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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
HIGHWAY 69 / MARSH LAKE ROAD UNDERPASS
HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559
TO 3.5 KM NORTH OF HIGHWAY 559
PARRY SOUND, ONTARIO
G.W.P 335-00-00
MINISTRY OF TRANSPORTATION, ONTARIO**

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation as part of the detailed design of the proposed Highway 69 / Marsh Lake Road underpass structure. The proposed work is part of the detailed design for the four-laning of Highway 69 and re-alignment of Highway 559 north of Nobel, Ontario including the construction of associated new highway on- and off-ramps, access and service roads, bridges and overhead truss sign structures. The general location of the Highway 69 and Highway 559 alignments are shown on the Site Location Map on Figure 1.

The terms of reference for the scope of work are outlined in Golder's proposal P31-1270 dated July 2003 that forms part of the Consultant's Agreement (Number P.O.5005-A-000320) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated October 2003. Two (2) configurations for the Highway 69 / Marsh Lake Road underpass structure were considered for design; the potential footing locations for each of the configurations are shown on Figure 2 based on the drawing provided to Golder by URS on July 30, 2004. A General Arrangement (GA) Drawing for the selected configuration was provided to Golder by URS on September 23, 2005.

This report addresses the investigation completed at the site of the proposed Highway 69 / Marsh Lake Road underpass; for the various structure alternatives and their shared approach embankments. Separate reports detail the foundation investigations carried out for the other aspects of this project; the swamp crossings, high fill areas, other bridge structures and overhead truss sign structures.

The purpose of this investigation is to establish the subsurface conditions at the site by borehole drilling, rock coring, in-situ testing and laboratory testing on selected samples. The boreholes were located in the field by Callon Dietz Incorporated (Callon Dietz), a professional surveying company retained by URS. The location of the investigated area is shown in plan on Drawing 1A.

2.0 SITE DESCRIPTION

The proposed Highway 69 / Marsh Lake Road underpass structure is located east of existing Highway 69 (north of Nobel, Ontario) near the intersection of existing Highway 559, adjacent to existing Marsh Lake Road (as shown on Figure 1). Access to this site was gained via existing ICI Road.

In general, the topography in the area of the overall project site consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamp areas. The proposed Marsh Lake Road underpass site is located within an area where bedrock is at relatively shallow depth or outcrops at ground surface. The area is moderately treed and the ground surface at the proposed structure and approach embankment areas generally lies between Elevations 208 m and 206 m, referenced to Geodetic Datum.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the investigation of the Highway 69 / Marsh Lake Road underpass site was carried out between September 29 and October 15, 2004 during which time a total of twenty-seven (27) explorations consisting of sampled boreholes, shallow hand excavations and probe holes (ML-APP-1, ML1-WA1 to ML1-WA5, ML2-01 to ML2-05, ML1-P1 to ML1-P5, ML2-06 to ML2-10, ML1-EA1 to ML1-EA5 and ML-APP-2) were put down at the site. Twenty-five (25) explorations were advanced at the five footing locations (five per foundation element - including one at each corner of the foundation units and one in the central portion of each foundation unit) – the outer eastern and western abutments (shared by both Alternatives 1 and 2), the central pier (Alternative 1) and the inner eastern and western abutments (Alternative 2). All of the investigated locations were advanced to refusal on inferred bedrock. At each abutment and at the central pier, bedrock coring was carried out at three (3) of the investigated locations to a minimum depth of 3 m.

The field investigation was carried out using a track-mounted CME 55 drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. The boreholes put down with the drill rig were advanced using 108 mm inside diameter (I.D.) continuous flight hollow 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). Where the bedrock surface was inferred to be relatively shallow, hand excavations or probe holes were advanced instead of boreholes in order to confirm the depth to bedrock. Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes were advanced to auger and/or sampler refusal (i.e. inferred bedrock) which occurred at depths ranging from ground surface (i.e. bedrock outcrop) to 1.8 m below the existing ground surface (not including rock coring). At investigated locations ML1-WA1, ML1-WA3, ML1-WA5, ML2-01, ML2-03, ML2-05, ML1-P1, ML1-P3, ML1-P5, ML2-06, ML2-08, ML2-10, ML1-EA1, ML1-EA3 and ML1-EA5, located within the footprints of the proposed foundation units, the boreholes were further advanced into the bedrock by coring about 3.1 m to 7.3 m. The groundwater level in the open boreholes / drillholes was observed throughout drilling operations and piezometers were installed in ML1-WA3 and ML1-EA1 to permit monitoring of the groundwater level at these locations. The piezometers consist of 38 mm O.D. threaded PVC tubing with a slotted screen at depth and were backfilled with a sand filter and sealed with bentonite within the boreholes. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report. All boreholes and piezometers were abandoned in accordance with O.Reg. 128 (amendment to O.Reg. 903). The piezometers were abandoned on January 4, 2006.

The field work was supervised throughout by members of our engineering and technical staff, who confirmed the investigated locations, 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. For a number of drillholes at the site, supervision of rock coring was carried out by a specialist rock engineer. 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 and grain size distribution were carried out on selected samples of the overburden soils. Point load index testing was carried out on specimens of the rock core.

All investigated locations were located in the field by Callon Dietz prior to drilling operations. The surveying of the elevations of the as-drilled boreholes was carried out by members of our engineering staff, referenced to benchmark geodetic elevations provided by URS / Callon Dietz. The borehole locations and ground surface elevations are shown on Drawing 1A.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geology

From published geologic information, the site is located in the physiographic region known as the Georgian Bay Fringe. The Georgian Bay Fringe borders Georgian Bay as 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 and migmatites of the Britt 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, hand excavations and probe holes 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 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 investigated locations.

The inferred soil stratigraphy as encountered at the exploration locations at the Highway 69 / Marsh Lake Road underpass site are shown on Drawings 1A and 1B.

In general, the subsoils at the structure site consist of a surficial layer of topsoil and/or leaf litter underlain by a relatively thin deposit of silt and sand to sand, subsequently underlain by bedrock. The total overburden thickness ranges from no cover (i.e. bedrock outcrops present at ground surface) to about 1.8 m below ground surface. All of the boreholes, hand excavations and probe holes were terminated at the inferred bedrock surface; with the exception of fifteen (15) investigated locations at foundation areas which were cored at least three (3) metres into the bedrock.

In the area of the west approach embankment and outer west abutment (Alternatives 1 and 2), bedrock was encountered at depths ranging from 0.4 m to 1.1 m below the existing ground surface.

In the area of the inner east and west abutments (Alternative 2) and central pier (Alternative 1), bedrock was encountered at depths ranging from 0.4 m to 1.8 m below the existing ground surface.

In the area of the east approach embankment and outer east abutment (Alternatives 1 and 2), bedrock was encountered at depths ranging from 0.0 m (i.e. bedrock outcrop) to 0.3 m below the existing ground surface.

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

4.2.1 Topsoil

A layer of topsoil was encountered at ground surface at all investigated locations where bedrock was not observed to outcrop. The surface of the topsoil (i.e. ground surface) ranged between Elevations 207.6 m and 205.2 m and the thickness ranged between 0.1 m and 0.6 m.

4.2.2 Silt and Sand to Sand

A light brown to brown, occasionally oxidized silt and sand to sand deposit containing cobbles and boulders, trace organics and trace gravel was encountered below the topsoil at the majority of the investigated locations where topsoil did not directly overly bedrock or where bedrock was not observed to outcrop at ground surface. The top of this deposit ranged from about Elevations 207.3 m to 205.0 m and thickness ranged from about 0.1 m to 1.7 m. The bottom of this deposit was defined by refusal to further auger advancement or sampler penetration and was confirmed by rock coring at select locations.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values ranging from 2 blows to greater than 100 blows per 0.3 m of penetration. Lower blow counts were generally measured near ground surface and higher blow counts may be attributed to cobbles and boulders in the deposit, particularly at or near the interface of the bottom of the deposit and the bedrock surface. The 'N' values indicate a very loose to very dense relative density within the deposit.

The natural water content measured on samples of this deposit ranged between 4 percent and 18 percent, with an average of about 9 percent.

Grain size distributions for three (3) samples from this deposit are shown on Figure A-1 of Appendix A.

4.2.3 Bedrock

Bedrock was encountered and cored at investigated locations ML1-WA1, ML1-WA3, ML1-WA5, ML2-01, ML2-03, ML2-05, ML1-P1, ML1-P3, ML1-P5, ML2-06, ML2-08, ML2-10, ML1-EA1, ML1-EA3 and ML1-EA5. The presence of bedrock was confirmed by hand excavations or inferred from refusal to further drilling or sampler advancement or probe holes at the remaining investigated locations. The surface of the bedrock varies from ground surface (i.e. bedrock outcrop) to a depth of about 1.8 m. At the investigated locations, the bedrock surface ranges between about Elevations 207.5 m and 204.2 m.

The bedrock samples are described as fresh to slightly weathered, light grey to pink, medium to coarse grained, non-porous to faintly porous granitic gneiss containing near horizontal, distinct foliation. The Total Core Recovery measured on the core samples was between 72 percent and 100 percent. The Rock Quality Designation (RQD) measured on the core samples of the upper 0.4 m of the bedrock is variable, ranging from 0 percent (ML1-WA5) to 100 percent, indicating a very poor to excellent quality. Below the upper 0.4 m of bedrock, the RQD generally ranges from about 67 to 100 percent, but is typically greater than 80 percent, indicating a rock mass of good to excellent quality.

Axial and diametral point load strength tests were performed on samples of the rock core. Diametral point load strength index values are shown on the Record of Drillhole Sheets. Axial point load strength index values ranged from 2.5 MPa to 10.3 MPa, typically greater than 6 MPa, and diametral point load strength index values ranged from 1.9 MPa to 6.4 MPa, typically greater than 3 MPa, indicating a strong to very strong rock mass. A summary of the point load index values on the rock core from the fifteen (15) investigated locations 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 along with the associated approximate Unconfined Compressive Strength (UCS) value for each test.

Borehole (Drillhole) No.	Average Axial Point Load Index (MPa)	Average Diametral Point Load Index (MPa)
ML1-WA1	6.7	3.5
ML1-WA3	7.5	4.5
ML1-WA5	7.8	4.5
ML2-01	8.0	4.8
ML2-03	7.7	5.4
ML2-05	9.3	4.9

Borehole (Drillhole) No.	Average Axial Point Load Index (MPa)	Average Diametral Point Load Index (MPa)
ML1-P1	7.9	5.0
ML1-P3	7.6	4.6
ML1-P5	7.5	4.6
ML2-06	7.5	4.5
ML2-08	5.3	5.3
ML2-10	8.0	5.2
ML1-EA1	7.6	4.4
ML1-EA3	6.6	4.8
ML1-EA5	7.9	4.3

4.2.4 Groundwater Conditions

In general, the samples taken in the overburden in the boreholes were noted to be dry to moist. The groundwater level in the piezometer installed in the bedrock of ML1-WA3 was measured at Elevation 204.0 m (3.4 m depth) on November 14, 2004 and at Elevation 203.3 m (4.1 m depth) on January 4, 2006. The groundwater level in the piezometer installed in the bedrock of ML1-EA1 was measured at Elevation 202.4 (3.7 m depth) on November 14, 2004 and January 4, 2006. Groundwater levels measured during drilling operations were noted to range from about Elevation 204.2 m to 202.4 m (1.9 m to 3.7 m depth). Details of the piezometer installations, groundwater conditions and water levels observed in the open boreholes / drillholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

4.3 CLOSURE

This Foundation Investigation Report was prepared by Mr. Chad Gilfillan and reviewed by Ms Anne Poschmann, P.Eng., a Principal with Golder Associates. Mr. Fintan Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

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PART B

FOUNDATION DESIGN
HIGHWAY 69 / MARSH LAKE ROAD UNDERPASS
HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559
TO 3.5 KM NORTH OF HIGHWAY 559
PARRY SOUND, ONTARIO
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5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of design of the Highway 69 / Marsh Lake Road underpass structure. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes, hand excavations and probe holes advanced during the subsurface investigation.

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 two (2) design alternatives were being considered for the proposed Highway 69 / Marsh Lake Road underpass structure:

- Alternative 1: A two-span bridge with about 51.5 m span lengths and abutments located east and west of the proposed four-lane (NBL and SBL) Highway 69 and a pier located in the median (rock removed from the median).

- Alternative 2: Two single span bridges with lengths of about 45 m (NBL) and 43 m (SBL) with the rock left in the median and the median abutments perched on the rock.

