1 75 (24)
0.3	Silty sand, trace gravel (FILL)		3	SS	18		230											
230.9	Compact Brown Moist		RC	REC 21%	229													
1.2	Blast rock (FILL)				228													
	Auger refusal at 1.4 m depth. Switched to NW Casing.				227													
					226													
					225													
					224													
					223													
					222													
228.1	CLAYEY SILT to SILTY CLAY, some sand, trace gravel		4	SS	7												2 20 41 37	
4.0	Firm to stiff Brown to grey Wet		5	SS	11													
225.9	Silty SAND, some clay, trace gravel		6	SS	39												4 30 54 12	
6.2	Dense Brown Wet																	
224.3	GNEISS (BEDROCK)		1	RC	REC 100%												RQD = 94%	
7.8	Bedrock cored from 7.8 m depth to 10.8 m depth.		2	RC	REC 100%													
	For coring details see Record of Drillhole BH-VMS#18E.		3	RC	REC 100%													
221.3	END OF BOREHOLE																	
10.8	END OF BOREHOLE																	
<p>Note:</p> <p>1. Water level at a depth of 2.9 m below ground surface (Elev. 229.2 m) upon completion of drilling.</p> <p>2. Borehole was advanced to refusal on October 29, 2010. Returned to site on May 31, 2011 to core bedrock at borehole.</p>																		

SHEET 1 OF 1

DATUM: GEODETIC

DRILL RIG:

DRILLING CONTRACTOR:

CHECKED: AB

SUD-RCK 1011910006.GPJ GAL-MISS.GDT 28/06/11 DATA INPUT:

PROJECT		10-1191-0006		RECORD OF BOREHOLE No BH-VMS#18W				1 OF 1 METRIC									
W.P.		5122-06-00		LOCATION		N 5119523.1; E 316308.5		ORIGINATED BY EHS									
DIST		HWY 11		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring		COMPILED BY DAM									
DATUM		GEODETIC		DATE		October 28, 2010		CHECKED BY AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
232.5	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT		1	AS	-												46 47 (7)
232.1	Sand and gravel to sand, trace to some silt (FILL)		2	AS	-												19 72 (9)
231.7	Grey to brown Moist																
0.8	Sand, some gravel, trace to some silt (FILL)																
	Brown Moist																
	Blast rock (FILL)																
	Auger refusal at 1.1 m depth. Switched to NW casing.																
229.0			3	SS	6/0.2												
3.5	FELSIC GNEISS (BEDROCK)		1	RC	REC 100%												RQD = 55%
	For coring details see Record of Drillhole BH-VMS#18W for details.		2	RC	REC 100%												RQD = 94%
			3	RC	REC 100%												RQD = 100%
			4	RC	REC 100%												RQD = 100%
225.6	END OF BOREHOLE																
6.9	Note: 1. Water level at a depth of 3.7 m below ground surface (Elev. 228.8 m) upon completion of drilling. 2. An Auger Probe (AP) and Dynamic Cone Penetration Test (DCPT) was advanced 2.5 m north and 2.5 m south of the borehole respectively. Auger and DCPT refusal at 1.3 m and 1.1 m below ground surface, (Elev. 231.1 m and 231.4 m), respectively.																

SUD-MTO 001 1011910006.GPJ GAL-MISS.GDT 28/06/11 DATA INPUT:

SHEET 1 OF 1

DATUM: GEODETIC

DRILLING CONTRACTOR: Landcore Drilling

CHECKED: AB

SUD-RCK 1011910006.GPJ GAL-MISS.GDT 28/06/11 DATA INPUT:

