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**REPORT ON**

**FOUNDATION INVESTIGATION AND DESIGN  
DETAIL DESIGN  
HIGHWAY 11 AND PINEDALE ROAD/HEWITT STREET UNDERPASS  
HIGHWAY 11 / MUSKOKA ROAD 169 INTERCHANGE  
GRAVENHURST, ONTARIO  
G.W.P 314-00-00**

Submitted to:

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GEOCRES NO: 31D00-410

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**PART A**

**FOUNDATION INVESTIGATION REPORT  
DETAIL DESIGN  
HIGHWAY 11 AND PINEDALE ROAD/HEWITT STREET UNDERPASS  
HIGHWAY 11 / MUSKOKA ROAD 169 INTERCHANGE  
GRAVENHURST, ONTARIO  
G.W.P 314-00-00**

## **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:

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

This report addresses the new Highway 11 underpass structure at Pinedale Road/Hewitt Street and the associated approach embankments. A foundation investigation has been carried out to assess the subsurface conditions at this site. The foundation investigations for the related swamp crossing, Gull Lake Bridge structure widening, 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 underpass structure at Highway 11 and Pinedale Road / Hewitt Street was provided to Golder by MRC in February 2005, and revised July 22, 2005. This report supercedes the technical memorandum sent to MRC dated March 17, 2005 included as part of the Structural Design Report.

The purpose of this investigation is to establish the subsurface conditions at the proposed structure by borehole drilling, rock coring, in-situ testing and laboratory testing on selected samples. The boreholes and probeholes 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.

## **2.0 SITE DESCRIPTION**

The site is located about 75 metres south of the existing at-grade intersection of Pinedale Road / Hewitt Street and Highway 11 in Gravenhurst, Ontario. Highway 11 in this area is presently a four-lane freeway, with northbound and southbound lanes (i.e. two lanes in each direction) separated by a grass median. The proposed underpass is located at about Station 12+555 and Station 12+610 relative to the chainage along the Highway 11 NBL and SBL centrelines, respectively.

Adjacent to the existing roadways the site consists of a rolling terrain including densely treed areas, numerous bedrock outcrops and swamp areas. The ground surface within the limits of the proposed Highway 11 Underpass and approach embankment area generally lies between about Elevation 258.5 m and 263 m, referenced to Geodetic Datum. In the area of the proposed structure, the surface of existing Highway 11 is at about Elevation 260 m and the drainage ditches that run along and between the north and south bound lanes of the highway are at an elevation of about 258.5 m. Bedrock cuts/outcrops are exposed at ground surface on the west and east sides of the proposed structure foundation areas at elevations as high as 263 m. Beyond these locations, to the immediate west and east along the proposed Pinedale/Hewitt alignment, the site consists of dense trees, vegetation and rolling bedrock outcrops.

### **3.0 INVESTIGATION PROCEDURES**

#### **3.1 Foundation Investigation**

The field work for the two-span bridge underpass investigation was carried out between November 15 and November 19, 2004 during which time a total of fourteen (14) sampled boreholes and twelve (12) probeholes were put down at the site. Twelve (12) boreholes were drilled at the proposed locations of the centre pier, east and west abutment footings and one (1) borehole was advanced at each of the proposed east and west approach embankments. The probeholes were carried out to confirm the elevation of bedrock in the vicinity of the east and west abutments. All of the boreholes and probeholes were advanced to refusal on inferred bedrock. In five (5) of the boreholes at the pier and abutment locations, bedrock coring was carried out for a minimum length of 3 m.

The field investigation was carried out using a CME 55 Bombadier drill rig supplied and operated by Walker Drilling Co. Ltd. of Barrie, Ontario. The boreholes put down with the drill rig were advanced using 108 mm solid stem augers. Soil samples were obtained, where possible, continuously or at intervals of about 0.75 m depth, using a 50 mm outer diameter (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 (i.e. inferred bedrock) which occurred at depths ranging from ground surface (i.e. bedrock outcrop) to about 2.6 m below the existing ground surface (not including rock coring). At boreholes BH04-1, BH04-4, BH04-9, BH04-10, and BH04-11, located within the footprint of the proposed foundations, the drilling was further advanced into the bedrock by coring about 3.2 m to 5.9 m. The groundwater level in the open boreholes was observed and recorded throughout the drilling operations. The probeholes were all located where bedrock was exposed (i.e. outcropping) at the ground surface.

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, organic content and grain size distribution were carried out on samples of the overburden soils. Strength testing such as point load index was carried out on specimens from the rock core.



On completion of the field work, all investigated borehole/probehole 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/probeholes 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/probehole locations were calculated based on measurements from adjacent survey control points provided by J.D. Barnes Ltd. The borehole and probehole locations are summarized in the following table and are shown on Drawings 1 and 2.

<b>Borehole Number (BH)</b>	<b>Borehole Location</b>	<b>MTM NAD83 Northing (m)</b>	<b>MTM NAD83 Easting (m)</b>	<b>Ground Surface Elevation (m)</b>
04-1	West abutment	4974462.2	316615.8	261.7
04-2	West abutment	4974426.3	316620.3	261.4
04-3	West abutment	4974465.6	316623.3	261.4
04-4	West abutment	4974461.7	316626.4	259.4
04-5	West approach	4974468.9	316616.8	261.3
04-6	Centre pier	4974435.2	316641.5	259.0
04-7	Centre pier	4974434.0	316636.7	258.6
04-8	Centre pier	4974432.7	316631.1	258.8
04-9	Centre pier	4974430.2	316632.8	259.0
04-10	Centre pier	4974437.1	316639.6	258.7
04-11	East abutment	4974405.0	316652.0	261.8
04-12	East abutment	4974405.0	316648.0	262.0
04-13	East abutment	4974406.4	316658.5	261.5
04-14	East approach	4974396.4	316657.5	262.0

<b>Probehole Number (PH)</b>	<b>Probehole Location</b>	<b>MTM NAD83 Northing (m)</b>	<b>MTM NAD83 Easting (m)</b>	<b>Ground/Bedrock Surface El. (m)</b>
1	West abutment	4974464.8	316630.0	259.9
2	West abutment	4974460.6	316620.4	260.7
3	West abutment	4974478.3	316611.5	261.9
4	West abutment	4974480.4	316627.0	261.3
5	East abutment	4974409.4	316648.8	262.3
6	East abutment	4974401.5	316639.6	261.5
7	East abutment	4974410.7	316656.1	259.9
8	East abutment	4974394.6	316651.7	262.8
9	East abutment	4974401.5	316647.9	262.5
10	East abutment	4974405.0	316646.0	262.3
11	East abutment	4974410.7	316656.1	261.6
12	West abutment	4974451.4	316616.6	261.8

### **3.2 Bedrock Mapping and Rock Hazard Assessment**

The exposed bedrock conditions at the proposed bridge abutments were assessed based on visual observations and detailed geotechnical mapping of the rock cuts and outcrops along Highway 11 in the immediate area of the proposed bridge. In addition, the rock core and geotechnical logging from the boreholes was reviewed by a rock mechanics engineer.

The detailed geotechnical field mapping of the exposed rock conditions was carried out by an experienced rock mechanics engineer 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 and B2). 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 abutments and identify any rockfall hazards for the existing rock cuts. For proposed new rock cut faces, the field data was used to identify potential failure modes which might require stabilization or remedial measures during excavation.

## **4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS**

### **4.1 Geology**

From published geologic information, the site is located mostly within the physiographic region known as the Number 11 Strip with portions of Highway 11 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 rock 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). However, 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 are typically highly deformed gneisses 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 following the text of this report. The results of the inferred bedrock surface elevations from the probeholes are given on the attached Record of Probehole sheets following the text of this report. The results from the laboratory testing are provided in Appendix A. 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 as encountered in the boreholes at the proposed two-span bridge location are shown on Drawings 1 and 2. The boreholes drilled for the bridge pier and abutments are located within the existing Highway 11 median and right-of-way, respectively. As a result, the overburden soils encountered in some of the boreholes drilled at the site (especially in the median area) consist of probable fill materials from the previous construction of Highway 11.

In general, the subsoils at the structure site consist of a surficial layer of topsoil underlain by a sand to silty sand and/or fill containing some organics, in turn, underlain by a granite gneiss bedrock. The total overburden thickness ranges from no cover (i.e. bedrock outcrops present at ground surface) to about 2.6 m below ground surface. All of the boreholes were terminated at the

inferred bedrock surface; with the exception of five (5) boreholes that were cored to depths between 3.2 m and 5.9 m into the bedrock.

At the investigated locations in the area of the proposed west and east abutments, bedrock ranged from the ground surface (i.e. as encountered at Probeholes PH-1 to PH-12, inclusively) to as much as 2.6 m below existing ground surface.

In the area of the proposed centre pier, bedrock was encountered in the boreholes at depths between 0.2 m to 0.6 m below the existing ground surface; however, the bedrock at this location was poor quality with zones of broken rock (gravel- to boulder-sized pieces) evident up to 2.1 m below ground surface (i.e. about 1.5 m below top of bedrock).

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

#### **4.2.1 Topsoil**

Topsoil was encountered at the ground surface in boreholes BH04-7, BH04-8, BH04-10 and BH04-11. The surface of the topsoil (i.e. the ground surface) ranged between Elevation 258.6 m and 261.8 m and was about 0.1 m to 0.3 m thick.

#### **4.2.2 Sand and Silty Sand (Fill)**

Sand and silty sand fill was encountered either at the ground surface or below the topsoil at the five boreholes located at the proposed center pier (Boreholes BH04-6 to BH04-10) and in Borehole BH04-4 located at the west abutment. Boreholes 04-6 to 04-8 terminated at the base of this layer on probable bedrock. The fill materials consist of variable amounts of sand, silt, gravel, clayey silt pockets, topsoil and organic matter. The elevation of the top of the sandy fill ranged from Elevation 258.4 m to 259.4 m and the thickness ranged from about 0.2 m to 0.7 m.

Standard Penetration Testing (SPT) measured 'N' values in the silty sand and sand fill typically ranging between 3 and 34 blows per 0.3 m of penetration, indicated a very loose to dense state of packing. The natural water content measured on samples of the fill ranged between 11 and 41 percent. The higher water content values may indicate the presence of organics.

#### **4.2.3 Silty Sand / Sand**

A layer of silty sand to sand was encountered at ground surface extending to bedrock in boreholes BH04-1 to BH04-3, BH04-5 and BH04-12 to BH04-14. Each of these boreholes terminated at the base of this layer on probable bedrock except BH04-1 which was advanced an additional 3.2 m into bedrock by coring. The orange to reddish brown silty sand to sand layer contained some

organics (typically decreasing organics with depth) and trace gravel. The top of the silty sand to sand layer (i.e. ground surface) ranged from Elevation 261.3 m to 262 m and the thickness ranged from about 0.4 m to 2.6 m.

Standard Penetration Testing (SPT) measured 'N' values in the silty sand to sand ranging from 2 to greater than 66 blows per 0.3 m of penetration, but typically between 2 and 25 blows per 0.3 m of penetration, indicated a very loose to compact state of packing. The higher 'N' values were typically encountered approaching or at refusal upon the underlying bedrock.

The natural water content measured on samples of the silty sand to sand layer ranged between 7 and 34 percent. The higher water contents may be attributed to the samples containing varying amounts of organic material. A laboratory organic content test on one sample of the silty sand to sand measured 6.7 percent. A single grain size distribution curve taken on a selected sample of the sand is shown on Figure A1.

#### **4.2.4 Bedrock**

Bedrock was encountered and cored in boreholes BH04-1, BH04-4, and BH04-9 to BH04-11, inclusive and was cored for depths of 3.2 m to 5.9 m. The presence of bedrock was inferred from auger or sampler refusal in boreholes BH04-2, BH04-3, BH04-5 to BH04-8 and BH04-12 to BH04-14. At the locations of probeholes PH-1 to PH-12, bedrock was present at the ground surface. The surface of the bedrock varies from ground surface (bedrock outcrops) to a depth of 2.6 m below ground surface, corresponding to elevations ranging from Elevation 258.2 m to 262.8 m.

At the east abutment location, the bedrock surface slopes from a high of about Elevation 262.3 m (PH-10) at the south side where it is outcropping to a low at about Elevation 260.4 m (BH04-13) at the northeast corner where it is about 1.1 m below ground surface.

At the west abutment location, the bedrock surface slopes from a high of about Elevation 261.8 m (PH-12) at the south side where it is outcropping to a low of about Elevation 258.7 m (BH04-4 and BH04-3) at the north side where it is up to about 2.6 m below ground surface.

Within the existing median and at the proposed pier location, the bedrock surface ranges from about 0.2 m to 0.6 m below ground surface (Elevation 258.2 m to 258.6 m). However, the bedrock at this location contained zones of broken rock with gravel- to boulder-size pieces to depths up to about 2.1 m below ground surface. Broken rock zones and gravel size pieces were not evident in the cored boreholes at the abutment locations (with the exception of BH04-4, located outside of the west abutment footprint and within the existing drainage ditch) and as such it is considered that this layer/zone in the median is related to the blasting / rock shattering that was probably carried out during construction of the existing highway. It is also possible that the

broken rock could be due to frost penetration/action as the boreholes in this area (BH04-6 to BH04-10) were located in the existing median storm water ditch.