Following completion of the field investigation work and review of the rock core and point load index testing results, the feasibility of the above two alternatives was assessed. Given the inferred good to excellent quality of the rock mass (based on RQD), the strong to very strong nature of the bedrock and the predominantly horizontal nature of the joints/fractures encountered during coring, Alternative 2 was recommended from a foundations perspective. This recommendation was based on the suitability of the bedrock for the support of shallow abutment foundations within the median and consideration of the potential difficulties associated with rock removal in this area.

After the selection of Alternative 2, two additional refinements to the design were considered that required foundations input. The first refinement considered the use of either a square or parallel abutment design. The parallel abutment configuration offered the advantage of ensuring that all abutment footings would be located outside the typically recommended 0.5H:1V imaginary setback line drawn extending back from the toe of the rock cuts. The square abutment configuration would result in one corner of two of the abutments encroaching as much as 0.75 m within the set-back line. However, given the high quality of the bedrock, the encroachment in these two areas was considered acceptable at this site so long as rock dowels are installed prior to excavation along the crest of the cut (to pre-support the rock face and control over-break) and careful and controlled blasting techniques (to minimize over-break) are adopted. Based on these foundation recommendations, the square abutment configuration was selected.

The second and final refinement considered two different top of pavement profiles for Marsh Lake Road, relative to the proposed Highway 69 grade, which would affect the required amount of horizontal bedrock excavation in the median area, the final rock cut heights and the extent of footing encroachment within the imaginary 0.5H:1V set-back line. Considering the strong to very strong nature of the rock mass, the profile of Marsh Lake Road was selected so as to minimize the amount of horizontal bedrock excavation required in the median area that could be potentially detrimental to the quality of the bedrock on which the footings would be constructed. Although the extent of the footing encroachment within the 0.5H:1V set-back line would be greater with this option, given the high quality of the bedrock at this site, the drawbacks associated with the encroachments were considered to be more than offset by the benefits of minimizing the amount of potentially detrimental horizontal rock excavation.

Based on the information provided on the preliminary bridge alternative drawings provided on July 30, 2004 and the GA Drawing of the selected alternative (Alternative 2) provided on September 28, 2005, the grade of the proposed Marsh Lake Road bridge deck varies between about Elevation 211.4 m at the outer east abutment location and Elevation 208.4 m at the outer west abutment location. The new Highway 69 top of pavement grades at the structure site are proposed at about Elevation 200.4 m for the northbound (NBL) alignment and about Elevation 200.6 for the southbound (SBL) alignment. The height above the existing ground/bedrock surface for the proposed approach embankments is up to about 5 m at the east side of the bridge and 2 m at the west side of the bridge. The existing ground surface varies from about Elevation 207.6 m to 205.2 m at the investigated locations.

The proposed Highway 69 in this area will be constructed in cut (into the bedrock) and it is understood that this excavation will be carried out prior to construction of the proposed structure. The recommendations given in the following sections have taken this into account as it pertains to foundation design and construction, excavation, drainage and other considerations.

5.2 Bridge Foundation Options

The native soils at the bridge site consist of topsoil overlying a thin (typically less than 1.8 m deep) deposit of very loose to very dense silt and sand to sand containing cobbles and boulders. The thin native overburden soils are underlain by strong to very strong granitic gneiss bedrock. The bedrock surface at the proposed foundations, as established at the borehole locations, ranges from about Elevation 206.9 m to 205.0 m at the outer west abutment; about Elevation 204.8 m to 204.2 m at the inner west abutment; about Elevation 205.2 m to 204.4 m at the central pier; about Elevation 205.6 m to 204.9 m at the inner east abutment; and about Elevation 207.6 m to 206.2 m at the outer east abutment.

Given the shallow and variable nature of the overburden, these soils are not considered suitable for the support of shallow spread footings. The underlying granitic gneiss bedrock is suitable for the support of the proposed abutments and pier on shallow foundations.

Due to the shallow nature of the overburden deposits at the site, it is understood that integral abutments are not being considered at this location. For integral abutments, a minimum pile length of about 5 m is generally required for support of the abutments which, at this site, would require significant excavation/trenching into the very strong bedrock and would likely be cost prohibitive. Instead, the following foundation options could be considered:

- Shallow spread footings placed on/within the bedrock; or
- Perched abutments, founded on spread footings placed on well compacted granular pads within the approach embankment fill.

For the latter option, the proposed grade of Marsh Lake Road (Elevation 208.4 m at the outer west abutment and Elevation 211.4 m at the outer east abutment) would result in the use of a relatively thin granular pad, probably varying from 0 m to 2 m in thickness. Recommendations for spread footings (founded on bedrock or perched on granular pads) for the foundation units of both alternatives are presented in the following sections. A summary of the advantages, disadvantages, relative costs and risks/consequences for the foundation alternatives is given in Table 2 following the text of this report.

5.3 Spread Footings

The bridge abutments for both structure alternatives and pier for Alternative 1 may be supported on spread footings placed on the properly prepared granitic gneiss bedrock. The details of the bedrock surface elevation as encountered in the boreholes at the different foundation elements is summarized in the following table.

Foundation Element	*Borehole Numbers*	*Depth to Bedrock**	*Bedrock Surface Elevation*
Outer east abutment *(Alternative 1 and 2)*	ML1-EA1 to ML1-EA5	0.1 m to 0.3 m	206.2 m to 207.6 m
Inner east abutment *(Alternative 2)*	ML2-06 to ML2-10	0.8 m to 1.1 m	204.9 m to 205.6 m
Central pier *(Alternative1)*	ML1-P1 to ML1-P5	0.4 m to 1.7 m	204.4 m to 205.2 m
Inner west abutment *(Alternative 2)*	ML2-01 to ML2-05	0.9 m to 1.8 m	204.2 m to 204.8 m
Outer west abutment *(Alternative 1 and 2)*	ML1-WA1 to ML1-WA5	0.4 m to 1.1 m	205.0 m to 206.9 m

* depth below existing ground surface

Based on the proposed Highway 69 NBL and SBL top of pavement grades (Elevation 200.4 m and 200.6 m), the central pier (Alternative 1) would be founded well below the bedrock surface. It is anticipated that the bedrock at or below this elevation would be of good quality assuming that proper excavation/blasting techniques are utilized for removing the excess rock (as discussed in Section 5.8).

For the abutments of the two alternatives, the options for spread footings on the bedrock are to either maintain the footing relatively high and allow for some mass concrete placement or to assume a lower founding level that will require bedrock excavation. Based on the borehole results, the bedrock surface within the limits of each proposed abutment is variable with about 1 m to 2 m of variation. In addition, the upper portion of the bedrock is, in a few local areas, of very poor quality (i.e. RQD values as low as 0 percent and 22 percent as encountered in boreholes ML1-WA1, ML1-WA5 and ML2-05) and it may be necessary to subexcavate loose or fractured rock from within some areas of the foundation footprints. For design, the following options for founding levels of the abutments may be considered:

1. The following foundation elevations may be assumed for design:

Outer East Abutment (Alternatives 1 and 2):	Elevation 207.8 m
Inner East Abutment (Alternative 2):	Elevation 205.8 m
Inner West Abutment (Alternative 2):	Elevation 205 m
Outer West Abutment (Alternatives 1 and 2):	Elevation 207 m

   In this case, following the removal of the overburden, the bedrock surface would have to be cleaned and then mass concrete would be placed to raise the grade to the founding level. A Non-Standard Special Provision (NSSP) should be made in the Contract Documents for additional mass concrete placement to accommodate variations in the bedrock surface (an example is provided in Appendix B). The benefit of this approach is that excavation into the strong to very strong bedrock is minimized. Where horizontal bedrock excavation is locally required, all loose or fractured rock should be removed from the foundation footprint and replaced with mass concrete. All horizontal excavation should be performed prior to the vertical excavation / rock cuts required for the Highway 69 NBL and SBL.

2. Alternatively, the following design founding levels may be assumed:

Outer East Abutment (Alternatives 1 and 2):	Elevation 206 m
Inner East Abutment (Alternative 2):	Elevation 204.3 m
Inner West Abutment (Alternative 2):	Elevation 203.7 m
Outer West Abutment (Alternatives 1 and 2):	Elevation 204.4 m

In this case, following the removal of the overburden, excavation of the upper portion of the bedrock will be required within the foundation footprints. Based on the borehole results, subexcavation of up to about 2.5 m will be required. This depth of excavation is recommended so that the footing is founded below the very poor quality bedrock that was encountered in a few localized areas. Should the bridge configuration require lower founding elevations than those noted above, the bedrock at depth would be of good quality assuming that proper excavation/blasting techniques are utilized for removing the excess rock. It is noted that the bedrock is classified as strong to very strong (i.e. estimated unconfined compressive strengths typically in the range of 50 MPa to 150 MPa) and the level of fracturing in the upper portion 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 (refer to Section 5.8 for bedrock excavation-blasting recommendations).

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 in order to minimize shattering and over-break. This recommendation is particularly important in areas where horizontal bedrock excavation may be locally necessary to reach the required founding level (such as at the outer east abutment footing). Additional recommendations on bedrock excavation are provided in Section 5.8.

It should be noted that footing excavations to expose the founding bedrock surfaces may, in some places, extend below the anticipated groundwater level (generally 1.9 m to 3.7 m below the existing ground surface). Groundwater control measures (as discussed in Section 5.7.2) may be locally required to maintain dry and stable excavations especially during periods of high groundwater levels.

The abutment footings will be situated (perched) on the bedrock above the adjacent Highway 69 road grade. The footings must be maintained an adequate distance away from the edge of the rock cut and the rock face adequately cleaned and/or protected such that the integrity of the rock face/founding rock is maintained. In this regard, the abutment footing should be located away from the rock face at least a distance as defined by an imaginary line projected at 0.5 horizontal to 1 vertical from the toe of the rock cut. If the layout does not allow for this setback zone, a NSSP should be made for vertical rock dowels to be installed behind the crest of the cut prior to excavation in order to control and pre-support the rock face. An example is provided in Appendix B.

In all areas where mass concreting is to be employed, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the footprint of the footings to ensure a proper bond to the bedrock. A provision should be included in the Contract Documents to address the requirements for field inspection. In order to carry out this inspection, the excavation should be dry. 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).

As an alternative to supporting the abutment footings on bedrock (or mass concrete), consideration could be given to the use of abutment footings perched within the approach embankments. This option would require that the spread footing be founded on a well compacted granular fill pad (i.e. not founded on rock fill) and that the overburden soils are removed prior to placing the granular fill.

The simplest option for the abutment footings, from a foundation perspective, is spread footings placed on the bedrock surface or on mass concrete placed on the bedrock surface which should minimize bedrock excavation difficulties. If the perched abutment option was to be used, consideration must be given to how variable the granular pad may be and whether there is a concern for the use of a combination of different foundation types at the same structure. The cost effectiveness of each of the foundation options should be considered in the design.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the surface of the properly prepared granitic 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 axial resistance at ULS, since the granitic gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

For spread footings placed (or perched) within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. Depending on the thickness of the granular pad, it may be feasible to achieve a higher bearing resistance. The geotechnical resistance at SLS (for 25 mm of settlement) will depend on the thickness of the Granular 'A' pad; a value of 350 kPa may be assumed for preliminary design. If this "perched" abutment option is adopted for the design of the foundation at the east abutment, these resistances would have to be confirmed once the elevation and location of the abutment footing is known.

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 granitic 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 δ, may be taken as 0.70 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 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 bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted for resistance to sliding at this site, a NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels (an example is provided in Appendix B).

If "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

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

For "perched" abutments, all footings should be provided with a minimum of 1.8 m of soil cover for frost protection. Where rock fill is employed as a cover material, the minimum cover thickness required will be approximately twice that of a conventional soil cover give the open nature of the rock fill structure. Alternatively, rigid insulation could be used to reduce the required thickness of soil cover over the foundation units. For preliminary design, it can be assumed that 25 mm of rigid insulation is equivalent to 0.6 m of conventional soil cover. The insulation should be installed on the abutment stem extending down from ground surface to the top of the footing and then extend to a distance of 1.8 m beyond the perimeter of each foundation unit.

5.4 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 (refer to Section and Figure 6.9.1 (e) of the *CHBDC*).

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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.

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northeastern Region Directive for backfill to structures adjacent to rock embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3505.00.

- 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 OPSS 501.06. 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 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(l) 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 (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC)*.

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

	SSM	Rock Fill
Soil unit weight:	20 kN/m^3	19 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.35	0.24
At rest, K_o	0.50	0.38

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

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

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.9.1(a) of the *Commentary to the CHBDC*.