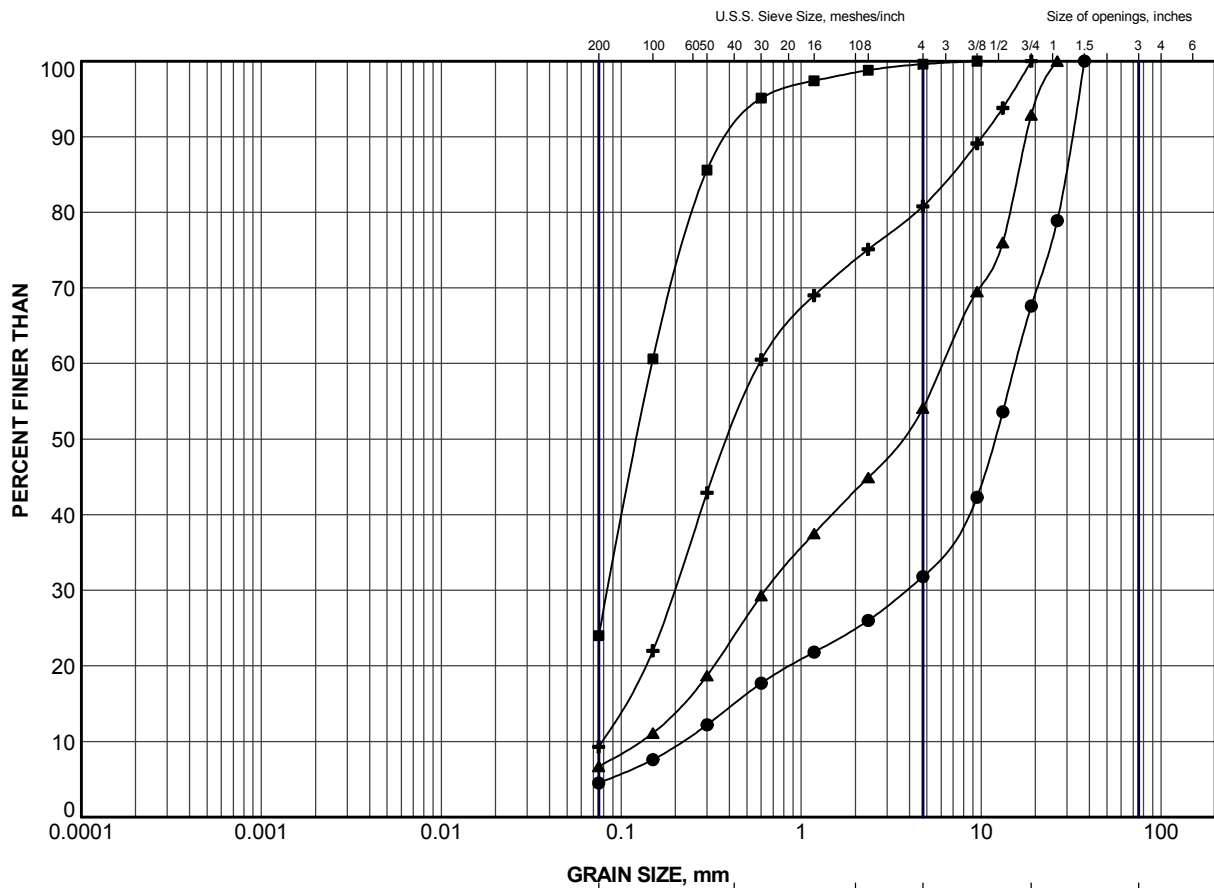


+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE



APPENDIX B

Laboratory Test Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-VMS#18E	1	231.9
■	BH-VMS#18E	3	231.1
▲	BH-VMS#18W	1	232.2
+	BH-VMS#18W	2	231.9