In general, the bedrock samples are described as fresh to weathered, foliated, blackish grey and pink, fine to medium grained, strong to very strong Granite Gneiss. The bedrock samples typically contained distinct foliation planes and medium to coarse grained quartz and feldspar veins. The Rock Quality Designation (RQD) measured on the core samples typically ranged from about 60 to 100 percent, indicating a rock mass of fair to excellent quality. The Total Core Recovery was typically between 90 percent and 100 percent. However, in boreholes BH04-4, BH04-9, and BH04-10 (located at or near the elevation of the existing highway and within the storm water ditches), the RQD measured on core samples within the top 3 m of rock core ranged from about 21 to 60 percent and Total Core Recovery was between 56 and 95 percent, indicating a rock mass of very poor to fair quality.

Point load strength tests were performed on samples of the rock core. 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 3.9 MPa to 8.8 MPa with an average of about 6.1 MPa for diametral tests (i.e. testing carried out perpendicular to the core axis) and range from approximately 4.0 MPa to 6.2 MPa with an average of about 4.8 MPa for axial tests (i.e. testing carried out parallel to the core axis). It should be noted that within boreholes BH04-9 and BH04-10, the zones containing broken rock pieces (i.e. the upper 2.1 m of rock core) did not have long enough intact samples for accurate point load testing and as a result the point load tests in this region are not considered representative of the general intact rock strength. As such, the strength results from the point load tests 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. Table 1 following the text of this report presents a detailed list of all point load index testing results performed for this investigation.

<b>Borehole (Drillhole) No.</b>	<b>Average Axial Point Load Index, <math>Is_{50}</math> (MPa)</b>	<b>Average Diametral Point Load Index, <math>Is_{50}</math> (MPa)</b>
04-1	-	6.1
04-4	4.7	4.9
04-9	-	6.2
04-10	4.8	6.5
04-11	-	7.4

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

As discussed earlier, the existing rock cuts at the proposed east and west abutment locations were mapped by an experienced rock mechanics engineer. Based on the geotechnical mapping of the existing rock cuts, there are four main joints sets including the foliation. The first joint set dips steeply to the SW (dip/dip direction 80°/205°), the second dips steeply to the NW or SE (dip/dip direction 87°/332°), the third (foliation) has a shallow dip to the NE (dip/dip direction 37°/071°) and the fourth set has a shallow dip to the SW (dip/dip direction 23°/244°). Based on the visual assessment it is evident that there has been some blast damage to the rock mass from past blasting practices. In general, many areas of the rock cut faces are blocky with numerous loose or partially detached blocks on the face (refer to Figures B1 and B2 for the proposed west and east abutment locations, respectively). In some areas, the detached or partially detached blocks extend back approximately 2 m from the crest of the rock cut. Other areas consist of relatively clean joint faces running obliquely to the road axis.

#### **4.2.5 Groundwater Conditions**

In general, the samples taken in the overburden boreholes were noted to be moist. Details of the groundwater conditions and 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. The boreholes were typically dry upon completion of drilling, with the exception of the five boreholes (BH04-6 to BH04-10) advanced within the existing Highway 11 median stormwater drainage ditch where there was standing water present at some locations at the time of the investigation. Water was encountered at ground surface or within about 0.5 m below the ground surface (Elevations 258.5 m to 259.0 m) at these borehole locations. Boreholes BH04-4 and BH04-13, advanced at the lower lying north side of the west and east abutment locations also encountered water at about 0.6 m and 1.1 m below ground surface (Elevation 258.8 m and 260.4) respectively, upon completion of drilling.

The water levels may indicate perched water conditions within the existing Highway 11 stormwater drainage paths, on top of the bedrock surface. It should be noted that groundwater conditions in the area are subject to seasonal fluctuations.

### 4.3 Closure

The field technician supervising the drilling program was Mr. Suresh Bainey. This report was prepared by Ms. Shannon Palmer, EIT and Mr. Kevin J. Bentley, P.Eng., a geotechnical engineer, and reviewed by Mr. J. Paul Dittrich, P.Eng., and quality control review was provided by Mr. Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder.

#### GOLDER ASSOCIATES LTD.

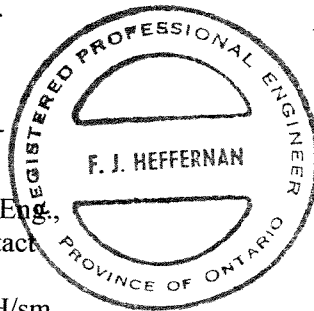


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Geotechnical Engineer

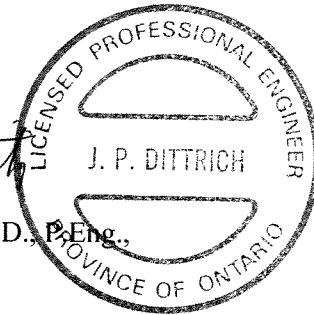


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**PART B**

**FOUNDATION DESIGN REPORT  
DETAIL DESIGN  
HIGHWAY 11 AND PINEDALE ROAD/HEWITT STREET UNDERPASS  
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 Highway 11 and Pinedale Road/Hewitt Street underpass structure. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes advanced during the subsurface investigation and from the bedrock mapping and rock hazard assessments carried out at the existing rock cuts/outcrops at the site.

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**

It is understood that a new two-span, slab-on-girder bridge structure is to be constructed over Highway 11 with abutments located on the east and west sides of the existing Highway 11 alignment, about 75 m south of the existing Pinedale Road/Hewitt Street intersection. The span lengths are to be about 30 m long and the bridge width is to be about 11.6 m. The existing grade/ground surface at the bridge site is at about Elevation 258.5 m at the pier location, ranges from approximate Elevation 258.5 m to 262 m at the west abutment and approach embankment location, and ranges from approximately Elevation 259.5 m to 264 m at the east abutment and approach embankment location. There are exposed bedrock outcrops at both abutment locations, and the proposed center pier is located within the grassy area of the existing Highway 11 median ditch.

The overburden soils at the site, where present, consist predominantly of surficial topsoil underlain by loose to dense sand and silty sand fill containing organics at the pier location and surficial very loose to very dense native sand containing organics at the abutment locations. The sandy overburden soils are typically underlain by strong to very strong granite gneiss bedrock of fair to excellent quality; with the exception at the proposed pier location where the upper 1.5 m of bedrock contains broken rock zones with gravel-sized pieces of very poor quality.

At the east abutment location, the bedrock surface slopes from a high of about Elevation 262.3 m (PH-10) at the south side where it is outcropping to a low at about Elevation 260.4 m (BH04-13) at the northeast corner where it is about 1.1 m below ground surface.

At the west abutment location, the bedrock surface slopes from a high of about Elevation 261.8 m (PH-12) at the south side where it is outcropping to a low of about Elevation 258.7 m (BH04-4 and BH04-3) at the north side where it is up to about 2.6 m below ground surface.

Within the existing median and at the proposed pier location, the bedrock surface ranges from about 0.2 m to 0.6 m below ground surface (Elevation 258.2 m to 258.6 m). However, the bedrock at this location contained zones of broken rock with gravel- to boulder-size pieces to depths up to about 2.1 m below ground surface. Broken rock zones and gravel size pieces were not evident in the cored boreholes at the abutment locations (with the exception of BH04-4, located outside of the west abutment footprint and within the existing drainage ditch) and as such it is considered that this layer/zone in the median is related to the blasting / rock shattering that was probably carried out during construction of the existing highway. It is also possible that the broken rock could be due to frost penetration/action as the boreholes in this area (BH04-6 to BH04-10) were located in the existing median storm water ditch.

## **5.2 Bridge Foundation Options**

Various alternatives for the abutment and pier foundations are considered in the sections below and a summary of these alternatives is presented in Table 2 following the text of this report. In order to eliminate the requirement for expansion joints and bearings at the ends of the bridge deck, semi-integral abutment configurations supported on shallow spread footings could be considered for the design as discussed in the next section. Due to the raised bedrock outcrops present at the abutment locations, integral abutments (in the form of steel H-piles) were not considered a practical bridge foundation option for this site. Perched abutments on compacted granular fill were also not considered due to the highly variable bedrock surface elevation and potential for differential settlements due to variable fill heights.

It is considered that spread footings founded on properly prepared bedrock for support of the bridge abutments and pier are the most feasible option from a geotechnical/foundation perspective.

## **5.3 Spread Footings**

The bridge abutments and pier may be supported on shallow spread footings founded on the properly prepared granite gneiss bedrock. The range in bedrock surface elevation as encountered in the boreholes and probeholes at the different foundation elements is summarized in the following table.

<i>Foundation Element</i>	<i>Borehole / Probehole</i>	<i>Bedrock Surface</i>		<i>Design Founding Level on “intact” Bedrock</i>	
		<i>Depth below ground surface (m)</i>	<i>Elevation (m)</i>	<i>Sub-excavation depth below bedrock surface (m)</i>	<i>Elevation (m)</i>
West Abutment	BH04-1 to BH04-4, PH-2, PH-12	0.0 to 2.6	258.7 to 261.8	-	258.7 to 261.8
Centre Pier	BH04-6 to BH04-10	0.2 to 0.6	258.2 to 258.5	1.4 to 1.7 (about 1.5)	257.0
East Abutment	BH04-11 to BH04-13, PH-9 to PH-11	0.0 to 1.1	260.4 to 262.5	-	260.4 to 262.5

### Abutment Locations

Based on the borehole results and exposed rock mapping, there is high variability in the bedrock surface elevation within the limits of each foundation element – particularly at the abutments. In addition, all loose or fractured rock encountered at the bedrock surface will need to be subexcavated and removed which may lower footing founding elevations as indicated in the table above. 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.

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

East Abutment: Elevation 262.5 m

For the east abutment, 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 Contract Documents for additional mass concrete placement to accommodate variations in the bedrock surface (an example is provided in Appendix C). The sloping bedrock may require installation of dowels between the bedrock and concrete to increase sliding resistance (see Section 5.3.2). The benefit of this option is that excavation into the strong to very strong bedrock is avoided.

The southern portion of the west abutment footprint is located directly on the partially detached rock identified during our bedrock mapping investigation (see Figure B1) and almost directly on the crest of the existing rock cut. As a result, mass concrete placed directly on the partially detached rock at the west abutment location is not considered an option.

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

West Abutment: Elevation 258.7 m

East Abutment: Elevation 260.4 m

In this case, following the removal of the overburden, excavation of the higher portions of the bedrock and/or partially detached bedrock (i.e. at the west abutment location to expose the underlying “intact” rock) will be required within the foundation footprints. Based on the borehole results, subexcavation of up to about 3 m of bedrock will be required in some foundation areas. It is noted that the bedrock is classified as strong to very strong (i.e. estimated unconfined compressive strengths in the range of about 90 MPa to 200 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. Bedrock excavation should be carried out using line drilling and pre-shearing techniques (see recommendations in Section 5.8). This method would provide better control over the configuration of the founding surface, and minimize blast damage to the rock. This procedure would be the preferred approach where deeper excavation into the bedrock is required for footing construction.

**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.

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 abutment footings should be located a minimum 2 m behind the face of the existing and/or proposed rock cut. Additional recommendations on bedrock excavation and footing setback are provided in Section 5.8 and Section 5.7.5, respectively.

The simplest option for the east abutment footing, from a foundation perspective, is a spread footing 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. However, at the west abutment location, the bedrock will need to be subexcavated (Option No. 2) to remove the partially detached rock and to avoid placing the foundation near the edge of the existing rock cut (i.e. within the minimum setback distance of 2 m).

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).

### **Pier Location**

For the centre pier location, the near surface bedrock underlying the shallow overburden fills contains zones of broken rock and low RQD values extending to depths up to about 2 m below ground surface (or about 1.5 m below top of rock). The broken rock may be the result of

previous rock blasting/shattering during construction of the existing highway and/or frost penetration/action due to the presence of standing water in the drainage ditch at this location. Based on the two cored boreholes in this area, the zones of broken rock are present from about 0.3 m and 0.5 m below existing ground surface (i.e. top of bedrock) to a depth of about 1.9 m and 2.1 m below ground surface (1.6 m below top of rock). RQD values were as low as 21 and 35 percent to a depth of 2.1 m below top of rock. As a result, the overburden soils and loose broken rock should be removed to expose the underlying “intact” rock and mass concrete used to raise the grade as required to provide a level surface for pier construction.

For design of the pier foundation, it is recommended that a founding level of Elevation 257.0 m be assumed (i.e. sub-excavate and remove rock to a minimum of approximately 1.5 m below the bedrock surface). Provision should be made in the contract for placement of mass concrete if the “intact” rock level is lower than anticipated. In addition, provision should be made for excavation of rock (i.e. by hoe-ram and possibly line drilling or controlled blasting where necessary) to reach the recommended footing level.