A restrained structure is typically concrete box culverts or rigid frame bridge structures where the rotational and/or horizontal movement is not sufficient to mobilize the active pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and/or 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 for Parry Sound is 0.05. Based on experience, for the thin overburden soils at this site, a 10 to 20 percent amplification factor of the ground motion could 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 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 $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

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

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

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

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

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

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

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

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

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

5.5 Approach Embankment Design

Both alternatives for the Highway 69 / Marsh Lake Road underpass structure will require placement of up to about 5 m of fill for the east approach embankment and up to about 2 m of fill for the west approach embankment (assuming that the existing bedrock surface is maintained within the footprint of the approach embankments).

Based on the investigated locations at this site, the approach embankments will be founded on either bedrock or a thin deposit (typically less than 1.8 m deep) of very loose to very dense silt and sand to sand. All topsoil and organic matter should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement.

Where the abutment embankments will be close to the excavated rock cut faces (i.e. rock cuts made for the Highway 69 SBL and NBL), the minimum setback from the crest of the rock cut to the toe of the embankment should be a minimum of 1.5 m. Again, good quality controlled blasting methods, under the guidance of a blasting specialist, will be critical in order to maintain the excavation lines and preserve the integrity of the rock mass.

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

5.5.1 Stability

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

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W (Version 5.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used in the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this sites considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries.

At the east approach area, bedrock is either outcropping or at very shallow depth. The thin overburden soils present will largely be removed as part of the excavation required to remove the surficial organics from the footprint of the embankment. As such, the east approach has been assumed to be founded on bedrock for the purposes of stability analyses.

At the west approach area and the central area between the two bridges (median approach embankment), the very loose to compact cohesionless subsoils are up to about 1 m and 1.7 m thick, respectively. 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). The correlations proposed by Peck et al. (1974), Schmertmann (1975) and US Navy (1971) were employed and the results were 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 measured in piezometers installed in ML1-WA3 and ML1-EA1 and noted during drilling of the boreholes. In general, the groundwater level is located between 1.9 m and 3.7 m below the existing ground surface.

The following table summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach areas. It is understood that consideration is being given to the use of earth fill or rock fill for the construction of the approach embankments, and as indicated in the table below, both fill types were considered in the analysis. 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 5.5.1.1.

Soil Type	**Unit Weight (kN/m^3)**	**Strength Parameters**
Rock Fill	19	$c' = 0$ kPa, $\phi' = 38^o$
Earth Fill (Sand and Gravel)	21	$c' = 0$ kPa, $\phi' = 35^o$
Very loose to compact Silty Sand	20	$c' = 0$ kPa, $\phi' = 30^o$

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 trial failure surface that would impact the operation of the roadway.

Location	*Embankment Height at Critical Section (m)**	*Earth Fill Option*		*Rock Fill Option*	
		Recommended Side Slope Profile	*Minimum Factor of Safety*	*Recommended Side Slope Profile*	*Minimum Factor of Safety*
East Approach	5	2H : 1V	> 1.3	1.25H : 1V	> 1.3
Median Approach	4.5				
West Approach	2				

Note : *assuming that the existing bedrock surface is maintained within the footprint of the approach embankments

5.5.1.1 Embankment Fill Types and Berm Requirements

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.

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.

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 overall project site, 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 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 6 m such that the uninterrupted rock fill slope never exceeds a height of 6 m (as per MTO Northeastern Region guidelines). We understand that the Northeastern Region requirements for berms have recently changed from 6 m to 10 m height. However, we have been instructed to maintain the original guidelines (i.e. for berms at 6 m height) for this project.

5.5.2 Liquefaction Potential

The liquefaction potential of the soils below the west and east approach embankments and median approach embankment (if centre pier option is not chosen) under seismic loading has been considered. Given that the boreholes were generally dry upon completion of drilling and the fact that the proposed Highway 69 grade will be more than 5 m below the base of the bedrock at the west, east, and median embankment locations (i.e. allowing for gravity drain of any perched water), it is considered that the native silt and sand to silty sand soils will generally remain unsaturated. Since liquefaction requires the development of excess pore pressures in saturated soils, the site is not considered to be susceptible to liquefaction.

Total seismic settlements within the native dry sandy soils are calculated to be less than 5 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 0.3 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.

5.5.3 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 5 m at the east approach, 4.5 m at the median approach and 2 m at the west approach (up to about 5 m near the outer west abutment with proposed rock cut as shown on Drawing 1A). The unit weights and slope profiles for the embankment fill described in Section 5.5.1 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 approach embankment area, bedrock is either outcropping or at very shallow depth and the thin overburden soils will largely be removed as part of the excavation required for removing the surficial organics. As such, the east approach embankment will be founded primarily on bedrock. At the west approach and median approach areas, the very loose to compact cohesionless subsoils are about 1 m and 1.7 m thick, respectively, underlain by bedrock. Surficial deposits of topsoil were encountered at the majority of the investigated locations.

Provided that the surficial topsoil is removed prior to the new embankment fill placement (as discussed in Section 5.6.1), settlements of the new approach embankments, due to compression of the foundation soils, are expected to be minimal. 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.

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

5.5.3.1 Settlement of Cohesionless Foundation Soils

The immediate compression of the very loose to compact silt and sand to sand subsoils encountered in the boreholes in the area of the west approach and median approach 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 results of the new embankment construction in the area of the approaches.

Location of Embankment	*Approximate Chainage*	*Maximum New Embankment Height* (m)*	*Estimated Settlement of Foundation Soils (mm)*
West Approach	9+920 to *9+930*	2	10
West Approach	*9+930* to 9+940	5	---**
Median Approach	9+985 to 10+000	4.5	15
East Approach	10+045 to 10+065	5	---***

Notes : *includes additional fill required after removal of maximum depth of organics/topsoil
** no foundation soils in this area due to proposed cut into bedrock (see Drawing 1A)
***minimal foundation soils in this area 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.

5.5.3.2 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 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, SP 206S03 dated January 2004, the settlement of the newly placed rock fill is expected to be small. In general, it is estimated that for the granitic gneiss rock fill likely to be used at this site, for the up to 5 m high approach embankments, the settlement of the rock fill will be about 1% of the new effective height of rock fill.

Location of Embankment	***Approximate Chainage***	***Maximum New Embankment Height* (m)***	***Estimated Settlement of Embankment Rock Fill (mm)***
West Approach	9+920 to *9+930*	2	20
West Approach	*9+930* to 9+940	5	50
Median Approach	9+985 to 10+000	4.5	45
East Approach	10+045 to 10+065	5	50

Notes : *includes additional fill required after removal of maximum depth of organics/topsoil

It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction. If rock fill is used, consideration should be given to delaying the final paving for about 1 year to allow the majority of the settlement to take place.

5.6 Subgrade Preparation and Embankment Construction

The existing native subsoils are considered to be 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 subgrade soils should be proof-rolled.

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

5.6.1 Removal of Organics

Based on the information from the borings obtained during the field investigation, organic deposits (i.e. topsoil and leaf litter) of up to about 0.6 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.

5.6.2 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 Special Provision 206S03 (January 2004) – Section 206.07.07, 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.

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

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 Special Provision 206S03 (January 2004) – Section 206.07.08. 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.

5.7 Design and Construction Considerations

5.7.1 Excavation

As noted in Section 5.3, excavations for the construction of spread footings for the bridge abutments and pier and/or as part of the cutting required to establish the grade for the NBL and SBL of the new Highway 69 will extend up to about 2 m through the cohesionless subsoils and up to about 7 m into the underlying bedrock. In addition, as noted in Section 5.6, excavation within the plan limits of the approach embankments will be required in order to remove topsoil / organic deposits about 0.2 m deep (up to about 0.6 m deep locally) prior to fill placement.

If space permits, temporary excavations (i.e. those that are open only for a relatively short period) through the native soils may be made with side slopes no steeper than about 1.5H:1V. Temporary excavations within the bedrock may be made with near vertical cut.

It is noted that the bedrock is classified as strong to very strong (i.e. estimated unconfined compressive strengths in the range of 50 MPa to 150 MPa). This will make 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 would likely have to 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.

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.

5.7.2 Groundwater and Surface Water Control

The groundwater level at the site is generally at about 1.9 m to 3.7 m below the existing ground surface and therefore excavations to expose the founding bedrock surface for spread footings may require groundwater control. A dry and stable excavation will be required to permit placement of mass concrete and construction of footings in the dry.

It is likely that open-cut excavations with sufficient pumping and/or controlled drainage will adequately manage the groundwater. Surface water should be directed away from the excavations at all times.

5.7.3 Obstructions

The native subsoils at the site are expected to contain cobbles and boulders as was inferred from grinding of augers during borehole advancement (particularly at or near the bedrock surface).

Conventional excavation equipment should be suitable for the majority of excavation through the on-site soils. However, the presence of the boulders may interfere with or slow the progress of stripping and excavation.

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. 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 blast damage to the rock mass. Line drilling should also be utilized to minimize rock shatter and overbreak.

5.8.2 Special Provisions

5.8.2.1 Blasting

Good blasting practises will be critical to maintaining the excavation lines and preserving the integrity of the rock mass in the area of the structure foundations. It is recommended that the Contractor retain a blast engineer and submit proposed blast plans for review at least 3 weeks in advance of rock excavation.

Abutment footings should be located away from the rock face at least a distance as defined by an imaginary line projected at 0.5 horizontal to 1 vertical from the toe of the rock cut. Where the layout does not allow for this setback zone, an NSSP should be included in the contract for vertical rock dowels to be installed behind the crest of the cut prior to excavation in order to control and pre-support the rock face. An example NSSP is provided in Appendix B.

The use of explosives shall follow the general specifications outlined in OPSS 120 and the *Guidelines for Safe Blasting in Ontario Highway Construction Operations, ORBA October 2001* should be followed. It is recommended that a separate Special Provision for the control of all blasting operations be prepared (refer to SP 299F06). The Special Provision 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.

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. It is also recommended that all new rock cut faces in the area of the proposed structure foundations be inspected by a Quality Verification Engineer to assess if the blasting operations have affected the integrity of the rock mass that will ultimately be supporting the new footings. A provision for rock bolting, if necessary, should be included in the Contract Specifications in the event that additional support is required in these areas.

5.9 CLOSURE

This Foundation Design Report was prepared by Mr. Chad Gilfillan, E.I.T. and technical aspects were reviewed by Ms. Anne S. Poschmann, P.Eng., a Principal with Golder. Technical support was provided by Dr. J. Paul Dittrich, Ph.D., P.Eng., and Mr. Mark Telesnicki, P.Eng., both Associates with Golder. Mr. Fintan J. Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

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REFERENCES

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

Canadian Foundation Engineering Manual. 1992. Third Edition. Canadian Geotechnical Society, Technical Committee on Foundations.

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

Chapman, L.J. and Putnam, D.F. 1984. The Physiography of Southern Ontario, 3rd Edition (Ontario Geological Survey, Special Volume 2). Ontario Ministry of Natural Resources.

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

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

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

Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, 2nd Edition, John Wiley and Sons, New York.

Schmertmann, J.H. 1975. Measurement of In-Situ Shear Strength. In Proceedings, ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Vol. 2, Raleigh, pp. 57-138.

Tokimatsu, K., and Seed, H. 1987. Evaluation of Settlements in Sands Due to Earthquake Shaking. ASCE Journal of Geotechnical Engineering, V.113, N.8.

U.S. Navy. 1971. Soil Mechanics, Foundations and Earth Structures. NAVFAC Design Manual DM-7, Washington, D.C.

RECORD OF BOREHOLE SHEETS

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

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

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

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING 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	

Form...G.A.-R-3

PROJECT 03-1111-028	
W.P. 335-00-00	LOCATION N 5032715.6 ;E 256878.8
DIST 52 HWY 69	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATUM Geodetic	DATE September 29, 2004

RECORD OF BOREHOLE No ML-APP-1

 METRIC

ORIGINATED BY EHS
COMPILED BY DD
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
207.0	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty sand, trace gravel, trace clay Loose to dense Brown Moist		1	SS	5						
			2	SS	22/0.15 50/0.0						
206.1 0.9	End of Borehole Spoon refusal Auger refusal Note: 1. Open borehole dry upon completion of drilling.										

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

Golder Associates

RECORD OF BOREHOLE No ML-APP-2

 METRIC

PROJECT 03-1111-028

W.P. 335-00-00 | LOCATION N 5032845.1 ;E 256837.2 | ORIGINATED BY EHS

DIST 52 HWY 69 | BOREHOLE TYPE Hand excavated | COMPILED BY DD

DATUM Geodetic | DATE September 29, 2004 | CHECKED BY CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.9	GROUND SURFACE										
0.0	Topsoil										
	Sandy silt										
0.3	End of Borehole										
	Hand excavated to bedrock.						206				
	Note:										
	1. Open borehole dry upon completion of excavation.										