PROJECT

HIGHWAY 11 SOUTHBOUND VMS #18

TITLE

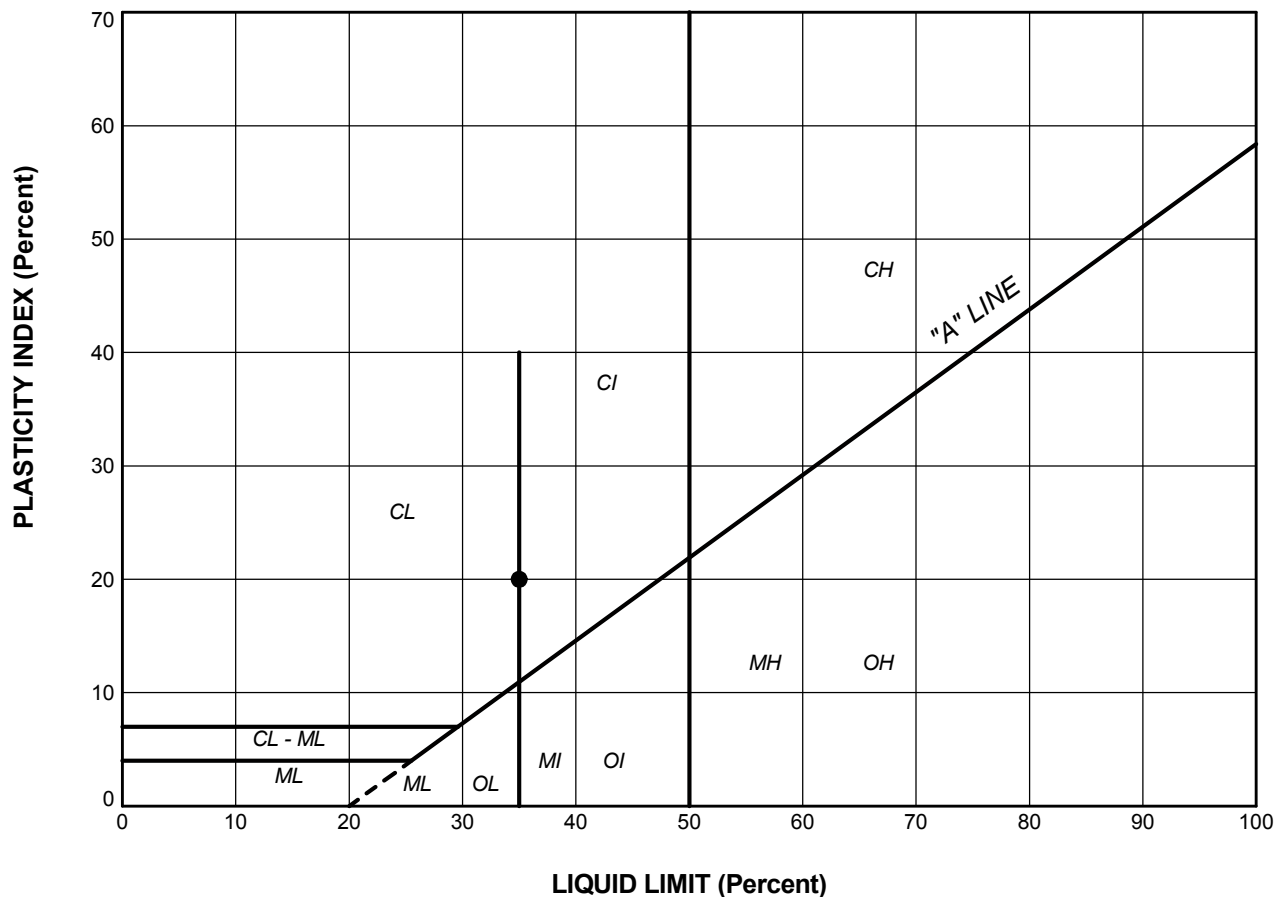
GRAIN SIZE DISTRIBUTION
SANDY GRAVEL TO SILTY SAND (FILL)