It is noted that footing excavations to expose the bedrock surface may, within the area of the centre pier, extend through the existing water-filled median ditch. A suitable dewatering/diversion scheme (as discussed in Section 5.8.2) will be required in order to maintain a dry and stable excavation especially during periods of high groundwater levels.

### **5.3.1 Geotechnical Resistance**

For the abutments and pier, spread footings placed on the surface of the properly prepared “intact” granite 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 intact granite 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 mucked 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 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 also check the sliding resistance between the base of the mass concrete and the bedrock. The coefficient of friction,  $\tan \delta$ , may be taken as 0.7 between the base of the concrete footings and/or mass concrete and the 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.

An Ultimate Limit State (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 C).

### 5.3.3 Frost Protection

For spread footings or mass concrete founded on the properly prepared intact granite gneiss bedrock at this site, frost susceptibility is not an issue. However, where broken or highly fractured rock is present, footings should be provided with a minimum of 1.7 m of soil cover for frost protection.

## 5.4 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.

## 5.5 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 SP 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):

	<b>SSM (sand fill)</b>	<b>SSM (rock fill)</b>
Soil / rock unit weight:	20 kN/m <sup>3</sup>	19 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:		
Active, $K_a$	0.33	0.24
At rest, $K_o$	0.50	0.38

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

	<b>Granular 'A'</b>	<b>Granular 'B'</b>
		<b>Type II</b>
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
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 and retaining walls. 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 8.5 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;
- $\gamma'$  is the effective unit weight of the soil ( $\text{kN/m}^3$ )
  - taken as soil unit weights given above for fill materials
  - taken as  $19 \text{ kN/m}^3$  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).

## **5.6 Approach Embankment Design and Construction**

Based on the information provided on the Preliminary General Arrangement Drawing for the site, the proposed top of grade for Pinedale Road/Hewitt Street at the structure location ranges from about Elevation 267.1 m to 267.9 m. The existing ground surface ranges from approximate Elevation 258.5 m to 262 m at the west approach embankment location, and ranges from approximately Elevation 259.5 m to 264 m at the east approach embankment location. As a result, the embankments will range from approximately 4.5 m to 8.5 m in height at each approach. The south side of both the east and west approach embankments will be less about 5 m in height; whereas, the north side of the east and west embankments will be up to 8.5 m in height.

Based on the borehole results, the subgrade soils at the proposed embankment locations consist of a thin layer of loose sand to silty sand with trace to some organics/roots underlain by bedrock at shallow depth. Within some of the areas of the west and east approaches, bedrock is exposed at the ground surface. There is also an existing maintenance roadway within the east approach embankment area (near BH 04-14) that contains sand fill at the ground surface. 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.

### **5.6.1 Stability**

Analyses were performed on the critical (i.e. highest) sections of the proposed new approach embankments to assess stability and liquefaction potential.

At both the east and west approach areas, bedrock is either outcropping or at shallow depth. The overburden soils, where present, are relatively thin (i.e. typically less than about 1.1 m thick on the east side and less than about 1.6 m on the west side) and composed of loose to dense cohesionless soils. For these soils, effective stress parameters were employed in the analysis assuming drained conditions and the shear strength parameters were estimated from empirical correlations using the results of the in situ Standard Penetration Tests (SPT) tempered by engineering judgment based on precedent experience in similar soils.

At all areas, the analyses assume that organic soils (encountered at or below the ground surface during field investigation operations) have been removed prior to construction of the new embankments. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling of the boreholes in and immediately adjacent to this

area. In general, the soils within the approach embankment areas were not saturated and groundwater was not observed in the open boreholes in the overburden.

The following table summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach areas. For the purposes of analysis, both earth fill and rock fill were considered for the construction of the approach embankments, and as indicated in the table below. Rock fill is assumed to have side slopes at 1.25H:1V and the earth fill is assumed to have side slopes at 2H:1V. A discussion on the different fill types, with respect to stability, is provided in Section 5.6.1.1.

#### East and West Approach Embankments

Soil Type	Unit Weight (kN/m <sup>3</sup> )	Strength Parameters
Rock Fill	19	$c' = 0 \text{ kPa}, \phi' = 38^\circ$
Earth Fill (Sand and Gravel)	21	$c' = 0 \text{ kPa}, \phi' = 35^\circ$
Loose to dense Sand to Silty Sand	20	$c' = 0 \text{ kPa}, \phi' = 30^\circ$

The results of the stability analyses for the two embankment fill options are summarized in the following table. At each area, the highest (i.e. most critical) embankment section has been analyzed. The minimum factor of safety is based on a deep-seated, global type failure surface that would impact the operation of the roadway.

<i>Location</i>	<i>Embankment Height at Critical Section (m)</i>	<i>Earth Fill Option</i>		<i>Rock Fill Option</i>	
		<i>Recommended Side Slope Profile</i>	<i>Minimum Factor of Safety</i>	<i>Recommended Side Slope Profile</i>	<i>Minimum Factor of Safety</i>
East Approach	8.5	2H : 1V	$\geq 1.3$	1.25H : 1V	$\geq 1.3$
West Approach	8.5				

The incorporation of a 2 m wide bench (or berm) into the uniform side slope profile may be required at certain sections of the proposed fill embankments as per OPSD – 202.010 and MTO Northern Region guidelines. The presence of a berm will increase the internal and surficial stability of the embankment and aid in surface water control on the slope. The presence of this berm has been incorporated in the stability analysis, where required. Additional details on the berm requirements are described in the following section.

### **5.6.1.1 Embankment Fill Types and Berm Requirements**

Based on the anticipated embankment fill heights and existing soil 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. Generally, rock fill placed above earth fill is preferred to prevent loss of finer material. However, if 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).

#### **5.6.1.1.1 Earth Fill**

The main advantage of using earth fill (i.e. sand and gravel) 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.

For the earth fill option, the incorporation of a 2 m wide mid-height bench (or berm) into the uniform side slope profile is required wherever the embankment will exceed a height of 8 m.

#### **5.6.1.1.2 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 the rock cuts proposed for the underpass, thus providing an advantage in cost. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first and second year of construction.

For the rock fill option, the incorporation of 2 m wide berms (or successive benches) into the uniform side slope profile is required wherever the embankment will exceed a height of 10 m such that the uninterrupted rock fill slope never exceeds a height of 10 m (as per most recent MTO Northern Region guidelines).

### **5.6.2 Liquefaction Potential**

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.06 g, a factor of safety of greater than 3 against liquefaction is obtained for a magnitude 7.0 earthquake event. Total seismic settlements are calculated to be less than 10 mm based on analysis performed in accordance with Tokimatsu and Seed (1987). Pseudo-static methods of embankment stability analysis indicate that a yield acceleration of approximately 0.09 g results in a factor of safety against side slope instability of 1.0. Based on this yield acceleration and the correlation proposed by Makdisi and Seed (1978), it is estimated that very little additional deformations (i.e. less than about 5 mm) of the embankment could result under the design earthquake event.

### **5.6.3 Settlement**

Settlement analyses were performed on the critical sections of the proposed approach embankments. For these analyses, the critical sections are assumed to correspond to the greatest new embankment heights, approximately 8.5 m at both the east and west approaches. The unit weights and slope profiles for the embankment fill described in Section 5.6.1 were employed in the analyses. The analyses performed assume that the organic soils/topsoil have been removed prior to construction.

As noted previously, within the east and west approach embankment areas, bedrock is either outcropping or at shallow depth (i.e. typically less than 1.6 m deep) and the overburden soils are composed of cohesionless soils. Surficial deposits of topsoil were encountered at some of the investigated locations.

Provided that the surficial topsoil is removed prior to the new embankment fill placement (as discussed in Section 5.6.4), settlements of the new approach embankments, due to compression of the thin foundation soils, are expected to be small. For 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. Estimated post-construction settlements are summarized in Table 3.

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.

### 5.6.3.1 Settlement of Cohesionless Foundation Soils

The immediate compression of the loose to dense sand to silty sand native subsoils encountered in the boreholes in the area of the approaches were modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and correlations proposed by Bowles (1984) and Kulhway and Mayne (1990).

The following table presents the results of the estimated settlements of the foundation soils as a result of the new embankment construction in the area of the approaches.

<i>Location of Embankment</i>	<i>Approximate Chainage*</i>	<i>Maximum New Embankment Thickness** (m)</i>	<i>Estimated Settlement of Foundation Soils*** (mm)</i>
West Approach	9+950 to 9+975	7.5	0 to 30
East Approach	10+025 to 10+050	6.8	0 to 15

Notes : \* chainage referenced to proposed Pinedale/Hewitt Street centreline  
 \*\*includes additional fill after removal of estimated 0.3 m depth of organics/topsoil  
 \*\*\*range in settlement includes bedrock outcrops in some areas after organics/topsoil removed

These settlements are expected to occur rapidly (i.e. during or shortly after construction) in response to the filling based on the estimated relatively high permeability of the native soils as indicated by the results of the grain size distributions.

### 5.6.3.2 Settlement of Rock Fill

If rock fill is used for the construction of the 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 8.5 m high approach embankments, the settlement of the rock fill will be about 1% of the new effective height of rock fill. Estimated total and differential settlements within the approach embankments are shown in the table below.

<i>Location of Embankment</i>	<i>Approximate Chainage</i>	<i>Maximum Rock Fill Embankment Thickness* (m)</i>	<i>Estimated Maximum Settlement of Rock Embankment (mm)</i>	<i>Estimated Differential Settlement (mm)</i>
West Approach	9+950 to 9+975	7.5	75	50
East Approach	10+025 to 10+050	6.8	68	45

Notes : \*includes additional fill required after removal of estimated 0.3 m depth of organics/topsoil

It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction.

### **5.6.3.3 Settlement of Earth (Granular) Fill**

Where earth (granular) fill 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.

### **5.6.4 Mitigation of Approach Embankment Settlement and Stability**

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 fill (i.e. compacted 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 and as shown on Table 3. 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 granular earth 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 earth (granular) fill placed above rock fill should consist of Granular 'A' or Granular 'B' Type II material (OPSS 1010).

Although the use of granular earth fill mitigates settlement issues related to the approach slab, there is not enough distance between the crest of the embankment and the minimum clearance required at the embankment toe (i.e. adjacent to Highway 11 at the foreslope and due to property restrictions along the length of the embankment) to allow for the required 2H:1V side-slope profile for earth fill. In order to achieve steeper side-slopes while maintaining granular earth fill



below the approach slab, a detail similar to that shown in Figure 1A could be incorporated into the design. Figure 1A and 1B shows typical sections at the east abutment approach slab location which uses temporary granular earth fill side-slopes at 1.5H:1V, covered with rock fill having permanent side-slopes at 1.25H:1V.

Based on the design drawings, the proposed span lengths may require embankment foreslopes to act as one side of the drainage ditch at both abutment locations. As a result, we understand that rock protection is needed at the base of the foreslopes. Using a detail similar to that provided in Figure 1B (which is compatible with the detail shown in Figure 1A) will result in rock protection (i.e. rock fill) at the foreslopes, sloped at 1.25H:1V with a minimum thickness of 500 mm. As a result, the rock fill allows for steeper foreslopes which provide the minimum clearances required and also provides rock protection within the drainage ditches. The rock fill used for the drainage ditches should be incorporated into the hydraulic design.

#### **5.6.5 Subgrade Preparation and Embankment Construction**

The existing native subsoils 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 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.

Table 3 summarizes the recommended fill type to be placed for the widenings, the location and depth of organics, the recommended side slope profiles, the requirements for side berms, the anticipated differential settlements, platform widenings (in accordance with NRE 98-200) and the recommended method of removal of organics. The following sections provide details on the recommendations for subgrade preparation and embankment construction.

#### **5.6.6 Removal of Organics**

Based on the information from the borings obtained during the field investigation, organic deposits (i.e. topsoil, roots/organics and wood pieces) of up to about 0.7 m deep, but typically about 0.3 m deep, can be expected in some areas of the new approach embankments. These organic layers should be stripped from the plan limits of the approach areas prior to fill placement.

#### **5.6.7 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. For temporary earth fill embankments that are to be covered with rock fill (see Figure 1), side slopes should be no steeper than 1.5H:1V and should be protected from surficial erosion until the rock fill cover is provided.

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.

Vegetation cover should be established on all soil slopes to protect embankment fill against surficial erosion.

## **5.7 Design and Construction Considerations**

### **5.7.1 Excavation**

Excavations for construction of spread footings on bedrock will typically extend through about 0.2 m to 2.6 m of topsoil and very loose to very dense native sand and silty sand containing varying amounts of organics.

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 to silty sand soil containing organics is classified as Type 3 soil according to OHSA. Excavations at the abutments will extend through relatively dry soils (depending on the time of year) with only minor seepage expected near the bedrock surface in some areas. At the pier, the shallow excavations may extend below a perched water level located about 0.5 m below ground surface (Elevation 258.5 m to 259.0 m).

Temporary excavations (i.e. those that are open only for a relatively short period) greater than 1.2 m deep through the fill and native soil materials may be made with side slopes no steeper than about 1H:1V.

It is noted that the bedrock is classified as strong to very strong (i.e. estimated UCS in the range of 90 kPa to 200 kPa). 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 structure foundations should be carried out using line drilling and pre-shearing techniques (as discussed in Section 5.8). 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.