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

Golder Associates

RECORD OF BOREHOLE No ML2-01

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00 LOCATION N 5032769.0 ;E 256854.8 ORIGINATED BY KC
DIST 52 HWY 69 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY DD
DATUM Geodetic DATE October 14, 2004 CHECKED BY CG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.2	GROUND SURFACE								
0.0 / 0.1	Topsoil					205			
	Silty sand, trace gravel, trace clay Loose to compact Brown Moist	1	AS				o		
204.2 / 1.0	Bedrock Refer to Record of Drillhole ML2-01 for details				∇	204, 203, 202, 201, 200, 199, 198			
197.6 / 7.6	End of Borehole Notes: 1.) Auger refusal at 1.0m depth. 2.) Water level in open borehole at 1.5m depth upon completion of drilling.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

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

RECORD OF DRILLHOLE: ML2-01

PROJECT: 03-1111-028

SHEET 1 OF 1

LOCATION: N 5032769.0 ;E 256854.8

DRILLING DATE: Oct. 14, 2004

DATUM: Geodetic

DRILL RIG: TRACK CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
		- continued from Record of Borehole -	204.21		
1	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	0.99	1	Fx,, Fx,,
2				2	JN,, Fx,, JN,, Fx,, JN,,
3				3	Fx,, Fx,, JN,, Fx,,
5				4	JN,, Fx,, JN,,
7			197.58	5	Fx spun,, Fx,, Fx,,
		END OF DRILLHOLE	7.62		

Column headings: PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3 m; B Angle; DIP w.r.t. CORE AXIS; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.; NOTES WATER LEVELS INSTRUMENTATION

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: KC

CHECKED: CG

MIS-RCK 002 03111028AARCK.GPJ GAL-MISS.GDT 23/6/06 JFC

Golder Associates

RECORD OF BOREHOLE No ML2-02

 METRIC

PROJECT 03-1111-028

W.P. 335-00-00 LOCATION N 5032770.5 ;E 256854.3 ORIGINATED BY EHS

DIST 52 HWY 69 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY DD

DATUM Geodetic DATE October 1, 2004 CHECKED BY CG

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER (SAMPLES)	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.2	GROUND SURFACE										
0.0	Topsoil						205				
0.2	Silty Sand, trace gravel, trace clay Compact Light brown Moist		1	SS	23				o		
204.3 0.9	End of Borehole Auger refusal Note: 1. Open borehole dry upon completion of drilling										

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

Golder Associates

RECORD OF BOREHOLE No ML2-03

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032774.5 ;E 256866.7
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY DD
DATUM Geodetic
DATE September 30, 2004
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.5	GROUND SURFACE								
0.0	Topsoil								
0.2	Silty Sand, trace gravel, trace clay Compact Oxidized	1	SS	15		206			
205.9 0.6	Sand, trace gravel, trace silt, trace clay Very Dense Light brown Moist Oxidized	2	SS	52		205	○		
		3	SS	50/0	∇				
204.8 1.7	Bedrock Refer to Record of Drillhole ML2-03 for details					204, 203, 202			
201.5 5.1	End of Borehole Notes: 1. Auger and spoon refusal at 1.7 m depth 2. Water level in open borehole at 1.8 m depth upon completion of drilling								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

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

RECORD OF DRILLHOLE: ML2-03

PROJECT: 03-1111-028
LOCATION: N 5032774.5 ;E 256866.7
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Sept. 30, 2004
DRILL RIG: TRACK CME 55
DRILLING CONTRACTOR: MARATHON DRILLING LTD.

SHEET 1 OF 1
DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	204.71			∇
2	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	1.79	1	FO,IR, FO,IR, JN,IR, FO,IR,	
3				2	MB,, FO,IR, MB,, JN,IR, FO,IR, MB,, FO,IR,	
4				3	MB,, JN,IR, FO,IR, JN,IR, FO,IR, JN,IR, JN,IR, JN,IR,	
5		END OF DRILLHOLE	201.45 5.05			

Depth scale: 6, 7, 8, 9, 10, 11

MIS-RCK 002 03111028AARCK.GPJ GAL-MISS.GDT 23/6/06 JFC

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No ML2-04

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032773.0 ;E 256867.2
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY DD
DATUM Geodetic
DATE September 30, 2004
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.6	GROUND SURFACE						
0.0	Topsoil						
	Silt, some clay, trace sand, trace organics Compact	1	SS	22		o	
0.5	Silty Sand, trace to some gravel, trace clay, contains cobbles and boulders Very dense	2	SS	62	206		
205.1	Light brown, oxidized Moist	3	SS	10/0.07 50/0.00	205	o	20 68 11 1
1.8	Sand, trace to some silt, some gravel, trace clay Dense Brown Moist						
	End of Borehole Auger refusal Spoon refusal						

Note:

1. Open borehole dry upon completion of drilling

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L
WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³
STRAT PLOT | GROUND WATER CONDITIONS

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

Golder Associates

RECORD OF BOREHOLE No ML2-05

 METRIC

PROJECT	03-1111-028		
W.P.	335-00-00	LOCATION	N 5032771.7 ;E 256860.8
DIST	52 HWY 69	BOREHOLE TYPE	Power Auger 108 mm I.D. Hollow Stem Auger
DATUM	Geodetic	DATE	October 14, 2004

ORIGINATED BY KC
COMPILED BY DD
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
205.9	GROUND SURFACE					
0.0	Topsoil					
0.2	Silty Sand, trace gravel, trace clay Loose to compact Brown and grey Moist	1	AS			
		2	AS		205	o
204.7 1.2	Bedrock Refer to Record of Drillhole ML2-05 for details				204 203 202 201 200 199	
198.3 7.6	End of Borehole Notes: 1.) Auger refusal at 1.2m depth.					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML2-05

LOCATION: N 5032771.7 ;E 256860.8

DRILLING DATE: Oct. 14, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

Column headings: DEPTH SCALE METRES; DRILLING RECORD; DESCRIPTION; SYMBOLIC LOG; ELEV. DEPTH (m); RUN No.; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3 m; DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.; NOTES WATER LEVELS INSTRUMENTATION

Depth scale (m)	Drilling record	Description	Elev. / Depth (m)	Run No.	Discontinuity type and surface description
		- continued from Record of Borehole -			
	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	204.68 1.22	1	JN,, JN,, JN,, JN,,
2				2	
3					JN,, JN,, JN,,
4				3	Fx,,
5				4	JN,,
6					Fx spun,,
7				5	Fx,, JN,,
		END OF DRILLHOLE	198.30 7.60		
8					
9					
10					
11					

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: KC
CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No ML2-06

 METRIC

PROJECT 03-1111-028			
W.P. 335-00-00	LOCATION N 503278.3 ;E 256850.8		ORIGINATED BY EHS
DIST 52	HWY 69	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY DD
DATUM Geodetic	DATE October 14, 2004		CHECKED BY CG

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.8	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty Sand, trace clay Loose to compact Brown Moist		1	AS							
204.9 0.9	Bedrock Refer to Record of Drillhole ML2-06 for details					∇ (at 204)	205 204 203 202				
201.2 4.6	End of Borehole Notes: 1.) Auger refusal at 0.9m depth. 2.) Water level in open borehole at 1.9m depth upon completion of drilling.										

MIS-MTO 001 031111028AAMTO.GPJ GAL-MISS.GDT 23/6/06

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML2-06

SHEET 1 OF 1

LOCATION: N 503278.3 ;E 256850.6

DRILLING DATE: Oct. 14, 2004

DATUM: Geodetic

DRILL RIG: TRACK CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	204.94			
0.86–4.60	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	0.86	1	JIR,, JIR,, JIR,, JIR,, JIR,, JIR,,	
				2	FOIR,, MB,, FOIR,, MB,, MB,,	∇
				3	MB,, JIR,, MB,, MB,,	
		END OF DRILLHOLE	201.20 4.60			

Other column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3 m; B Angle; DIP w.r.t. CORE AXIS; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

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

MIS-RCK 002 03111028AARCK.GPJ GAL-MISS.GDT 23/6/06 JFC

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No ML2-07

 METRIC

PROJECT 03-1111-028

W.P. 335-00-00 | LOCATION N 5032783.8 ;E 256850.1 | ORIGINATED BY EHS

DIST 52 HWY 69 | BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger | COMPILED BY DD

DATUM Geodetic | DATE October 1, 2004 | CHECKED BY CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.9	GROUND SURFACE										
0.0	Topsoil, some sand		1	SS	2						
205.4											
205.1	Sandy Silt, trace organics Dark brown Moist						205				
0.8	End of Borehole Auger refusal Note: 1. Open borehole dry upon completion of drilling										

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

Golder Associates

RECORD OF BOREHOLE No ML2-08

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032787.8 ;E 256862.4
ORIGINATED BY KC
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY DD
DATUM Geodetic
DATE October 13, 2004
CHECKED BY CG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.6	GROUND SURFACE										
0.0	Topsoil										
0.1	Silty Sand, trace gravel, trace clay Loose to compact Brown Moist		1	AS			206				
			2	AS					○		
205.6 1.0	Bedrock Refer to Record of Drillhole ML2-08 for details						205				
							204				
							203				
						∇	202				
							201				
							200				
199.0 7.6	End of Borehole Notes: 1.) Auger refusal at 1.0m depth. 2.) Water level in open borehole at 4.6m depth upon completion of drilling.						199				

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML2-08

SHEET 1 OF 1

LOCATION: N 5032787.8 ;E 256862.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Oct. 13, 2004
DRILL RIG: TRACK CME 55
DRILLING CONTRACTOR: MARATHON DRILLING LTD.

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	205.61			
1	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	0.99	1	Fx,,	
2				2	JN,,	
3				3	JN spun,, FX,, JN,, FX,,	
4					JN,,	∇
5				4	FX,,	
6						
7				5	FX,, FX,,	
		END OF DRILLHOLE	199.00 7.60			
8						
9						
10						

Column headings also include: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY TOTAL CORE % (80 60 40 20); SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 m (5 10 15 20); B Angle (0 90 180 270); DIP w.r.t. CORE AXIS (0 30 60 90); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

DEPTH SCALE
1 : 50

LOGGED: KC
CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No ML2-09

 METRIC

PROJECT 03-1111-028

W.P. 335-00-00 LOCATION N 5032786.3 ;E 256862.9 ORIGINATED BY EHS

DIST 52 HWY 69 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY DD

DATUM Geodetic DATE October 1, 2004 CHECKED BY CG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.7	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty Sand, trace to some clay Compact to very dense Brown Dry to moist		1	SS	23		206				
205.7			2	SS	33/.13 50/.0				o		
	Weathered Bedrock										
1.1	End of Borehole Auger refusal Spoon refusal										

Note:

1. Open borehole dry upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No ML2-10

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032785.1 ;E 256856.5
ORIGINATED BY KC
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY DD
DATUM Geodetic
DATE October 13, 2004
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L – WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.4	GROUND SURFACE										
0.0 / 0.1	Topsoil										
	Silty Sand, trace gravel, trace clay Loose to compact Brown Dry to moist		1	AS			206				
205.6 / 0.8			2	AS					o		
	Bedrock Refer to Record of Drillhole ML2-10 for details						205				
							204				
						∇	203				
							202				
							201				
							200				
198.8							199				
7.6	End of Borehole Notes: 1.) Auger refusal at 0.8m depth. 2.) Water level in open borehole at 3.7m depth upon completion of drilling.										

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML2-10

SHEET 1 OF 1

LOCATION: N 5032785.1 ;E 256856.5

DRILLING DATE: Oct. 13, 2004

DATUM: Geodetic

DRILL RIG: TRACK CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -				
	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	205.61 0.79	1		
2				2	FX spun,, FX,,	
3				3	FX spun,, FX,, JN,, FX,,	▽
5				4	FX spun,, JN,,	
6–7				5	JN,, JN,,	
		END OF DRILLHOLE	198.78 7.62			

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: KC

CHECKED: CG

MIS-RCK 002 03111028AARCK.GPJ GAL-MISS.GDT 23/6/06 JFC

Golder Associates

RECORD OF BOREHOLE No ML1-P1

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032775.7 ;E 256856.9
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Hand excavated
COMPILED BY DD
DATUM Geodetic
DATE September 30, 2004
CHECKED BY CG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.4	GROUND SURFACE										
0.0	Topsoil										
0.4	Silty Sand, trace gravel, trace clay Brown Dry						205				
	Bedrock Refer to Record of Drillhole ML1-P1 for details						204				
						∇	203				
201.8							202				
3.6	End of Borehole Notes: 1.) Hand excavated to bedrock. 2.) Water level in open borehole at 2.7m depth upon completion of excavation.										