Golder Associates
SUDBURY, ONTARIO

PROJECT No.	10-1191-0006	FILE No.	1011910006.GPJ
DRAWN	JJL	Jun 2011	SCALE N/A
CHECK	AB	Jun 2011	REV.
APPR		Jun 2011	

FIGURE B-1




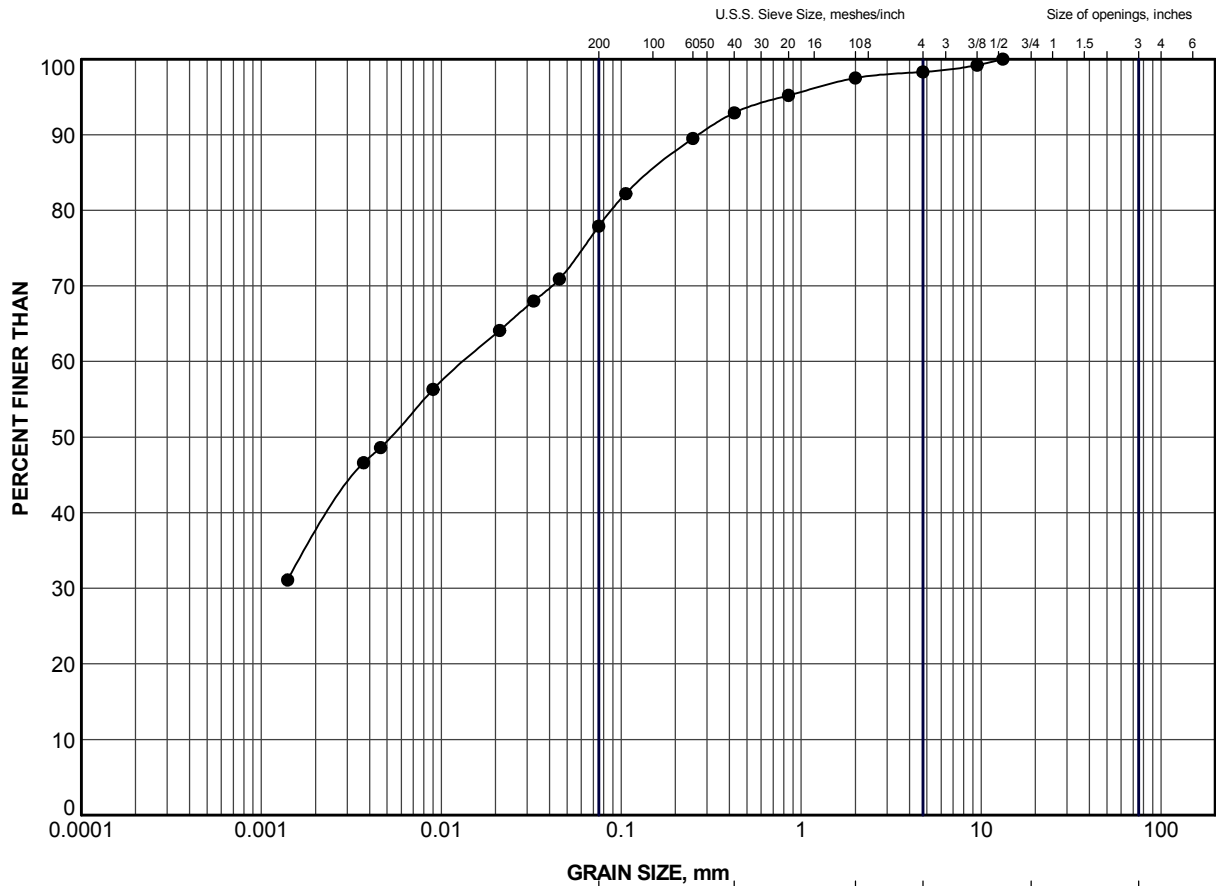
SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-VMS#18E	4	35.0	15.0	20.0


PROJECT					
HIGHWAY 11 SOUTHBOUND VMS #18					
TITLE					
PLASTICITY CHART CLAYEY SILT TO SILTY CLAY					
PROJECT No.		10-1191-0006		FILE No. 1011910006.GPJ	
DRAWN	JJL	Jun 2011	SCALE	N/A	REV.
CHECK	AB	Jun 2011			
APPR		Jun 2011			
 Golder Associates SUDBURY, ONTARIO			FIGURE B-2		