### **5.7.2 Groundwater and Surface Water Control**

At the low lying areas at the centre pier and adjacent to the west abutment, groundwater was encountered at depths ranging from 0 m (i.e. at ground surface) to about 0.6 m below ground surface (Elevation 258.5 m to 259.0 m). These low lying areas are located within the existing Highway 11 median and side storm water drainage ditches. Considering groundwater was not encountered in the other boreholes located within the abutment footprints, the water levels in the ditch areas are considered to be local “perched” water levels. At the abutments, the ground and bedrock surface are at generally higher elevations and as such, depending on the time of year, groundwater may not be encountered during abutment footing construction. However, in all cases a dry and stable excavation will be required to permit placement of mass concrete and construction of footings in the dry.

It is anticipated that for open-cut excavations through the soil or rock, groundwater where encountered (i.e. particularly at the centre pier), can be adequately controlled by diverting the existing stormwater drainage paths to promote run-off away from or around the proposed construction areas and by pumping from properly filtered sumps. Surface water should be directed away from the excavations at all times.

### **5.7.3 Obstructions**

The fill soils at the site (i.e. roadway embankment and/or median fills) may contain cobbles and boulders particularly near the soil/bedrock interface. Conventional excavation equipment should be suitable for the majority of excavation through the on-site soils; however, the presence of rock fragments or boulders may interfere with or slow the progress of stripping and excavation at some locations. Ultimately, provision will have to be made in the Contract Specifications to ensure that the Contractor is equipped to handle such obstructions.

### **5.7.4 Rock Hazards at Existing Rock Cuts**

For existing rock cuts at the Highway 11 / Pinedale Road Underpass location that do not require further excavation to increase the current clear zone (i.e. the west side), the available catchment

area is deemed sufficient in that the potential for rockfalls reaching the roadway is considered unlikely. It should be noted that for the catchment ditch area to be effective, rockfalls should be cleared from the ditch area promptly. If rock is allowed to accumulate in the ditch or clear zone area, subsequent rockfalls may overspill onto the roadway. Currently, rock cuts in the vicinity of the abutments are up to 4 m and 6 m high (approximately 2 m to 3 m high at the abutment locations) and the clear zone ranges from 6 m to 8 m wide. As shown on Figures B1 and B2, there are partially detached blocks of rock along the crest of the existing rock cuts which extend back from the face approximately 2 m. As a result, the proposed bridge abutment footings should be located behind these potentially loose blocks shown on the figures. The proposed minimum footing offset / setback distance required from the proposed rock cut at the east abutment is shown on Drawing 1. We understand that there is sufficient setback distance at the west abutment location; thus, no rock cut has been proposed.

#### **5.7.5 Proposed Permanent Rock Cut Slopes**

For the rock cuts which are planned to be excavated to create the required clear zone (i.e. at the east side), 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. However, with time the rock faces will weather due to the blocky nature of the jointing and as such the abutment footings should be located at least 2 m behind the face of the existing and/or proposed rock cut as indicated on Drawing 1.

For permanent cut slopes through the bedrock, the overall slope to the cut face may be formed vertical to near vertical (i.e. 0.25 horizontal (H) to 1 vertical(V)). 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.

### **5.8 Blasting Recommendations for Rock Excavations**

#### **5.8.1 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.

## **5.8.2 Special Provisions**

### **5.8.2.1 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; an example is provided in Appendix C. The NSSP 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.

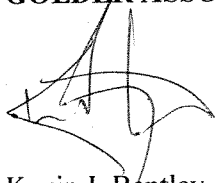
We recommend limiting ground vibration levels to 50 mm/s for adjacent services and buildings. Continuous monitoring of all blasting operations would dictate when changes to the blast procedures become necessary to meet these limits and how close to the blasting approaches the adjacent structures.

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.

## 5.9 Closure

This report was prepared by Ms. Shannon Palmer, EIT and Mr. Kevin Bentley, P.Eng, a geotechnical engineer, and reviewed by Mr. J. Paul Dittrich, P.Eng., an Associate 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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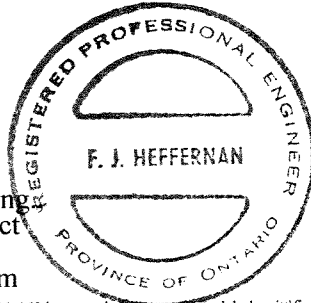
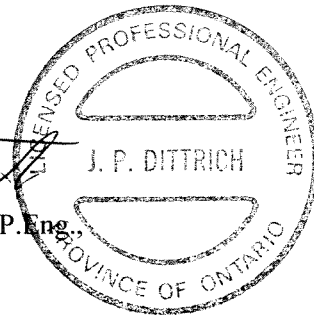
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**TABLE 1**  
**SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES**

PROJECT NO.:04-1111-039A

TITLE: Highway 11 and Pinedale Road/Hewitt Street Underpass, Gravenhurst

DATE: November 30, 2004

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core <sup>(2)</sup> Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	Is (50mm) (MPa)	Approx. <sup>(1)</sup> UCS (MPa)
1	1	2.1	D	260.4	46.5		14079.6	13.62		6.309	6.104	140
	2	2.9	D	298.5	47.2		14121.0	13.66		6.132	5.974	137
	3	3.7	D	165.1	47.4		15258.6	14.76		6.569	6.413	147
	4	4.3	D	279.4	47.4		10039.1	9.71		4.313	4.212	97
	5	4.3	D	158.8	47.4		14920.8	14.43		6.410	6.260	144
4	1	1.3	D	114.3	47.3		12238.6	11.84		5.297	5.165	119
	2	1.7	A	53.8	47.2	56.86	14817.4	14.33	4.432		4.696	108
	3	2.5	D	158.8	47.2		13603.8	13.16		5.901	5.751	132
	4a	3.0	D	190.5	47.2		11142.3	10.78		4.828	4.706	108
	4b	3.0	A	45.7	47.2	52.44	12997.1	12.57	4.571		4.670	107
	5	3.7	D	215.9	47.3		9859.9	9.54		4.268	4.161	96
	6	4.4	D	285.8	47.4		12879.9	12.46		5.539	5.408	124
	7	3.6	D	158.8	47.4		10418.3	10.08		4.480	4.375	101

<sup>(1)</sup> Is<sub>50</sub> 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.

<sup>(2)</sup> 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.:04-1111-039A

TITLE: Highway 11 and Pinedale Road/Hewitt Street Underpass, Gravenhurst

DATE: November 30, 2004

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core <sup>(2)</sup> Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	Is (50mm) (MPa)	Approx. <sup>(1)</sup> UCS (MPa)
9	1	1.61544	D	101.6	47.4		16499.7	15.96		7.111	6.940	160
	2	2.98704	D	139.7	47.0		12438.6	12.03		5.454	5.302	122
	3	4.08432	D	222.25	47.3		16982.4	16.42		7.350	7.167	165
	4	5.51688	D	222.25	47.3		14638.1	14.16		6.336	6.178	142
	5	6.2484	D	158.75	47.2		13121.2	12.69		5.691	5.547	128
10	1	1.15824	D	266.7	47.0		14017.5	13.56		6.133	5.965	137
	2	1.40208	A	71.374	47.2	65.52	17182.3	16.62	3.871		4.371	101
	3	1.46304	A	45.212	47.2	52.15	10942.4	10.58	3.891		3.966	91
	4	2.1336	D	152.4	47.2		15038.0	14.54		6.530	6.362	146
	5	3.29184	D	86.36	47.3		14079.6	13.62		6.087	5.937	137
	6	3.41376	A	62.992	47.5	61.72	22201.9	21.47	5.636		6.197	143
	7	4.191	D	167.64	47.4		18182.1	17.58		7.836	7.648	176
11	1	0.7	D	130.8	47.0		9142.8	8.84		4.000	3.891	89
	2	2.0	D	108.0	47.4		20278.2	19.61		8.720	8.515	196
	3	3.1	D	207.0	47.4		20843.6	20.16		8.964	8.753	201
	4	4.5	D	311.2	47.4		20257.5	19.59		8.712	8.506	196

<sup>(1)</sup> Is<sub>50</sub> 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.

<sup>(2)</sup> Actual distance between point load cones at time of failure.

**TABLE 2**  
**EVALUATION OF FOUNDATION ALTERNATIVES**  
**Highway 11 and Pinedale Road / Hewitt Street Underpass**  
**G.W.P. 314-00-00**

<i>Footing Option</i>	<i>NF</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Spread Footings on bedrock or mass concrete pad on bedrock		Can minimize bedrock excavation depending on design footing level.	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.	Much lower relative costs than piled foundations since less bedrock excavation required.	If bedrock is higher than anticipated, additional bedrock removal is required;  Variability in bedrock surface will impact mass concrete quantities and excavation depths.
Spread Footings perched within embankment fill	NF	-	Not practical at abutment where bedrock surface is already 2 m to 3 m higher than existing Hwy. 11 road surface;  Potential for differential settlement between abutments (due to compression of embankment fill) and central pier (founded on unyielding bedrock).	-	-
Piles / Caissons	NF	-	Due to shallow depth of bedrock, bedrock excavation to form trench or rock drilling will be required to achieve minimum required pile/caisson lengths;  Significant depth of excavation required in strong to very strong bedrock.	-	-

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

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**TABLE 3**  
**Summary of Recommendations at Structure Approach Embankments (incl. Platform Widening)**  
**Highway 11 and Pinedale Road / Hewitt Street Underpass**  
**G.W.P. 314-00-00**

Highway	Approx. Station	Proposed Works	Surface Conditions	Recommended Embankment Fill Type	Organics Encountered along alignment	Recommended Side Slope	Side Berm Recommended	Estimated Post-Construction Settlements ( $\delta$ ) and Platform Widening ( $w$ ) (mm)	Stripping / Organics Removal OPSD
Highway 11 and Pinedale Road / Hewitt Street Underpass	9+950 to 9+975	West Approach (fill ranges from 4.5 m to 8.5 m high within footprint, and fill typically ranges from 5.3 m to 6.3 m thick immediately behind abutment within footprint of approach slab)	Sloping Bedrock at ground surface to a depth of 2.6 m below ground surface, below sandy overburden	Granular Fill below approach slab (see Figure 1)  Rock Fill beyond approach slab	Yes. Up to 0.7 m below ground surface; but typically 0.3 m bgs.	1.25H : 1V for surficial rock fill;  1.5H:1V for temporary granular fill covered with rock fill (see Figure 1)	No.  (2 m wide berm required only if embankment height exceeds 10 m).	$\delta_{\max}$ = 75 mm (rock fill) $\delta_{\max}$ < 25 mm (granular fill) $\delta_{\text{diff}}$ < 25 mm (within approach slab footprint location, see Figure 1)  $w$ = 2000 mm (up to 5 m beyond limit of wingwalls, see Figure 1) transitioning to $w$ = 1000 mm in all other areas.	Strip and remove all organics within footprint of embankment.
	10+025 to 10+050	East Approach (fill ranges from 4.5 m to 8.5 m high within embankment footprint, and fill ranges from 5.0 m to 6.8 m thick immediately behind abutment within footprint of approach slab)	Bedrock at ground surface or shallow sandy overburden	Granular Fill below approach slab (see Figure 1)  Rock Fill beyond approach slab	Yes. Up to 0.6 m below ground surface; but typically 0.3 m bgs.	1.25H : 1V for surficial rock fill;  1.5H:1V for temporary granular fill covered with rock fill (see Figure 1)	No.  (2 m wide berm required only if embankment height exceeds 10 m).	$\delta_{\max}$ = 68 mm (rock fill) $\delta_{\max}$ < 25 mm (granular fill) $\delta_{\text{diff}}$ < 25 mm (within approach slab footprint location, see Figure 1)  $w$ = 2000 mm (up to 5 m beyond limit of wingwalls, see Figure 1) transitioning to $w$ = 1000 mm in all other areas.	Strip and remove all organics within footprint of embankment.

**Notes :**

\* Settlements below travelled lanes include compression of rockfill and earth fill but do not include settlements of foundation soils which should occur during construction.

\*\* Recommended embankment platform widening (per embankment side) where rock fill is used for construction based on guidelines in NRE 98-200.

$\delta_{\max}$  = maximum calculated settlement within footprint of approach embankment.

$\delta_{\text{diff}}$  = anticipated differential settlement that could occur directly behind the abutment and in the proposed approach slab footprint.

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## 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

### 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

### 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.)

#### 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

#### 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<sup>2</sup> 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.