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML1-P1

SHEET 1 OF 1

LOCATION: N 5032775.7 ;E 256856.9

DRILLING DATE: Sept. 30, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		205.04			
	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey		0.36		,SM,	
1					1	FO,UN,SM ,, JN,RO, JN,RO, IR,RO, ,, MB,,	
2					2	MB,,	
3					3	MB,,	
		END OF DRILLHOLE		201.76 3.64			Water level in open borehole at 2.7 m depth upon completion of drilling
4							
5							
6							
7							
8							
9							
10							

Other column headings: PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 m (5 10 15 20); B Angle (0 90 180 270); DIP w.r.t. CORE AXIS (0 30 60 90); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No ML1-P2

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032780.5 ;E 256855.3
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY DD
DATUM Geodetic
DATE September 29, 2004
CHECKED BY CG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.2	GROUND SURFACE										
0.0	Topsoil and wood fragments		1	SS	3		206				
205.6 0.6	Sand, trace silt, trace gravel, trace clay Compact to dense Brown Moist		2	SS	15/0.1 50/0.0				o		
205.2 1.0	End of Borehole Auger Refusal Spoon Refusal										

Note:

1. Open borehole dry upon completion of drilling

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

Golder Associates

RECORD OF BOREHOLE No ML1-P3

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032782.0 ;E 256860.1
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY DD
DATUM Geodetic
DATE October 1, 2004
CHECKED BY CG

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
206.3	GROUND SURFACE											
0.0	Topsoil											
0.2	Silty Sand to Sand, trace to some gravel, trace clay, containing cobbles and boulders Compact to very dense Brown, oxidized Moist	1	SS	21		206						
		2	SS	105		205	o		12	62	23	3
204.7		3	SS	10/0.07 50/0								
1.6	Bedrock Refer to Record of Drillhole ML1-P3 for details				∇	204 203 202						
201.6 4.7	End of Borehole Notes: 1. Auger refusal at 1.6 m depth 2. Water level in open borehole at 2.2 m depth upon completion of drilling											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML1-P3

SHEET 1 OF 1

LOCATION: N 5032782.0 ;E 256860.1

DRILLING DATE: Oct. 1, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	204.62			
2	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	1.68	1	JN,RO, JN,RO, JN,IR, FO,UN,RO	
3				2	MB,, JN,IR, JN,IR, JN,IR, JN,IR, JN,IR,	
4				3	MB,, MB,, JN,IR,	
			201.56			
		END OF DRILLHOLE	4.74		MB,,	Water level in open borehole at 2.2 m depth upon completion of drilling

Column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 m (5 10 15 20); DISCONTINUITY DATA: B Angle (0 90 180 270), DIP w.r.t. CORE AXIS (0 30 60 90); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

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

MIS-RCK 002 03111028AARCK.GPJ GAL-MISS.GDT 23/6/06 JFC

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

RECORD OF BOREHOLE No ML1-P4

 METRIC

PROJECT 03-1111-028

W.P. 335-00-00 LOCATION N 5032777.3 ;E 256861.6 ORIGINATED BY EHS

DIST 52 HWY 69 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY DD

DATUM Geodetic DATE October 1, 2004 CHECKED BY CG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.0	GROUND SURFACE										
0.0	Topsoil										
0.2	Silty Sand, trace clay Compact Light brown Dry to moist		1	SS	13				o		
205.1 0.9	Sand, some gravel, trace silt, trace clay Dense Brown Moist		2	SS	45		205				
204.4 1.7	End of Borehole Auger refusal Spoon refusal		3	SS	50/0				o		

Note:

1. Open borehole dry upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No ML1-P5

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032778.9 ;E 256858.5
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY DD
DATUM Geodetic
DATE October 1, 2004
CHECKED BY CG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.1	GROUND SURFACE					206			
0.0	Topsoil	1	SS	19					
205.8 0.3	Silty Sand, trace gravel, trace clay Compact to very dense Light brown Oxidized Moist	2	SS	33/0.07 50/0			o		
205.2 1.0	Sand, some gravel, trace clay Brown Moist					205			
	Bedrock Refer to Record of Drillhole ML1-P5 for details				∇	204 203			
201.9 4.2	End of Borehole Notes: 1. Auger and spoon refusal at 1.0 m depth 2. Water level in open borehole at 2.0 m depth upon completion of drilling					202			

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML1-P5

SHEET 1 OF 1

LOCATION: N 5032778.9 ;E 256858.5

DRILLING DATE: Oct. 1, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -				
	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	205.06 1.04	1	JN,IR, JN,IR, MB,,	
2				2	JN,IR, JN,IR, JN,IR, JN,IR, JN,IR, JN,IR, JN,IR, MB,,	▽
3				3	MB,, MB,, JN,IR, JN,UN,RO FO,IR, FO,IR,	
4		END OF DRILLHOLE	201.92 4.18			Water level in open borehole at 1.9 m depth upon completion of drilling
5						
6						
7						
8						
9						
10						
11						

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

RECORD OF BOREHOLE No ML1-EA1

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00 LOCATION N 5032828.0 ;E 256835.9 ORIGINATED BY EHS
DIST 52 HWY 69 BOREHOLE TYPE Hand excavated COMPILED BY DD
DATUM Geodetic DATE October 12, 2004 CHECKED BY CG

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.4	GROUND SURFACE										
0.0 0.1	Topsoil and Silty Sand, trace gravel, trace clay Grey/Brown Dry to moist						206				
	Bedrock Refer to Record of Drillhole ML1-EA1 for details						205 204 203 202 201 200				
199.8 6.6	End of Borehole Notes: 1.) Hand excavated to bedrock. 2.) Water level in piezometer measured at 3.7 m depth on November 14, 2004.										

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML1-EA1

LOCATION: N 5032828.0 ;E 256835.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Oct. 12, 2004

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

DATUM: Geodetic

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

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

Columns: DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | PENETRATION RATE (m/min) | FLUSH COLOUR % RETURN | RECOVERY (TOTAL CORE %, SOLID CORE %) | R.Q.D. % | FRACT. INDEX PER 0.3 m | DISCONTINUITY DATA (B Angle; DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION) | HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}) | Diametral Point Load Index (MPa) | RMC -Q' AVG. | NOTES WATER LEVELS INSTRUMENTATION

Drilling record	Description	Elev. / Depth (m)	Run No.
	- continued from Record of Borehole -	206.32	
NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	0.08	1, 2, 3, 4, 5
	END OF DRILLHOLE	199.77 6.63	

Discontinuity type and surface description (top to bottom): Fx,, ; Fx,, ; Fx,, ; Fx,, ; Fx,, ; JN,, ; JN,, ; Fx,, ; Fx,, ; Fx,, ; Fx,, ; JN,, ; JN,, ; Fx,, ; JN,, ; JN,, ; JN,, ; JN,, ; Fx spun,,

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

Notes / Water levels / Instrumentation:

Bentonite

Sand

Screen

Water level measured at 3.7 m on Nov. 14, 2004

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: KC

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No ML1-EA2

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032828.0 ;E 256835.9
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Hand excavated
COMPILED BY DD
DATUM Geodetic
DATE October 12, 2004
CHECKED BY CG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
206.4	GROUND SURFACE										
0.0	Topsoil										
0.2	Hand excavated to bedrock						206				

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

Golder Associates

RECORD OF BOREHOLE No ML1-EA3

 METRIC

PROJECT	03-1111-028				
W.P.	335-00-00	LOCATION	N 5032833.5 ;E 256847.8	ORIGINATED BY	EHS
DIST	52 HWY 69	BOREHOLE TYPE	Hand excavated	COMPILED BY	DD
DATUM	Geodetic	DATE	October 12, 2004	CHECKED BY	CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
207.6	GROUND SURFACE										
0.0	Topsoil										
0.0	Bedrock Refer to Record of Drillhole ML1-EA3 for details						207 206 205				
204.2 3.4	End of Borehole Note: 1.) Hand excavated to bedrock.										

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

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML1-EA3

LOCATION: N 5032833.5 ;E 256847.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Oct. 2004

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	207.57			
0	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	0.03	1	JIR,, JIR,, JIR,, JIR,, JUR,, JUR,, JIR,, JIR,,	
				2	JIR,, MB,, JUR,, JIR,, FOIR,, MB,, FOPLR,, FOUR,, MB,, MB,, MB,,	
		END OF DRILLHOLE	204.24 3.36			

Other column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %) 80 60 40 20; R.Q.D. % 80 60 40 20; FRACT. INDEX PER 0.3 m 5 10 15 20; B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.

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

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No ML1-EA4

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00 LOCATION N 5032833.5 ;E 256847.8 ORIGINATED BY EHS
DIST 52 HWY 69 BOREHOLE TYPE Hand excavated COMPILED BY DD
DATUM Geodetic DATE October, 2004 CHECKED BY CG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
207.4	GROUND SURFACE										
0.0	Topsoil										
0.1	Hand excavated to bedrock						207				

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

MIS-MTO 001 031111028AAMTO.GPJ GAL-MISS.GDT 23/6/06

RECORD OF BOREHOLE No ML1-EA5

METRIC

PROJECT 03-1111-028			
W.P. 335-00-00	LOCATION N 5032830.8 ;E 256841.8		ORIGINATED BY EHS
DIST 52	HWY 69	BOREHOLE TYPE Hand excavated	COMPILED BY DD
DATUM Geodetic	DATE October 12, 2004		CHECKED BY CG

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L – WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.9	GROUND SURFACE										
0.0	Topsoil										
0.3	Silty Sand, trace gravel, trace clay Grey/Brown Dry to moist										
	Bedrock Refer to Record of Drillhole ML1-EA5 for details						206				
							205				
						∇					
							204				
							203				
							202				
							201				
							200				
199.3 7.6	End of Borehole Notes: 1.) Hand excavated to bedrock. 2.) Water level in open borehole at 2.3m depth upon completion of excavation.										

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

RECORD OF DRILLHOLE: ML1-EA5

PROJECT: 03-1111-028
SHEET 1 OF 1
LOCATION: N 5032830.8 ;E 256841.8
DATUM: Geodetic
INCLINATION: -90° AZIMUTH: ---
DRILLING DATE: Oct. 12, 2004
DRILL RIG: TRACK CME 55
DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	206.57			
0–7.62	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	206.57 0.00	1	Fx,, Fx,, JN,, Fx spun,,	
				2	Fx,, Fx,,	∇
				3	Fx,, JN,, Fx,, JN,, Fx,, JN,, JN,, JN,, Fx,,	
				4	Fx,,	
			199.28	5	Fx,, Fx,, Fx,,	
		END OF DRILLHOLE	7.62			

Other columns: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3 m; B Angle; DIP w.r.t. CORE AXIS; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: KC
CHECKED: CG

PROJECT 03-1111-028

RECORD OF BOREHOLE No ML1-WA1

 METRIC

W.P. 335-00-00 LOCATION N 5032727.1 ;E 256868.3 ORIGINATED BY EHS

DIST 52 HWY 69 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY DD

DATUM Geodetic DATE September 29, 2004 CHECKED BY CG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
205.6	GROUND SURFACE					
0.0	Topsoil					
0.1	Silty Sand, trace clay Dark brown Moist	1	SS	42/0.15		
205.1 0.5	Highly fractured, moderately weathered BEDROCK					205
204.4 1.2	0.6m to 1.2m depth advanced by NW casing. Bedrock Refer to Record of Drillhole ML1-WA1 for details					204
						203
					∇	
						202
201.0 4.6	End of Borehole Notes: 1.) Spoon refusal at 0.6m depth. 2.) Water level in open borehole at 3.2m depth upon completion of drilling.					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF DRILLHOLE: ML1-WA1

PROJECT: 03-1111-028

SHEET 1 OF 1

LOCATION: N 5032727.1 ;E 256868.3

DRILLING DATE: Sept. 29, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	204.43		
	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	1.17	FO,UN,SM FO,UN,Ro MB,, FO,PL,Ro FO,UN,Ro FO,PL,Ro FO,,IR FO,PL,Ro FO,UN,Ro JN,IR, FO,UN,Ro FO,UN,Ro ,, JN,IR, FO,IR, ,IR, FO,,Ro FO,,Ro FO,,Ro JN,IR,Ro FO,PL,Ro	∇
		END OF DRILLHOLE	201.03 4.57		Water level in open borehole at 3.2 m depth below ground surface upon completion of drilling

Other column headings: SYMBOLIC LOG; RUN No.; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %), 80 60 40 20; R.Q.D. %, 80 60 40 20; FRACT. INDEX PER 0.3 m, 5 10 15 20; B Angle, 0 90 180 270; DIP w.r.t. CORE AXIS, 0 30 60 90; HYDRAULIC CONDUCTIVITY K, cm/sec, 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa), 2 4 6; RMC -Q' AVG.