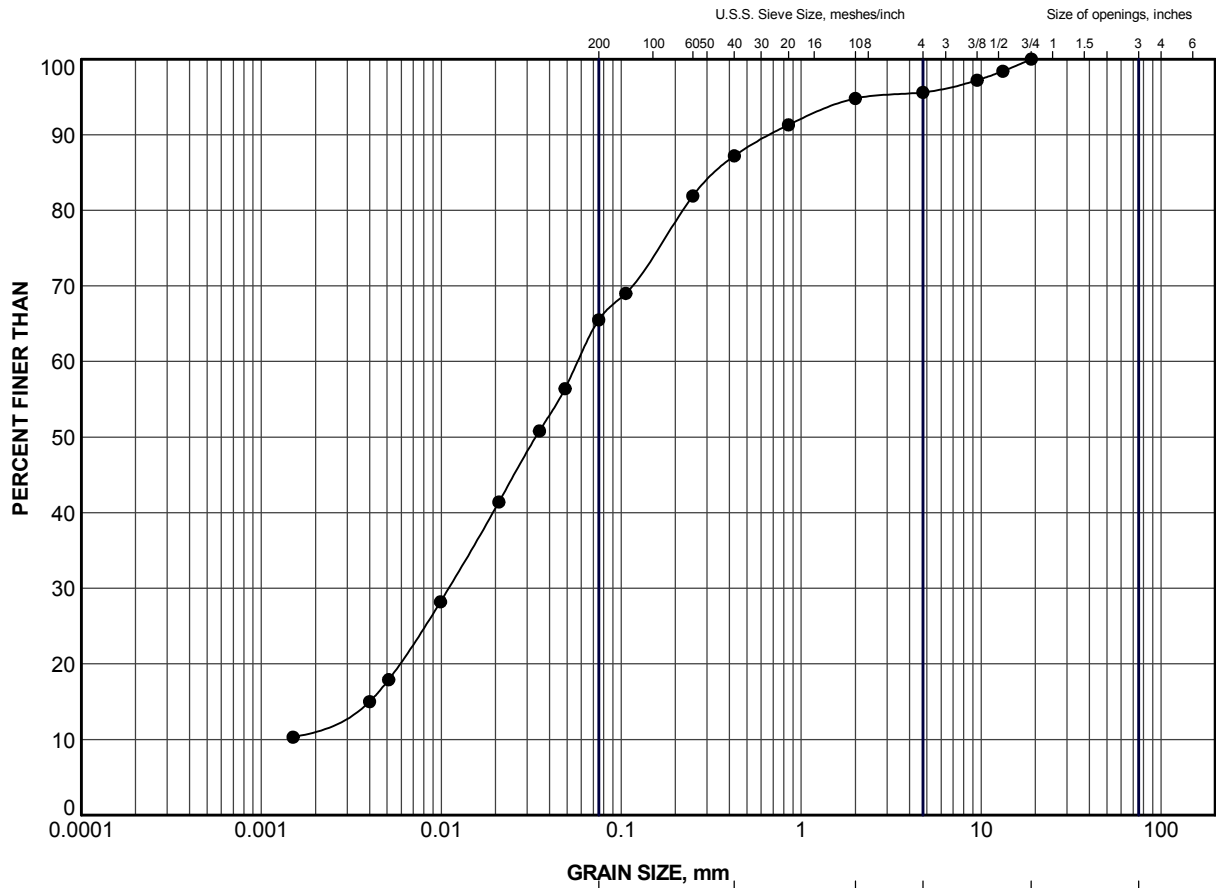


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-VMS#18E	4	227.7


PROJECT					
HIGHWAY 11 SOUTHBOUND VMS #18					
TITLE					
GRAIN SIZE DISTRIBUTION					
CLAYEY SILT TO SILTY CLAY					
PROJECT No.		10-1191-0006		FILE No. 1011910006.GPJ	
DRAWN	JJL	Jun 2011	SCALE	N/A	REV.
CHECK	AB	Jun 2011			
APPR		Jun 2011			
 Golder Associates SUDBURY, ONTARIO			FIGURE B-3		



CLAY AND SILT						Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-VMS#18E	6	225.4

PROJECT					
HIGHWAY 11 SOUTHBOUND VMS #18					
TITLE					
GRAIN SIZE DISTRIBUTION					
SILTY SAND					
PROJECT No.		10-1191-0006		FILE No. 1011910006.GPJ	
DRAWN	JJL	Jun 2011	SCALE	N/A	REV.
CHECK	AB	Jun 2011			
APPR		Jun 2011			
 Golder Associates SUDBURY, ONTARIO			FIGURE B-4		



APPENDIX C

Non-Standard Special Provisions



CAISSON

Non-Standard Special Provision

The Contactor is hereby notified that the native overburden soils at the location of the VMS #18 east foundation include cohesive material as well as cohesionless and water-bearing soils, which are susceptible to soil cave-in, sloughing and boiling. The Contractor shall ensure that appropriate construction procedures and equipment are used for construction of the caisson, including the use of a temporary steel liner and placement of the concrete by tremie methods as appropriate.

The Contractor is hereby notified that the bedrock is sloping at the VMS #18 site and the caisson at the east footing should be socketted into bedrock to the depth specified on the drawings.

END OF SECTION



LEVELLING BEDROCK SURFACE – Item No.

Non-Standard Special Provision

Scope of Work

The scope of work for the above noted tender item includes bedrock excavation at the west VMS #18 footing to provide a level founding surface for the footing.

Construction

Prior to placing concrete for the proposed west footing, the bedrock shall be levelled using hoe ram or equivalent such that the surface of the bedrock is sloping less than 10 degrees throughout the footprint of west footing. The exposed bedrock must be cleaned by removing loose debris and rock shatter. The QVE shall review the footing subgrade prior to placing concrete.

Basis of Payment

Payment at the Contract Price for the above tender item includes full compensation for all labour, equipment and material to do the required work.

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

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