### 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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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 <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	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

$\pi$	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
F	factor of safety
V	volume
W	weight

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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
$I_p$	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
$\sigma'_p$	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	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  $= (\text{compressive strength})/2$
  - \* density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density x acceleration due to gravity)

# 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

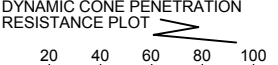
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

### 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	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/ Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	

PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-1</b>				1 OF 1 <b>METRIC</b>											
W.P. <u>314-00-00</u>		LOCATION <u>N 4974462.2;E 316615.8</u>				ORIGINATED BY <u>SB</u>											
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>											
DATUM <u>Geodetic</u>		DATE <u>15-Nov-04</u>				CHECKED BY <u>KB</u>											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub> W      W <sub>L</sub> WATER CONTENT (%)			γ kN/m <sup>3</sup>	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	10 20 30								
261.7	GROUND SURFACE																
0.0	Silty SAND, trace to some roots and organics Reddish brown Loose to dense Moist		1	SS	5		261										
			2	SS	66/0.23												
260.4																	
1.4	GRANITE GNEISS Fresh to slightly weathered, strong to very strong, fine to medium crystalline, foliated, dark grey/black and pink  Bedrock cored from 1.35 m to 4.6 m  For bedrock coring details refer to record of drillhole BH04-1						260										
							259										
							258										
257.1																	
4.6	End of Borehole  Note:  1. Borehole dry during drilling operations																

PROJECT: 04-1111-039

**RECORD OF DRILLHOLE: BH04-1**

SHEET 2 OF 2

LOCATION: N 4974462.2 ;E 316615.8

DRILLING DATE: 15-Nov-04

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	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.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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DEPTH SCALE

1 : 50



LOGGED: SB

CHECKED: KB

MIS-RCK 002 041111039AARCK.GPJ GAL-MISS.GDT 8/6/06 JDR



PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-2</b>				1 OF 1 <b>METRIC</b>										
W.P. <u>314-00-00</u>		LOCATION <u>N 4974426.3 ; E 316620.3</u>				ORIGINATED BY <u>SB</u>										
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>										
DATUM <u>Geodetic</u>		DATE <u>16-Nov-04</u>				CHECKED BY <u>KB</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
261.4	GROUND SURFACE															
0.0	Silty SAND, trace to some roots and organics, trace gravel Reddish brown Loose to dense Moist		1	SS	2		261									
			2	SS	21											
259.8			3	SS	30/08		260									
1.6	Auger Refusal End of Borehole  Note:  1. Borehole dry during drilling operations															

PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-3</b>				1 OF 1 <b>METRIC</b>										
W.P. <u>314-00-00</u>		LOCATION <u>N 4974465.6 ; E 316623.3</u>				ORIGINATED BY <u>SB</u>										
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>										
DATUM <u>Geodetic</u>		DATE <u>16-Nov-04</u>				CHECKED BY <u>KB</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
261.4	GROUND SURFACE															
0.0	Silty SAND, trace to some rootlets and organics, trace gravel Orangish brown Loose to compact Moist	1	SS	4		261										
260.0		2	SS	14		260										
1.4	SAND, trace to some silt Orangish brown Compact to very dense Moist	3	SS	25		259										
258.8		4	SS	56/0.15												
2.6	Auger Refusal End of Borehole  Note: 1. Borehole dry during drilling operations															

PROJECT		RECORD OF BOREHOLE				No BH04-4		1 OF 1		METRIC			
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE		COMPILED BY			
DATUM		DATE		CHECKED BY		GRAIN SIZE DISTRIBUTION (%)		REMARKS					
04-1111-039		N 4974461.7 ; E 316626.4		SB		108 mm Solid Stem Power Auger		JDR		KB			
314-00-00		11		17-Nov-04									
Geodetic													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		WATER CONTENT (%)		
259.4	0.0	GROUND SURFACE		1	SS	3		259	20	40	60	80	100
258.7	0.7	Sand, trace to some roots and organics, clayey silt pockets at 0.7 m depth, contains granitic fragments Brown Loose Moist to wet (Probable Fill) GRANITE GNEISS Fresh, strong to very strong, fine to medium crystalline, foliated, dark grey/black and pink Bedrock cored from 0.7 m to 4.8 m For bedrock coring details refer to record of drillhole BH04-4		2	SS	30/08		258	20	40	60	80	100
								257	20	40	60	80	100
								256	20	40	60	80	100
								255	20	40	60	80	100
254.7	4.8	End of Borehole											
		Note: 1. Water level in open borehole at 0.6m depth during drilling operations											

PROJECT: 04-1111-039

**RECORD OF DRILLHOLE: BH04-4**

SHEET 2 OF 2

LOCATION: N 4974461.7 ;E 316626.4

DRILLING DATE: 17-Nov-04

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COL- OUR % RETURN	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.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
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1	Rotary NQ	- continued from Record of Borehole -  GRANITE GNEISS Fresh, strong to very strong, fine to medium crystalline, foliated, dark grey/black and pink  Broken zones with gravel size pieces encountered between:  El. 258.56 and El. 258.54; El. 257.27 and El. 257.22; El. 256.66 and El. 256.58.		258.69 0.71	1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														

DEPTH SCALE

1 : 50






LOGGED: SB



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MIS-RCK 002 041111039AARCK.GPJ GAL-MISS.GDT 8/6/06 JDR


PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-5</b>				1 OF 1 <b>METRIC</b>										
W.P. <u>314-00-00</u>		LOCATION <u>N 4974468.9; E 316616.8</u>				ORIGINATED BY <u>SB</u>										
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>										
DATUM <u>Geodetic</u>		DATE <u>16-Nov-04</u>				CHECKED BY <u>KB</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
261.3	GROUND SURFACE															
0.0	SAND, some silt, trace to some organics (roots, leaves), trace gravel	1	SS	8		261										
260.9	Reddish brown															
0.5	Loose Moist Auger Refusal End of Borehole															
Note: 1. Borehole dry during drilling operations																

PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-6</b>				1 OF 1 <b>METRIC</b>											
W.P. <u>314-00-00</u>		LOCATION <u>N 4974435.2; E 316641.5</u>				ORIGINATED BY <u>SB</u>											
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>											
DATUM <u>Geodetic</u>		DATE <u>17-Nov-04</u>				CHECKED BY <u>KB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
259.0	GROUND SURFACE																
0.0	Sand, some silt, trace to some gravel, trace to some topsoil, roots, leaves (Fill)		1	SS	6												
258.4	Grey Loose Moist to wet																
0.6	Auger Refusal End of Borehole																
	Note:  1. Water level in open borehole at 0.5 m depth during drilling operations																

PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-7</b>				1 OF 1 <b>METRIC</b>											
W.P. <u>314-00-00</u>		LOCATION <u>N 4974434.0 ; E 316636.7</u>				ORIGINATED BY <u>SB</u>											
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>											
DATUM <u>Geodetic</u>		DATE <u>17-Nov-04</u>				CHECKED BY <u>KB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
258.6	GROUND SURFACE																
0.0	TOPSOIL		1	SS	34												
258.2	Silty sand, some organics, some gravel (granite fragments) (Fill) Dark brown Dense Wet Auger Refusal End of Borehole					258											
0.5																	
<p>Note:</p> <p>1. Water level at ground surface during drilling operations</p>																	

PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-8</b>				1 OF 1 <b>METRIC</b>										
W.P. <u>314-00-00</u>		LOCATION <u>N 4974432.7 ; E 316631.1</u>				ORIGINATED BY <u>SB</u>										
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>										
DATUM <u>Geodetic</u>		DATE <u>17-Nov-04</u>				CHECKED BY <u>KB</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
258.8	GROUND SURFACE						<div style="display: flex; justify-content: space-between;"> <span>20 40 60 80 100</span> <span>20 40 60 80 100</span> </div>									
0.0	TOPSOIL		1	SS	25/0.08											
0.2	Sand, some silt, some gravel, some organics and topsoil (Fill) Grey Compact Moist to wet Auger Refusal End of Borehole															
Note: 1. Water level at 0.2 m depth during drilling operations																



PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-9</b>				1 OF 1 <b>METRIC</b>											
W.P. <u>314-00-00</u>		LOCATION <u>N 4974430.2 ; E 316632.8</u>				ORIGINATED BY <u>SB</u>											
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>											
DATUM <u>Geodetic</u>		DATE <u>17-Nov-04</u>				CHECKED BY <u>KB</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
259.0	GROUND SURFACE																
0.0	Sand, some silt, trace gravel, some roots, topsoil and organics (Fill)		1	SS	7												
258.5	Dark brown Loose Wet																
0.5	GRANITE GNEISS Fresh, very strong, fine to medium crystalline, dark grey/black and pink																
	Bedrock cored from 0.5 m to 6.4 m																
	For bedrock coring details refer to record of drillhole BH04-9																
252.6	End of Borehole																
6.4	Note:  1. Water level at ground surface during drilling operations																

PROJECT: 04-1111-039

## RECORD OF DRILLHOLE: BH04-9

SHEET 2 OF 2

LOCATION: N 4974430.2 ;E 316632.8

DRILLING DATE: 17-Nov-04

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLLOUR % RETURN	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.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
									RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY K, cm/sec		Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
									TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION												10 10 10 10	10 10 10 10																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
		- continued from Record of Borehole -		258.47 0.53																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													

DEPTH SCALE

1 : 50



LOGGED: SB

CHECKED: KB

MIS-RCK 002 041111039AARCK.GPJ GAL-MISS.GDT 8/6/06 JDR

PROJECT		RECORD OF BOREHOLE				No BH04-10		1 OF 1		METRIC		
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE		COMPILED BY		
DATUM		DATE		CHECKED BY		GRAIN SIZE DISTRIBUTION (%)		REMARKS				
04-1111-039		N 4974437.1 ; E 316639.6		SB		108 mm Solid Stem Power Auger		JDR		KB		
314-00-00		11		18-Nov-04								
Geodetic												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		WATER CONTENT (%)	
258.7	0.0	GROUND SURFACE							20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>
0.0		TOPSOIL		1	SS	27			20 40 60 80 100			
0.3		Sand, trace to some organics, trace gravel, some silt (FILL) Grey Compact Wet GRANITE GNEISS Fresh to slightly weathered, very strong, fine to medium crystalline, dark grey/black and pink  Bedrock cored from 0.3 m to 4.5 m  For bedrock coring details refer to record of drillhole BH04-10						258				
								257				
								256				
								255				
254.2	4.5	End of Borehole										
		Note: 1. Water level at ground surface during drilling operations										

PROJECT: 04-1111-039

**RECORD OF DRILLHOLE: BH04-10**

SHEET 2 OF 2

LOCATION: N 4974437.1 ;E 316639.6

DRILLING DATE: 17-Nov-04

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLLOID % RETURN	RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3 m	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec			Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES WATER LEVELS INSTRUMENTATION				
									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.			
																							TOTAL CORE %	SOLID CORE %	CORROSION
		- continued from Record of Borehole -		258.40 0.30																					
1	Rotary NQ	GRANITE GNEISS Fresh to slightly weathered, very strong, fine to medium crystalline, dark grey/black and pink, with medium to coarse quartz and feldspar banding  Broken zones with gravel size pieces encountered between:  El. 257.25 and El. 257.20; El. 257.10 and El. 256.98.			1										BR,,  BR,IR,Ro, VN VN,IR,Ro  FO,IR,Ro " BR,, " BR,IR,Ro PL,Ro,FO " FO,PL,Ro JN,IR,Ro IR,Ro  FO,IR,Ro " FO,IR,Ro  FO,IR,Ro FO,IR,Ro SHR,,Ro										
2				2																					
3																									
4																									
		End of Drillhole		254.20 4.50																					
5																									
6																									
7																									
8																									
9																									
10																									

DEPTH SCALE



1 : 50



LOGGED: SB

CHECKED: KB

MIS-RCK 002 041111039AARCK.GPJ GAL-MISS.GDT 8/6/06 JDR

PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-11</b>				1 OF 1 <b>METRIC</b>								
W.P. <u>314-00-00</u>		LOCATION <u>N 4974405.0 ; E 316652.0</u>				ORIGINATED BY <u>SB</u>								
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>								
DATUM <u>Geodetic</u>		DATE <u>19-Nov-04</u>				CHECKED BY <u>KB</u>								
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) W <sub>p</sub> W W <sub>L</sub>			
261.8	GROUND SURFACE							20 40 60 80 100						
261.8	TOPSOIL with roots and organics													
0.3	GRANITE GNEISS Fresh to weathered, strong to very strong, fine to medium crystalline, dark grey/black and pink  Bedrock cored from 0.3 m to 4.85 m  For bedrock coring details refer to record of drillhole BH04-11													
256.9	End of Borehole													
4.9	Note: 1. Borehole dry during drilling operations													

SHEET 2 OF 2



DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling Ltd.