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

MIS-RCK 002 03111028AARCK.GPJ GAL-MISS.GDT 23/6/06 JFC

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No ML1-WA2

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00
LOCATION N 5032728.6 ;E 256867.8
ORIGINATED BY EHS
DIST 52 HWY 69
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY DD
DATUM Geodetic
DATE September 29, 2004
CHECKED BY CG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.8	GROUND SURFACE										
0.0	Topsoil										
0.2	Sand, some silt, trace clay, trace gravel Loose Brown, oxidized Moist		1	SS	4				o		
205.0 0.8	End of Borehole Auger refusal						205				

Note:

1. Open borehole dry upon completion of drilling

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

MIS-MTO 001 031111028AAMTO.GPJ GAL-MISS.GDT 23/6/06

Golder Associates

RECORD OF BOREHOLE No ML1-WA3 1 OF 1 **METRIC**

PROJECT: 03-1111-028			
W.P.: 335-00-00	LOCATION: N 5032732.6 ;E 256880.2	ORIGINATED BY: KC	
DIST: 52 HWY: 69	BOREHOLE TYPE: Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY: DD	
DATUM: Geodetic	DATE: October 15, 2004	CHECKED BY: CG	

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
207.4	GROUND SURFACE										
0.0	Topsoil										
0.1	Silty Sand, trace clay, trace gravel Loose to compact Grey/Brown Dry to moist		1	AS			207		o		
206.9 0.5	Bedrock Refer to Record of Drillhole ML1-WA3 for details						206				
							205				
						▼	204				
							203				
							202				
							201				
							200				
199.8 7.6	End of Borehole										

Notes:

1.) Auger refusal at 0.5m depth.

2.) Water level in piezometer measured at 3.4m depth on November 14, 2004

.

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

RECORD OF DRILLHOLE: ML1-WA3

PROJECT: 03-1111-028

SHEET 1 OF 1

LOCATION: N 5032732.6 ;E 256880.2

DRILLING DATE: Oct. 15, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -	206.87			
	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	0.53	1	JN,, Fx,, JN,,	
2				2	JN,, Fx,, JN,, Fx,, VN,, JN,, JN,,	Bentonite
3–4				3	JN,, JN,, JN,,	Sand
5				4	Fx,, Fx,,	
6–7				5	JN,, Fx spun,, JN,, Fx,, Fx,,	Screen
		END OF DRILLHOLE	199.78 7.62			Water level measured at 3.4 m below ground surface on Nov. 14, 2004

Column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3 m; B Angle; DIP w.r.t. CORE AXIS; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

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

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: KC

CHECKED: CG

Golder Associates

RECORD OF BOREHOLE No ML1-WA4

METRIC

PROJECT 03-1111-028			
W.P. 335-00-00	LOCATION N 5032731.1 ;E 256880.7		ORIGINATED BY EHS
DIST 52	HWY 69	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY DD
DATUM Geodetic	DATE September 29, 2004		CHECKED BY CG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
207.4	GROUND SURFACE										
0.0	Topsoil										
0.2	Sand, some silt, trace gravel, trace clay Compact Brown Moist		1	SS	15		207		○		
206.7 / 0.7	End of Borehole Auger refusal										

Note:

1. Open borehole dry upon completion of drilling

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

Golder Associates

RECORD OF BOREHOLE No ML1-WA5

 METRIC

PROJECT 03-1111-028
W.P. 335-00-00 LOCATION N 5032729.8 ;E 256874.2 ORIGINATED BY KC
DIST 52 HWY 69 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY DD
DATUM Geodetic DATE October 15, 2004 CHECKED BY CG

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.9	GROUND SURFACE						
0.0	Topsoil				▽		
0.1	Silt and Sand to Silty Sand, trace clay, trace gravel Loose to compact Grey/Brown Wet	1	AS				
		2	AS			206	4 48 45 3
205.8 / 1.1	Bedrock Refer to Record of Drillhole ML1-WA5 for details					205, 204, 203, 202, 201, 200	
199.3 / 7.6	End of Borehole Notes: 1.) Auger refusal at 1.1m depth. 2.) Water level in open borehole at 0.1m depth upon completion of drilling.						

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

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

MIS-MTO 001 031111028AAMTO.GPJ GAL-MISS.GDT 23/6/06

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: ML1-WA5

SHEET 1 OF 1

LOCATION: N 5032729.8 ;E 256874.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Oct. 15, 2004

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -				
	NQ CORING	GRANITIC GNEISS Medium to coarse grained Fresh to slightly weathered Faintly porous Light grey to pinkish grey	205.76 1.14	1	JN,,	
					Fx,,	
					Fx,,	
2				2	Fx,,	
					Fx,,	
					JN,,	
3				3	JN,,	
					Fx,,	
4					Fx,,	
5				4	JN,,	
					Fx,,	
					JN,,	
					JN,,	
					JN,,	
6				5		
7					Fx,,	
					Fx,,	
					Fx,,	Refer to Record of Borehole ML1-WA5
		END OF DRILLHOLE	199.44 7.46			
8						
9						
10						
11						

Column headings also include: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %) 80 60 40 20; R.Q.D. % 80 60 40 20; FRACT. INDEX PER 0.3 m 5 10 15 20; B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: KC

CHECKED: CG

TABLES

TABLE 1 - SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO. 03-1111-028
LOCATION: Proposed Highway 69 / Marsh Lake Road Underpass
DATE: January 07, 2005

Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type	**Is (50mm)** (MPa)	**Approx. UCS[1]** (Is_{50}x23)(MPa)
ML1-WA1	1	Granitic Gneiss	4.5	1.4	D	4.615	106
ML1-WA1	2	Granitic Gneiss	6.8	2.1	A	7.950	183
ML1-WA1	3	Granitic Gneiss	8.5	2.6	D	1.933	44
ML1-WA1	4	Granitic Gneiss	9.7	2.9	A	5.362	123
ML1-WA1	5	Granitic Gneiss	10.9	3.3	D	5.467	126
ML1-WA1	6	Granitic Gneiss	12.3	3.7	D	2.001	46
ML1-WA3	1	Granitic Gneiss	2.3	0.7	D	4.752	109
ML1-WA3	2	Granitic Gneiss	4.7	1.4	A	7.440	171
ML1-WA3	3	Granitic Gneiss	8.7	2.7	D	5.476	126
ML1-WA3	4	Granitic Gneiss	12.8	3.9	A	7.646	176
ML1-WA3	5	Granitic Gneiss	16.4	5.0	D	3.900	90
ML1-WA3	6	Granitic Gneiss	19.2	5.8	D	4.675	108
ML1-WA3	7	Granitic Gneiss	22.8	7.0	D	3.517	81
ML1-WA5	1	Granitic Gneiss	4.5	1.4	D	1.899	44
ML1-WA5	2	Granitic Gneiss	6.4	1.9	A	7.574	174
ML1-WA5	3	Granitic Gneiss	8.2	2.5	D	5.126	118
ML1-WA5	4	Granitic Gneiss	12.5	3.8	A	8.040	185
ML1-WA5	5	Granitic Gneiss	18.6	5.7	D	4.735	109
ML1-WA5	6	Granitic Gneiss	20.9	6.4	D	5.484	126
ML1-WA5	7	Granitic Gneiss	23.9	7.3	D	5.373	124
ML2-01	1	Granitic Gneiss	3.9	1.2	D	4.471	103
ML2-01	2	Granitic Gneiss	6.7	2.0	A	8.340	192
ML2-01	3	Granitic Gneiss	9.8	3.0	D	4.896	113
ML2-01	4	Granitic Gneiss	12.6	3.8	D	5.084	117
ML2-01	5	Granitic Gneiss	13.5	4.1	A	7.702	177
ML2-01	6	Granitic Gneiss	17.6	5.4	D	4.598	106
ML2-01	7	Granitic Gneiss	22.3	6.8	D	4.913	113
ML2-03	1	Granitic Gneiss	6.4	2.0	D	5.118	118
ML2-03	2	Granitic Gneiss	7.3	2.2	A	8.330	192
ML2-03	3	Granitic Gneiss	9.4	2.9	D	5.484	126
ML2-03	4	Granitic Gneiss	11.2	3.4	A	7.068	163
ML2-03	5	Granitic Gneiss	12.7	3.9	D	5.884	135
ML2-03	6	Granitic Gneiss	16.4	5.0	D	5.016	115
ML2-05	1	Granitic Gneiss	5.2	1.6	D	4.573	105
ML2-05	2	Granitic Gneiss	7.6	2.3	A	10.300	237
ML2-05	3	Granitic Gneiss	10.4	3.2	D	5.297	122
ML2-05	4	Granitic Gneiss	13.5	4.1	A	8.293	191
ML2-05	5	Granitic Gneiss	18.6	5.7	D	4.547	105
ML2-05	6	Granitic Gneiss	20.7	6.3	D	4.931	113
ML2-05	7	Granitic Gneiss	24.4	7.4	D	4.956	114

TABLE 1 - SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO. 03-1111-028
LOCATION: Proposed Highway 69 / Marsh Lake Road Underpass
DATE: January 07, 2005

Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type	**Is (50mm)** (MPa)	**Approx. UCS[1]** (Is_{50}x23)(MPa)
ML2-06	1	Granitic Gneiss	3.4	1.0	D	3.134	72
ML2-06	2	Granitic Gneiss	5.9	1.8	A	7.643	176
ML2-06	3	Granitic Gneiss	7.3	2.2	D	3.841	88
ML2-06	4	Granitic Gneiss	9.9	3.0	A	7.366	169
ML2-06	5	Granitic Gneiss	10.8	3.3	D	4.948	114
ML2-06	6	Granitic Gneiss	14.4	4.4	D	6.089	140
ML2-08	1	Granitic Gneiss	3.5	1.1	D	3.798	87
ML2-08	2	Granitic Gneiss	5.6	1.7	A	8.146	187
ML2-08	3	Granitic Gneiss	9.3	2.8	D	5.561	128
ML2-08	4	Granitic Gneiss	12.7	3.9	A	2.515	58
ML2-08	5	Granitic Gneiss	16.5	5.0	D	5.305	122
ML2-08	6	Granitic Gneiss	19.4	5.9	D	5.305	122
ML2-08	7	Granitic Gneiss	23.7	7.2	D	6.378	147
ML2-10	1	Granitic Gneiss	4.1	1.2	D	4.368	100
ML2-10	2	Granitic Gneiss	7.5	2.3	A	7.618	175
ML2-10	3	Granitic Gneiss	10.5	3.2	D	6.361	146
ML2-10	4	Granitic Gneiss	12.3	3.7	A	8.457	195
ML2-10	5	Granitic Gneiss	16.7	5.1	D	5.092	117
ML2-10	6	Granitic Gneiss	19.9	6.1	D	4.649	107
ML2-10	7	Granitic Gneiss	23.6	7.2	D	5.365	123
ML1-P1	1	Granitic Gneiss	1.4	0.4	D	4.752	109
ML1-P1	2	Granitic Gneiss	2.6	0.8	A	7.877	181
ML1-P1	3	Granitic Gneiss	5.0	1.5	D	5.518	127
ML1-P1	4	Granitic Gneiss	7.9	2.4	A	7.994	184
ML1-P1	5	Granitic Gneiss	8.8	2.7	D	5.331	123
ML1-P1	6	Granitic Gneiss	11.3	3.4	D	4.522	104
ML1-P3	1	Granitic Gneiss	5.7	1.7	D	4.019	92
ML1-P3	2	Granitic Gneiss	7.8	2.4	A	7.459	172
ML1-P3	3	Granitic Gneiss	9.7	2.9	D	4.803	110
ML1-P3	4	Granitic Gneiss	11.4	3.5	A	7.792	179
ML1-P3	5	Granitic Gneiss	12.9	3.9	D	5.791	133
ML1-P3	6	Granitic Gneiss	15.4	4.7	D	3.764	87
ML1-P5	1	Granitic Gneiss	3.6	1.1	D	3.977	91
ML1-P5	2	Granitic Gneiss	4.9	1.5	A	6.955	160
ML1-P5	3	Granitic Gneiss	6.6	2.0	D	5.033	116
ML1-P5	4	Granitic Gneiss	9.9	3.0	A	8.041	185
ML1-P5	5	Granitic Gneiss	10.5	3.2	D	5.441	125
ML1-P5	6	Granitic Gneiss	12.6	3.8	D	3.892	90

TABLE 1 - SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO. 03-1111-028
LOCATION: Proposed Highway 69 / Marsh Lake Road Underpass
DATE: January 07, 2005

Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type	**Is (50mm)** (MPa)	**Approx. UCS[1]** (Is_{50}x23)(MPa)
ML1-EA1	1	Granitic Gneiss	0.5	0.1	D	4.948	114
ML1-EA1	2	Granitic Gneiss	3.4	1.0	A	7.347	169
ML1-EA1	3	Granitic Gneiss	4.6	1.4	D	4.701	108
ML1-EA1	4	Granitic Gneiss	9.1	2.8	A	7.785	179
ML1-EA1	5	Granitic Gneiss	12.7	3.9	D	4.667	107
ML1-EA1	6	Granitic Gneiss	15.6	4.8	D	3.704	85
ML1-EA1	7	Granitic Gneiss	21.5	6.6	D	3.934	90
ML1-EA3	1	Granitic Gneiss	0.4	0.1	D	3.542	81
ML1-EA3	2	Granitic Gneiss	2.8	0.8	A	7.161	165
ML1-EA3	3	Granitic Gneiss	3.7	1.1	D	4.624	106
ML1-EA3	4	Granitic Gneiss	5.7	1.7	A	6.000	138
ML1-EA3	5	Granitic Gneiss	7.6	2.3	D	5.203	120
ML1-EA3	6	Granitic Gneiss	9.9	3.0	D	5.731	132
ML1-EA5	1	Granitic Gneiss	1.5	0.5	D	4.896	113
ML1-EA5	2	Granitic Gneiss	3.8	1.1	A	7.927	182
ML1-EA5	3	Granitic Gneiss	7.3	2.2	D	4.581	105
ML1-EA5	4	Granitic Gneiss	10.4	3.2	A	7.867	181
ML1-EA5	5	Granitic Gneiss	14.3	4.4	D	3.900	90
ML1-EA5	6	Granitic Gneiss	17.9	5.4	D	3.755	86
ML1-EA5	7	Granitic Gneiss	22.1	6.7	D	4.343	100
SUMMARY[2]					**Average Axial**	**7.668**	**176**
					Average Diametral	**4.714**	**108**
					St. Dev. Axial	**0.504**	**12**
					St. Dev. Diametral	**0.740**	**17**
					Number of Axial Tests	**30**	
					Number of Diametral Tests	**68**	

[1] UCS = Is x 23 is based on previous experience and would require UCS testing to further validate this relationship.

[2]Statistical summary based on the removal of the 2 highest and 2 lowest values.

Note: Specimens tend to be anisotropic in nature (ie. stronger axial than diametral).

TABLE 2
EVALUATION OF FOUNDATION ALTERNATIVES
Highway 69 / Marsh Lake Road Underpass
G.W.P. 335-00-00

Footing Option	***NF***	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks/Consequences***
Spread Footings on bedrock or mass concrete pad		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, bedrock removal is required. Variability in bedrock surface will impact mass concrete quantities and excavation depths.
Spread Footings perched within embankment fill		Can eliminate bedrock removal and/or mass concrete placement for inner abutments.	Not practical at outer west abutment where bedrock excavation proposed (as per Drawing 1A). Potential for differential settlement between inner abutments (due to compression of embankment fill) and outer abutments (founded on unyielding bedrock).	Lower relative costs than piled foundations since less bedrock excavation required. Possible higher relative costs then spread footing on bedrock since lower allowable bearing capacity will require larger footing size.	Different footing design required at inner abutments and outer abutments for both single span structures.
Steel H Piles			Due to shallow depth of bedrock, bedrock excavation to form trench will be required to achieve minimum required piles lengths.	Significant bedrock trench for H-piles will increase costs for blasting and backfilling as compared to costs for bedrock excavation for spread footing alternative.	Not recommended due to significant depth of excavation required in strong bedrock.

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

TABLE 3
Summary of Recommendations at Structure Approach Embankments (incl. Platform Widening)
Highway 69 / Marsh Lake Road Underpass
G.W.P. 335-00-00

Highway	Approx. Station	Proposed Works	Surface Conditions	Recommended Embankment Fill Type	Organics Encountered along alignment	Recommended Side Slope	Side Berm Recommended	Estimated Post-Construction Settlement (δ)* and Platform Widening (w)** (mm)	Swamp Excavation / Organic Removal OPSD
Highway 69 / Marsh Lake Road Underpass	9+920 to 9+940 (Marsh Lake Road)	West Approach (fill generally up to about 2 m high; about 5 m high behind outer west abutment)	Relatively shallow silt and sand to silty sand overburden	Rock fill	Yes. Up to about 0.2 m below ground surface.	1.25H : 1V	No.	δ = up to 50 w = 1000	Remove all organics within footprint of embankment.
	9+985 to 10+000 (Marsh Lake Road)	Median Approach (fill up to about 4.5 m high)	Relatively shallow silt and sand to silty sand overburden	Rock fill	Yes. Up to about 0.6 m below ground surface.	1.25H: 1V	No.	δ = 60 w = 1000	Remove all organics within footprint of embankment.
	10+045 to 10+065 (Marsh Lake Road)	East Approach (fill up to about 5 m high)	Bedrock at or near ground surface	Rock fill	Yes. Up to about 0.2 m below ground surface.	1.25H: 1V	No.	δ = 50 w = 1000	Remove all organics within footprint of embankment.

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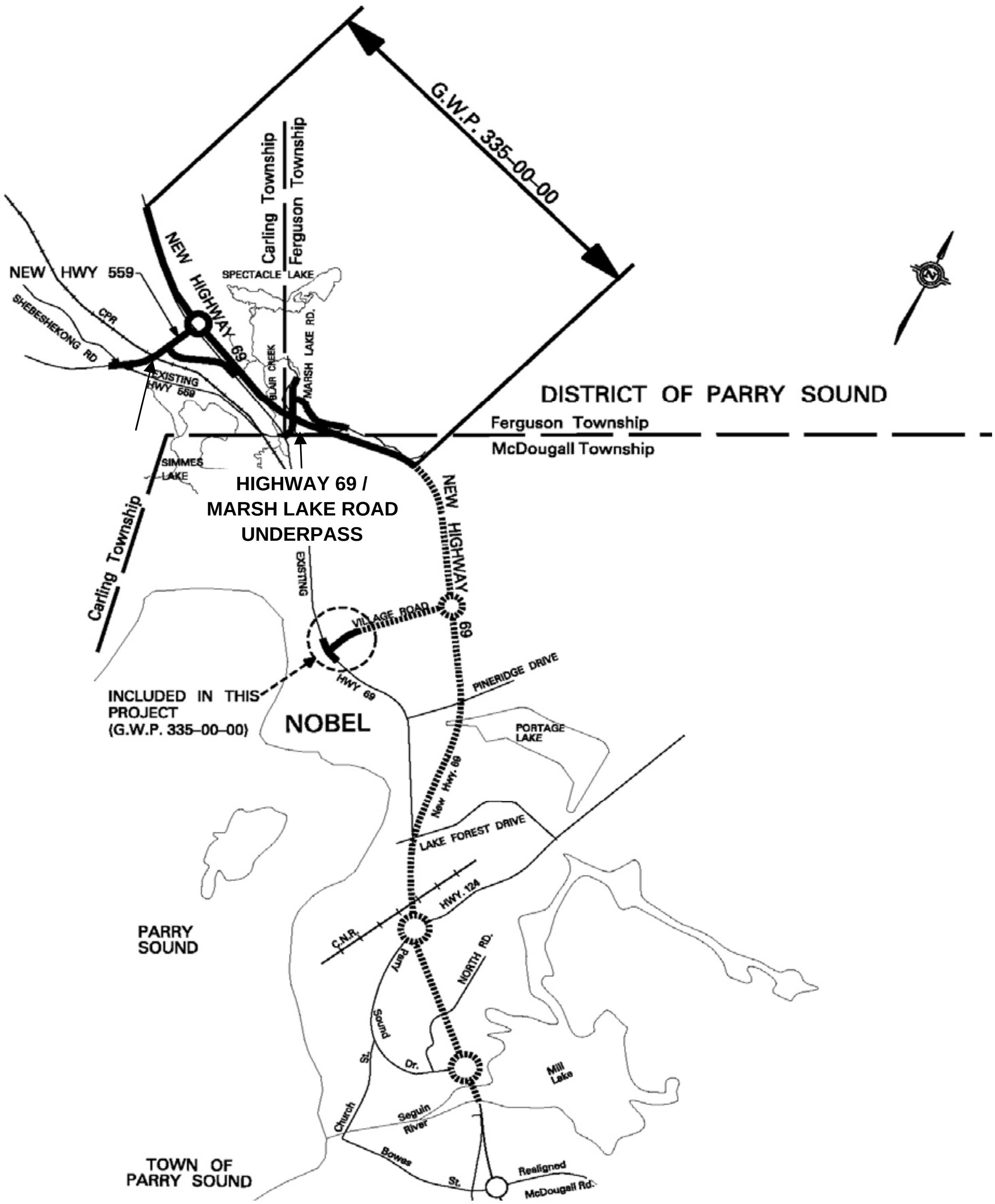
Note : *** Settlements include compression of rockfill plus compression of cohesive layers below embankment (where encountered).**
**** Recommended embankment platform widening (per embankment side) based on guidelines in NRE 98-200.**

FIGURES

SITE LOCATION MAP

HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559
TO 3.5 KM NORTH OF HIGHWAY 559

FIGURE 1



GWP No.	335-00-00		Drawn by:	CMG
Date:	March 2006		Checked by:	JPD
Project:	03-1111-028-6	**Golder Associates**	Provided in digital format by URS on January 7, 2005	

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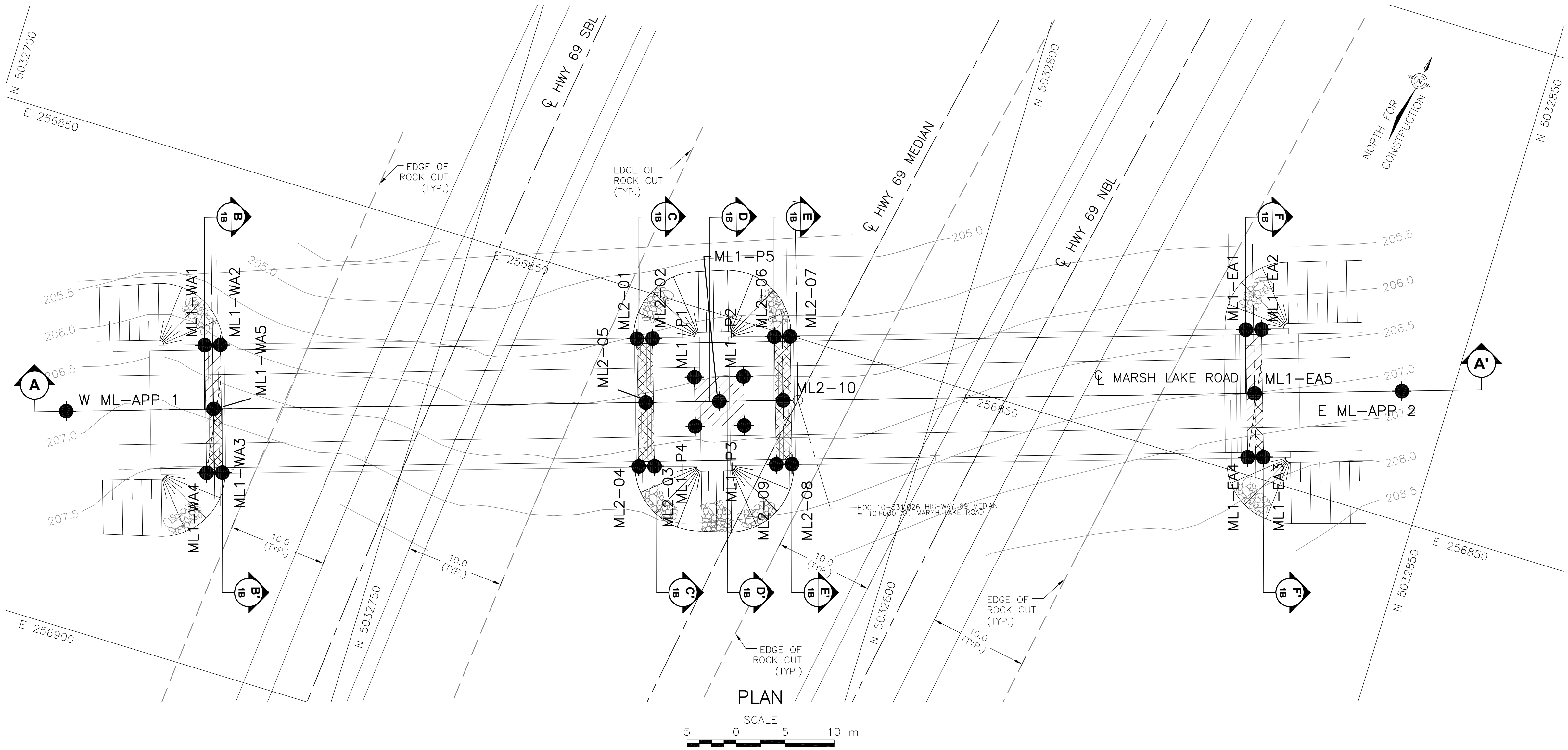


LEGEND:

BOREHOLE LOCATION

ALTERNATIVE 1

ALTERNATIVE 2

REFERENCES:

1. Mapping based on base plans provided in digital format by URS, drawing file 'FOOTING PLAN.dwg', dated May 2004, received July 27, 2004.