CHECKED: KB

MIS-RCK 002 041111039AARCK.GPJ GAL-MISS.GDT 8/6/06 JDR

PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-12</b>				1 OF 1 <b>METRIC</b>										
W.P. <u>314-00-00</u>		LOCATION <u>N 4974405.0 ; E 316648.0</u>				ORIGINATED BY <u>SB</u>										
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>										
DATUM <u>Geodetic</u>		DATE <u>19-Nov-04</u>				CHECKED BY <u>KB</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W <sub>p</sub>	W		
262.0	GROUND SURFACE															
0.0	Silty SAND, with topsoil, roots, organics		1	SS	4									○		O.C.=6.7%
261.6	Brown Loose Moist															
0.4	Auger Refusal End of Borehole						261									
<p>Note:</p> <p>1. Borehole dry during drilling operations</p>																

PROJECT <u>04-1111-039</u>		<b>RECORD OF BOREHOLE No BH04-13</b>				1 OF 1 <b>METRIC</b>										
W.P. <u>314-00-00</u>		LOCATION <u>N 4974406.4 ;E 316658.5</u>				ORIGINATED BY <u>SB</u>										
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm Solid Stem Power Auger</u>				COMPILED BY <u>JDR</u>										
DATUM <u>Geodetic</u>		DATE <u>19-Nov-04</u>				CHECKED BY <u>KB</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
261.5	GROUND SURFACE						20	40	60	80	100					
0.0	Silty SAND, roots, organics and wood pieces, trace gravel (granitic fragments) Brown Loose to Dense Moist		1	SS	3		261									
260.4			2	SS	50/0.2											
1.1	Auger Refusal End of Borehole  Note:  1. Water level in open borehole at 1.1 m depth upon completion of drilling operations.															





+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

MIS-MTO 001 041111039AAMTO.GPJ GAL-MISS.GDT 8/6/06

PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-1</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974464.8 ;E 316630.0</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20	40	60	80	100					
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × REMOULDED									
								20	40	60	80	100					
259.9	GROUND SURFACE																
0.0	TOPSOIL																
	Refusal on Inferred Bedrock																
	End of Probehole																
							259										

PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-2</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974460.6 ;E 316620.4</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20	40	60	80	100					
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × REMOULDED									
								20	40	60	80	100					
260.7	GROUND SURFACE																
0.0	Refusal on Inferred Bedrock																
	End of Probehole																

MIS-MTO 002 041111039AAMTO.GPJ GAL-MISS.GDT 8/6/06

PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-3</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974478.3 ;E 316611.5</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
261.9	GROUND SURFACE Refusal on Inferred Bedrock End of Probehole																

PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-4</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974480.4 ;E 316627.0</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								UNCONFINED QUICK TRIAXIAL	20	40	60	80						100	FIELD VANE REMOULDED	
261.3	GROUND SURFACE Refusal on Inferred Bedrock End of Probehole																			

MIS-MTO 002 041111039AAMTO.GPJ GAL-MISS.GDT 8/6/06



+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-7</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974410.7 ;E 316656.1</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
259.9	GROUND SURFACE Refusal on Inferred Bedrock End of Probehole																

PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-8</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974394.6 ;E 316651.7</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								UNCONFINED QUICK TRIAXIAL			FIELD VANE REMOULDED									
262.8	GROUND SURFACE Refusal on Inferred Bedrock End of Probehole							20	40	60	80	100		10	20	30				

MIS-MTO 002 041111039AAMTO.GPJ GAL-MISS.GDT 8/6/06

PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-9</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974401.5 ; E 316647.9</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
262.5 0.0	GROUND SURFACE Refusal on Inferred Bedrock End of Probehole																

PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-10</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974405.0 ; E 316646.0</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								UNCONFINED QUICK TRIAXIAL			FIELD VANE REMOULDED									
262.3 0.0	GROUND SURFACE Refusal on Inferred Bedrock End of Probehole							20	40	60	80	100		10	20	30				

MIS-MTO 002 041111039AAMTO.GPJ GAL-MISS.GDT 8/6/06

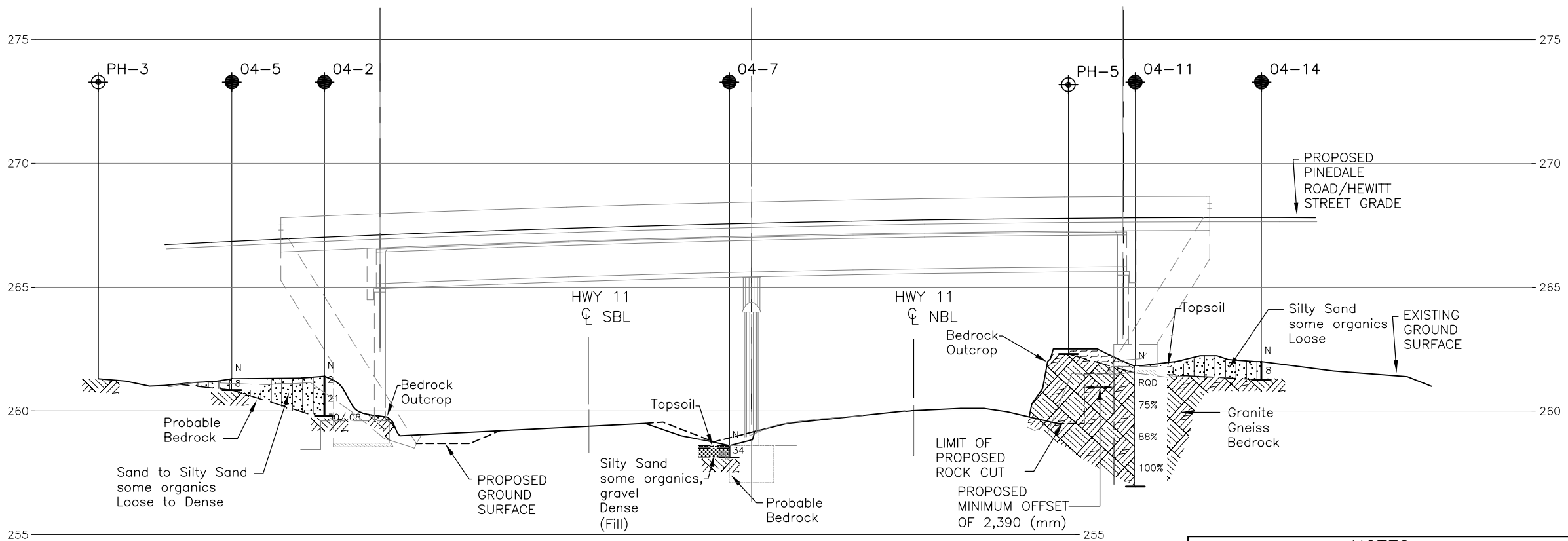
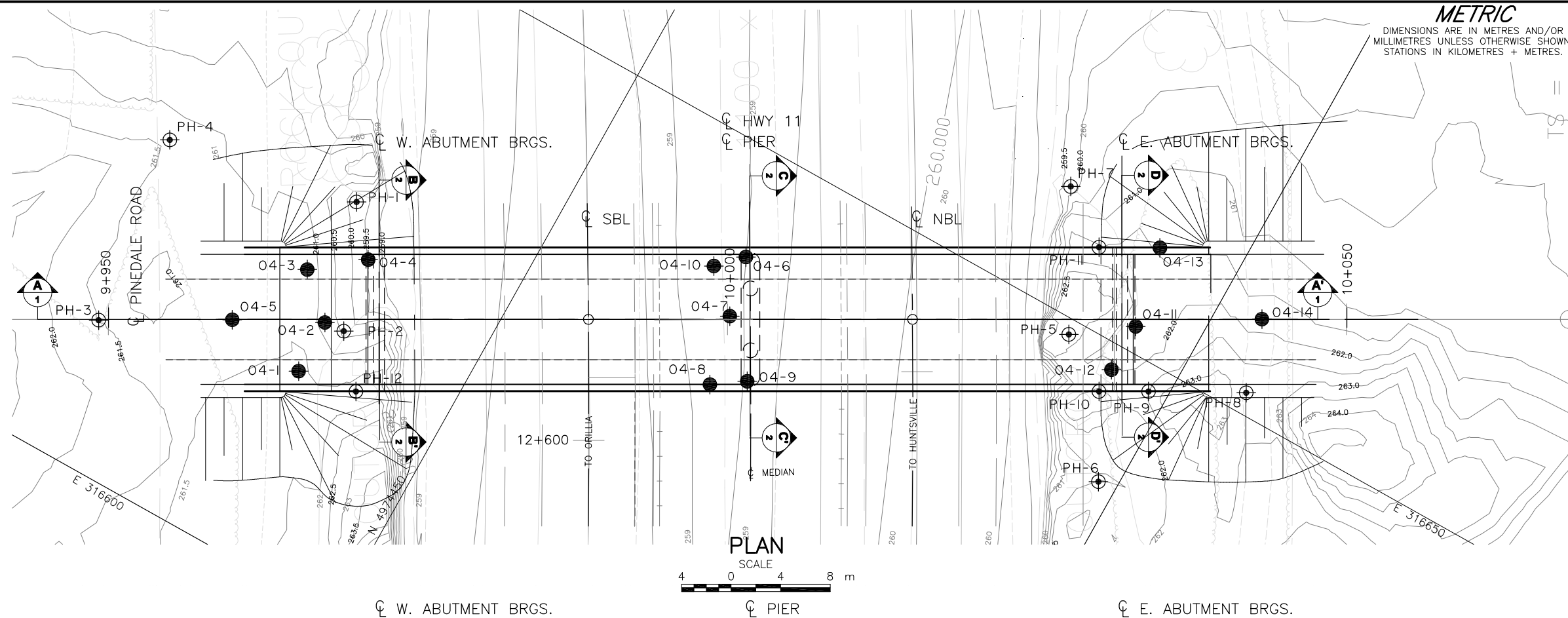
PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-11</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974410.7 ;E 316656.1</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W <sub>p</sub>	W			W <sub>L</sub>
							20	40	60	80	100						
							○ UNCONFINED   + FIELD VANE					WATER CONTENT (%)					
							● QUICK TRIAXIAL   × REMOULDED										
261.6	GROUND SURFACE						20	40	60	80	100		10	20	30	kN/m <sup>3</sup>	GR SA SI CL
0.0	Refusal on Inferred Bedrock End of Probehole																

PROJECT <u>04-1111-039</u>		<b>RECORD OF PROBEHOLE No PH-12</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>314-00-00</u>		LOCATION <u>N 4974451.4 ;E 316616.6</u>		ORIGINATED BY <u>KJB</u>	
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>10 mm diameter Steel Rod</u>		COMPILED BY <u>MSM</u>	
DATUM <u>Geodetic</u>		DATE <u>November 15, 2005</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								UNCONFINED QUICK TRIAXIAL		FIELD VANE REMOULDED										
261.8	GROUND SURFACE							20	40	60	80	100		10	20	30				
0.0	Refusal on Inferred Bedrock End of Probehole																			

MIS-MTO 002 041111039AAMTO.GPJ GAL-MISS.GDT 8/6/06



**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

CONT No.  
WP No. 314-00-00

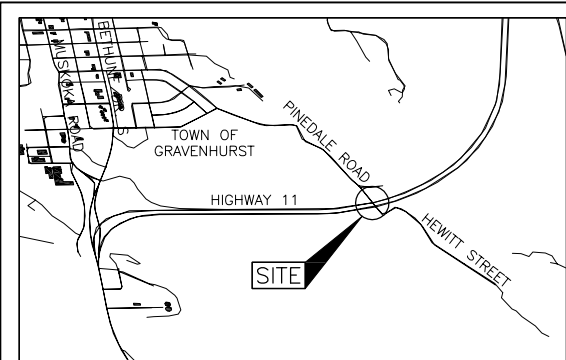


HIGHWAY 11 AND  
PINEDALE ROAD/HEWITT STREET  
UNDERPASS  
BOREHOLE LOCATION & SOIL STRATA

SHEET



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



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

**LEGEND**

- Borehole - Current Investigation
- ⊕ Probehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL upon completion of drilling

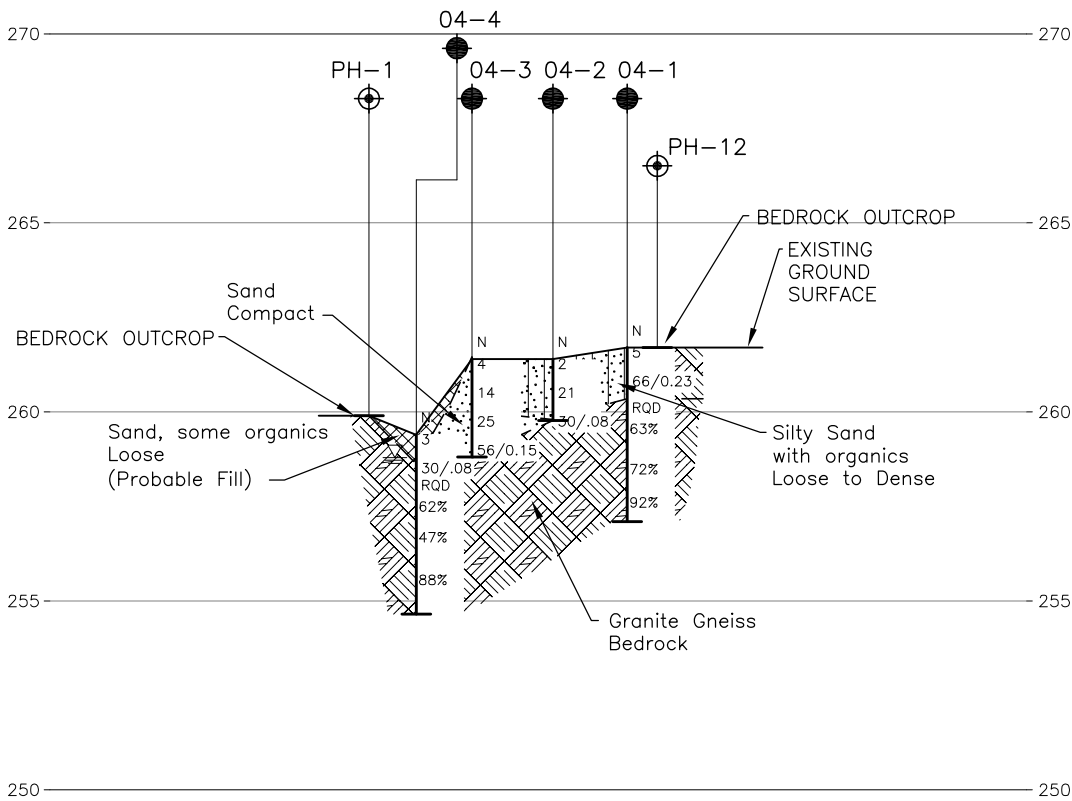
No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
04-1	261.7	4974462.2	316615.8
04-2	261.4	4974462.3	316620.3
04-3	261.4	4974465.6	316623.3
04-4	259.4	4974461.7	316626.4
04-5	261.3	4974468.9	316616.8
04-6	259.0	4974435.2	316641.5
04-7	258.6	4974434.0	316636.7
04-8	258.8	4974432.7	316631.1
04-9	259.0	4974430.2	316632.8
04-10	258.7	4974437.1	316639.6
04-11	261.8	4974405.0	316652.0
04-12	262.0	4974405.0	316648.0
04-13	261.5	4974406.4	316658.5
04-14	262.0	4974396.4	316657.5
PH-1	259.9	4974464.8	316630.0
PH-2	260.7	4974460.6	316620.4
PH-3	261.9	4974478.3	316611.5
PH-4	261.3	4974480.4	316627.0
PH-5	262.3	4974409.4	316648.8
PH-6	261.5	4974401.5	316639.6
PH-7	259.9	4974415.1	316659.3
PH-8	262.8	4974394.6	316651.7
PH-9	262.5	4974401.5	316647.9
PH-10	262.3	4974405.0	316646.0
PH-11	261.6	4974410.7	316656.1
PH-12	261.8	4974457.4	316616.6

**REFERENCE**

Base plans and General Arrangement dwg provided in digital format by McCORMICK RANKIN CORPORATION, drawing file nos. 5799-align-dec13-04.dwg, 5799-sp plan.dwg and 05799\_XB1.dwg, received December 15, 2004. General Arrangement dwg file no. 57989-302-001.dwg, received February 10, 2005, revised July 22, 2005

NO.	DATE	BY	REVISION
Geocres No. 31D00-410			
HWY. 11	PROJECT NO. 04-1111-039		DIST.
SUBM'D.	CHKD. KB	DATE: OCT 2005	SITE:
DRAWN: JFC/MSM	CHKD. JPD	APPD. FJH	DWG. 1

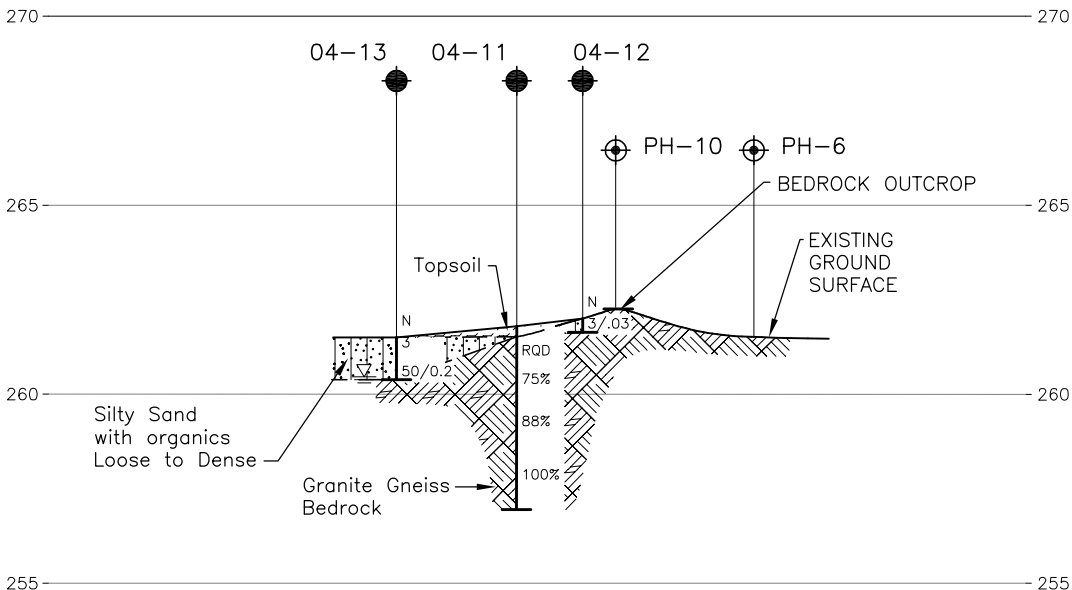




**B-B'**  
**1** CROSS SECTION - WEST ABUTMENT

HORIZ. SCALE 4 0 4 8 m

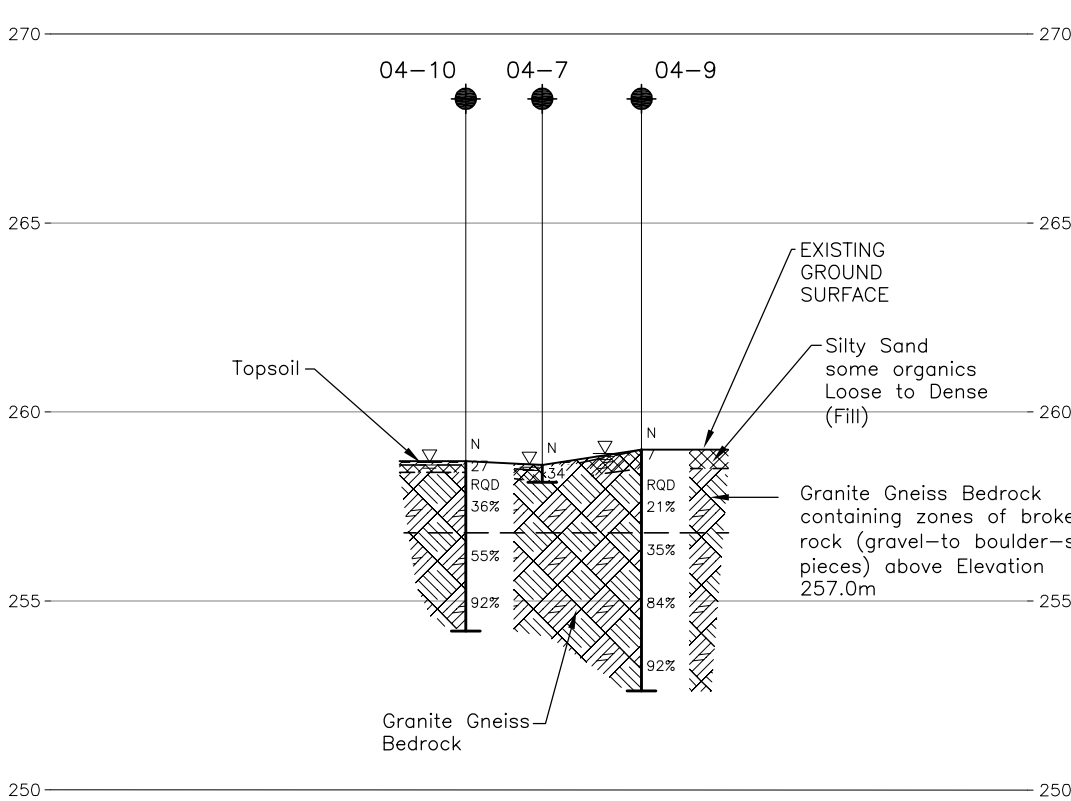
VERT. SCALE 2 0 2 4 m



**D-D'**  
**1** CROSS SECTION - EAST ABUTMENT

HORIZ. SCALE 4 0 4 8 m

VERT. SCALE 2 0 2 4 m



**C-C'**  
**1** CROSS SECTION - CENTRE PIER

HORIZ. SCALE 4 0 4 8 m

VERT. SCALE 2 0 2 4 m

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

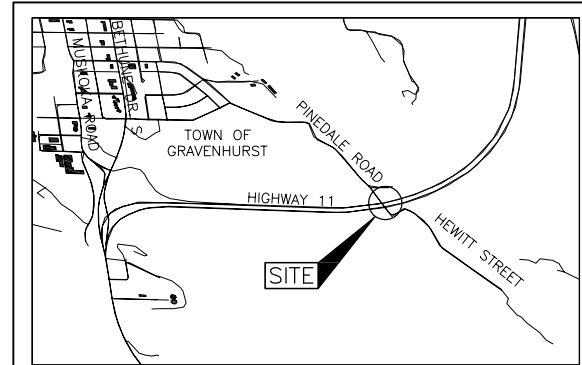
CONT No.  
WP No. 314-00-00

HIGHWAY 11 AND  
PINEDALE ROAD/HEWITT STREET  
UNDERPASS  
BOREHOLE SOIL STRATA

SHEET



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



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

**LEGEND**

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- Probehole - Current Investigation
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		NORTHING	EASTING
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04-4	259.4	4974461.7	316626.4
04-7	258.6	4974434.0	316636.7
04-9	259.0	4974430.2	316632.8
04-10	258.7	4974437.1	316639.6
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04-12	262.0	4974405.0	316648.0
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PH-1	259.9	4974464.8	316630.0
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PH-10	262.3	4974405.0	316646.0
PH-12	261.8	4974457.4	316616.6

**NOTES**

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For subsurface information only.

NO.	DATE	BY	REVISION
Geocres No. 31D00-410			
HWY. 11	PROJECT NO. 04-1111-039		DIST.
SUBM'D.	CHKD. KB	DATE: OCT 2005	SITE:
DRAWN: JFC/MSM	CHKD. JPD	APPD. FJH	DWG. 2

SCHEMATIC OF TYPICAL EMBANKMENT FILL DETAILS AT ABUTMENTS  
 HIGHWAY 11 AND PINEDALE ROAD/HEWITT STREET UNDERPASS  
 G.W.P. 314-00-00

FIGURE 1

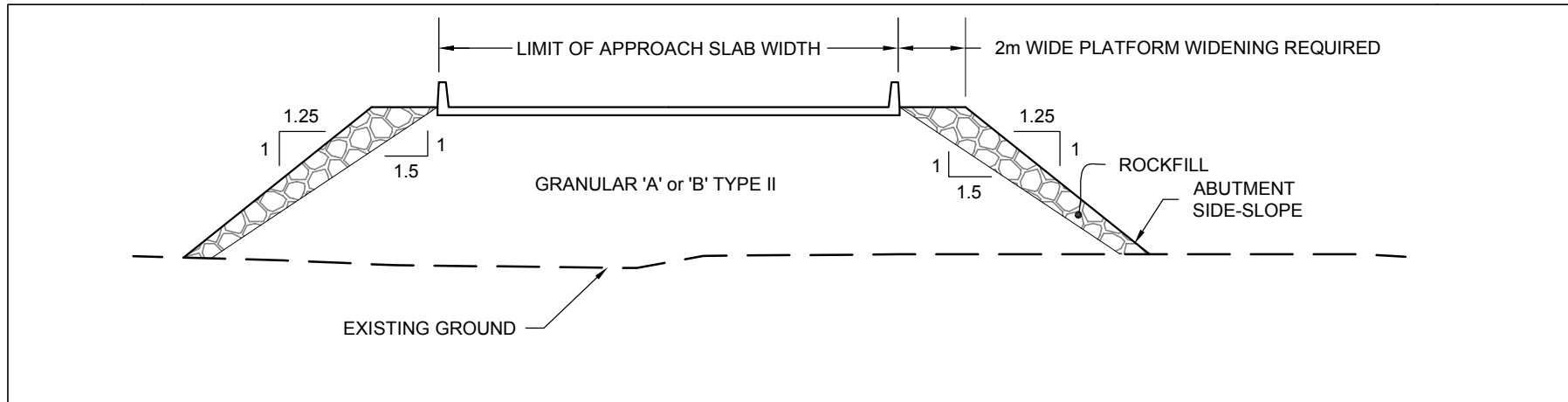


FIGURE 1A - TYPICAL SECTION ACROSS APPROACH SLAB

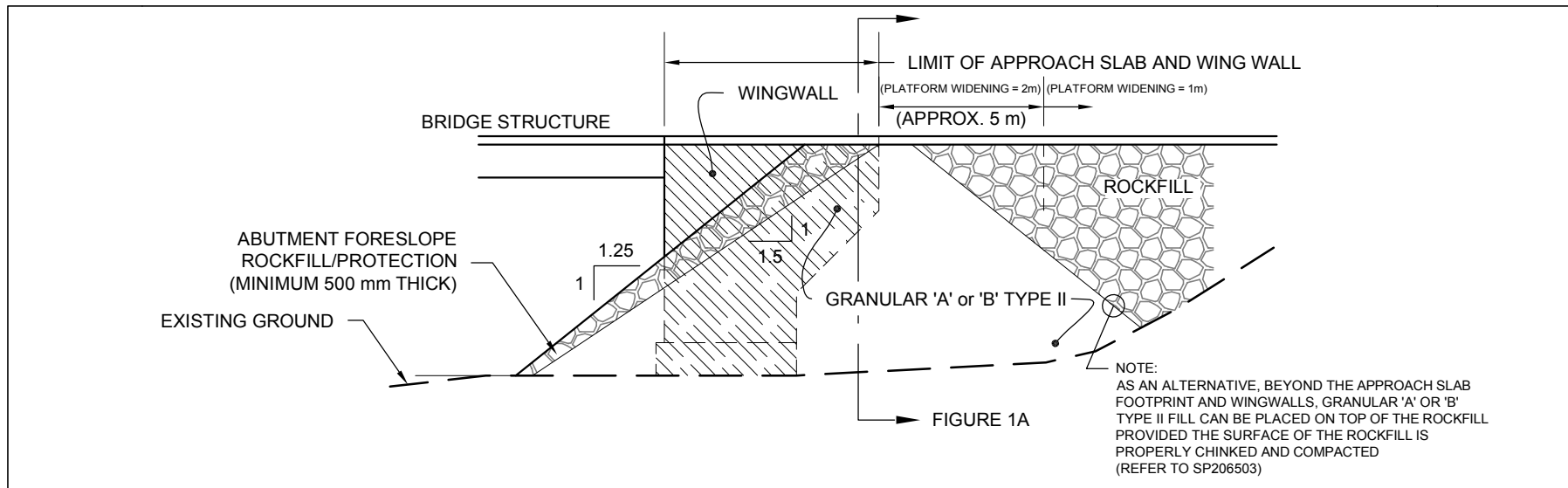


FIGURE 1B - TYPICAL PROFILE SECTION AT EAST ABUTMENT

DATE: JUNE 2006  
 PROJECT: 04-1111-039



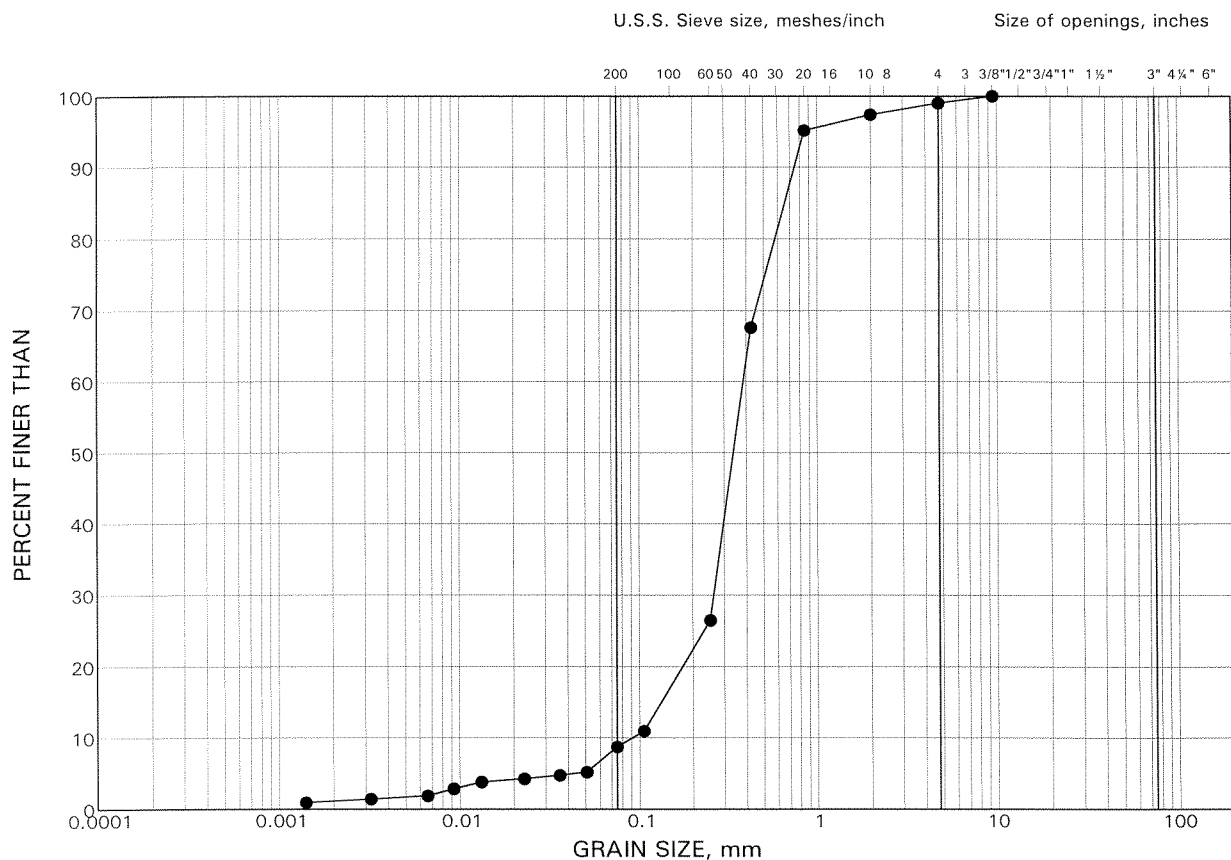
CAD: MSM  
 CHK: KJB

**APPENDIX A**  
**LABORATORY TEST DATA**

# GRAIN SIZE DISTRIBUTION

Sand

FIGURE A1



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

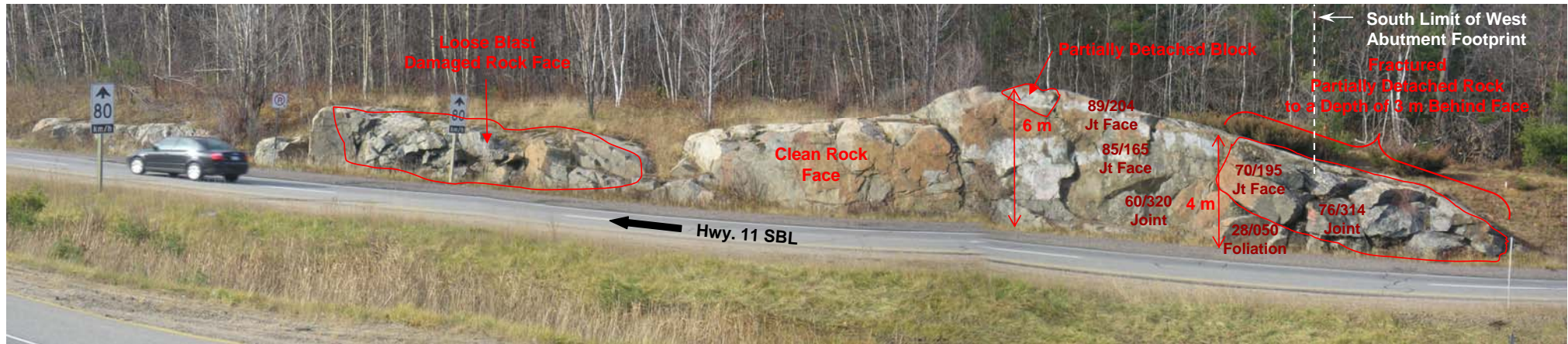
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	04-3	3	259.6

**APPENDIX B**

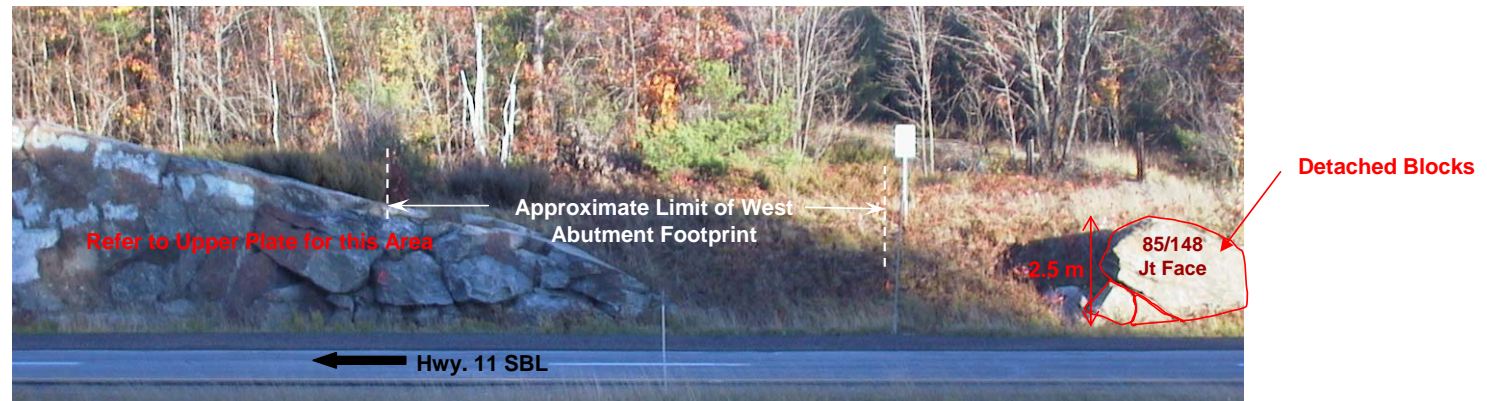
**ROCK HAZARD ASSESSMENT FIGURES**

Construction South

Construction North



South of West Abutment Location

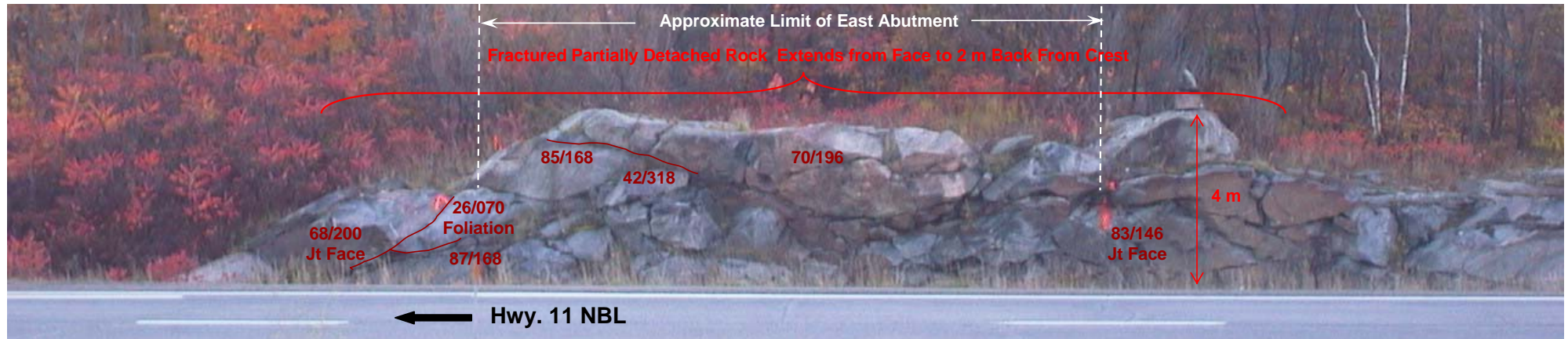


Approximate West Abutment Location



Construction North

Construction South



Proposed East Abutment Location



Area South of East Abutment Location

**APPENDIX C**  
**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 East and West abutment footings and the Central column footing.

#### **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.

<b>Bridge</b>	<b>Foundation</b>	<b>Number of Dowels for Performance Testing</b>
Highway 11 and Pinedale Road/Hewitt Street Underpass	West Abutment	2
Highway 11 and Pinedale Road/Hewitt Street Underpass	Central Pier	1
Highway 11 and Pinedale Road/Hewitt Street Underpass	East 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.

## **DOWELS Into Rock – Item No.**

---

### **Special Provision**

---

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

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.

## **Controlled Blasting at Foundation Locations and at Permanent Rock Cuts – Item No.**

### **Special Provision**

---

#### **Scope of Work**

Work under this item is for the complete removal using wall control blasting techniques from existing rock cuts by appropriate controlled drilling and blasting at locations indicated in the contract and disposal of rock material.

#### **Construction**

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

Drilling equipment shall consist of the following:

A hydraulic track drill capable of drilling the required controlled blasting holes accurately and uniformly across the top of a rock cut, or other suitable equipment, up to 5 m in height.

Removal shall be carried out in such a manner to minimize disturbance to any surrounding rock 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.

Ground and air vibration monitoring is required during the blasting operations. Ground vibration levels should be limited to 50 mm/s for adjacent services and buildings.

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 is required.

## **Controlled Blasting at Foundation Locations and at Permanent Rock Cuts – Item No.**

### **Special Provision**

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.

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 structure are located adjacent to rock cuts, new rock cut faces should be inspected by an independent rock engineering specialist and provisions made for rock bolting, 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 volume or rock in m<sup>3</sup> measured in-place.

### **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.