PLOT DATE: March 16, 2006
FILENAME: T:\Projects\2003\03-1111-028 (URSCI, Parry Sound)\-CB- (MARSH LAKE ROAD)\031111028CB002.dwg

Golder Associates
Mississauga, Ontario, Canada

FILE No. 031111028CB002.dwg		SCALE	AS SHOWN
PROJECT No. 03-111-028-6	REV.	DATE	MARCH, 2006
		DESIGN	
		CAD	MSM
		CHECK	CMG
		REVIEW	ASP

TITLE

STRUCTURE ALTERNATIVES

HIGHWAY 69/MARSH LAKE ROAD UNDERPASS

FIGURE **2**

DRAWINGS

METRIC

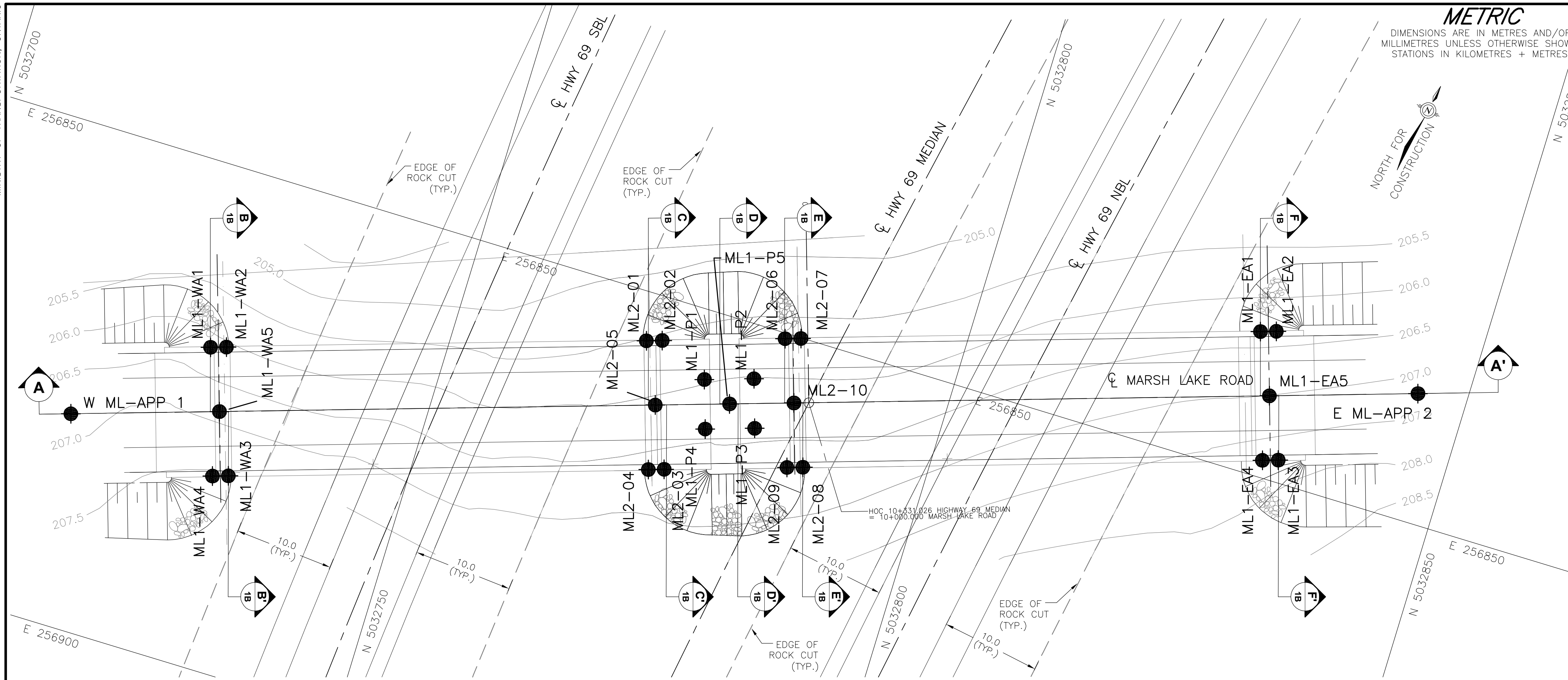
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 335–00–00

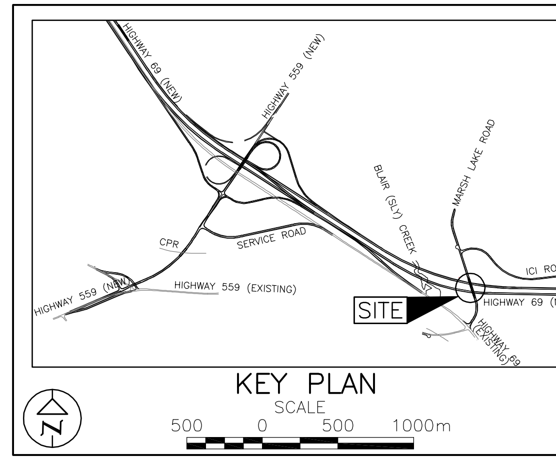
SHEET

HIGHWAY 69
MARSH LAKE ROAD UNDERPASS
BOREHOLE LOCATIONS & SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
5 0 5 10 m



KEY PLAN
SCALE
500 0 500 1000m

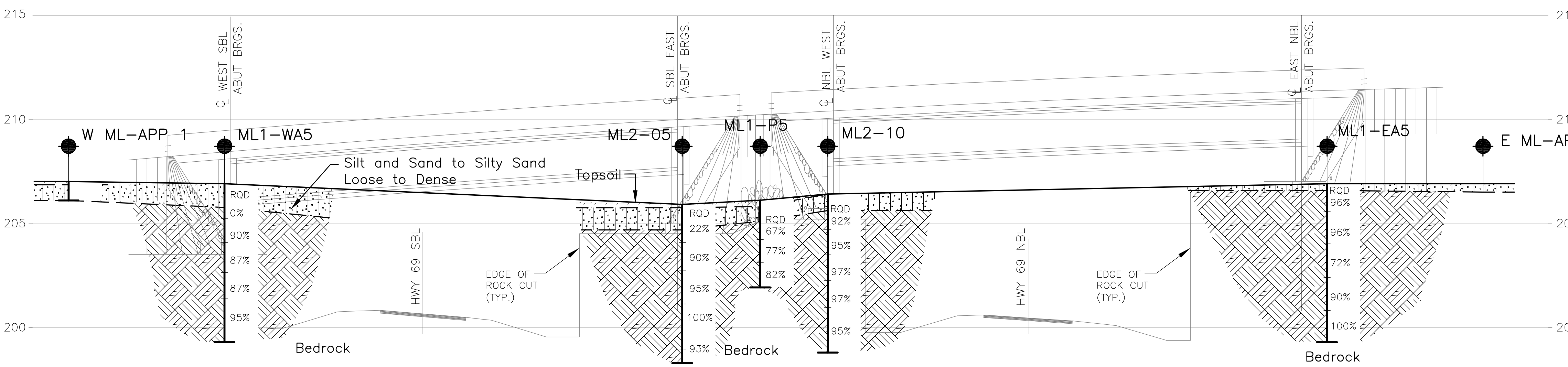
LEGEND

	Borehole – Current Investigation
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL in piezometer, measured on Nov. 14, 2004
	WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
W ML–APP 1	207.0	5032715.6	256878.8
ML2–01	205.2	5032769.0	256854.8
ML2–02	205.2	5032770.5	256854.3
ML2–03	206.5	5032774.5	256866.7
ML2–04	206.6	5032773.0	256867.2
ML2–05	205.9	5032771.7	256860.8
ML2–06	205.8	5032782.3	256850.6
ML2–07	205.9	5032783.8	256850.1
ML2–08	206.6	5032787.8	256862.4
ML2–09	206.7	5032786.3	256862.9
ML2–10	206.4	5032785.1	256856.5
ML1–P1	205.4	5032775.7	256856.9
ML1–P2	206.2	5032780.5	256855.3
ML1–P3	206.3	5032782.0	256860.1
ML1–P4	206.0	5032777.3	256861.6
ML1–P5	206.1	5032778.9	256858.5
ML1–EA1	206.4	5032828.0	256835.9
ML1–EA2	206.4	5032829.5	256835.4
ML1–EA3	207.6	5032833.5	256847.8
ML1–EA4	207.4	5032832.0	256848.3
ML1–EA5	206.9	5032830.8	256841.8
ML1–WA1	205.6	5032727.1	256868.3
ML1–WA2	205.8	5032728.6	256867.8
ML1–WA3	207.4	5032732.6	256880.2
ML1–WA4	207.4	5032731.1	256880.7
ML1–WA5	206.9	5032729.8	256874.2
E ML–APP 2	206.9	5032845.1	256837.2

REFERENCE

Base plans provided in digital format by URS, drawing files 'NBL_Marshl_GA.dwg and SBL_Marshl_GA.dwg', dated September 2005, received September 23, 2005.



A-A' CENTRELINE PROFILE
HORZ. SCALE
5 0 5 10 m
VERT. SCALE
2.5 0 2.5 5 m

REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN PROVINCE OF ONTARIO
REGISTERED PROFESSIONAL ENGINEER A.S. Poschmann PROVINCE OF ONTARIO

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.

NO.	DATE	BY	REVISION

Geocres No. 41H–56

HWY. 69	PROJECT NO. 03–1111–028	DIST. 52
SUBM'D. CMG	CHKD. CMG	DATE: MARCH, 2006	SITE:
DRAWN: JFC	CHKD. ASP	APPD. FJH	DWG. 1A

PLOT DATE: March 16, 2006
FILENAME: F:\Projects\2003\03-1111-028 (URS\GL Parry Sound)\~CB~ (MARSH LAKE ROAD)\031110280801A.dwg

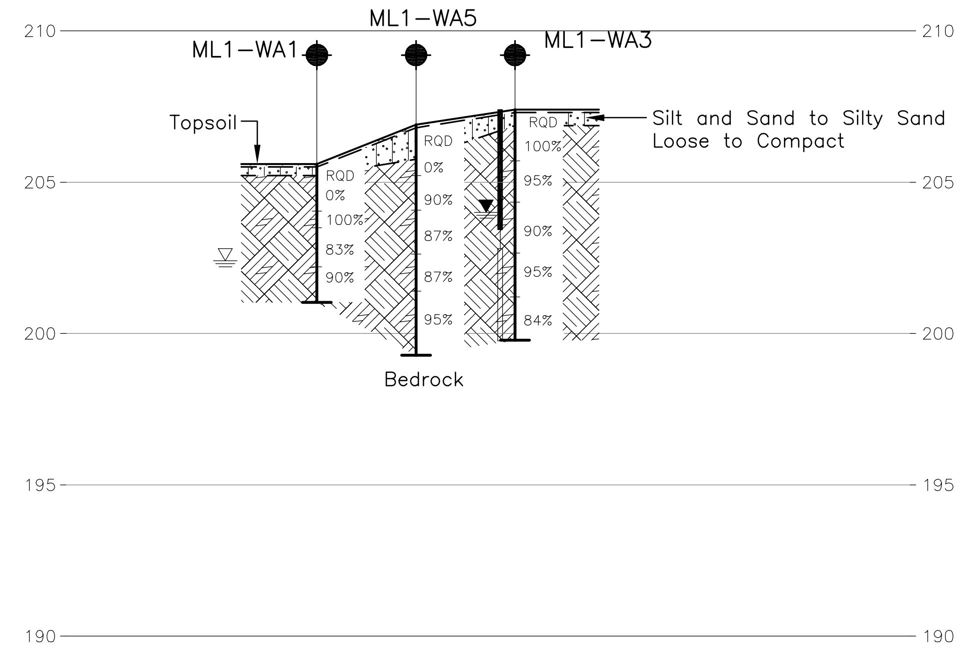
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 335-00-00

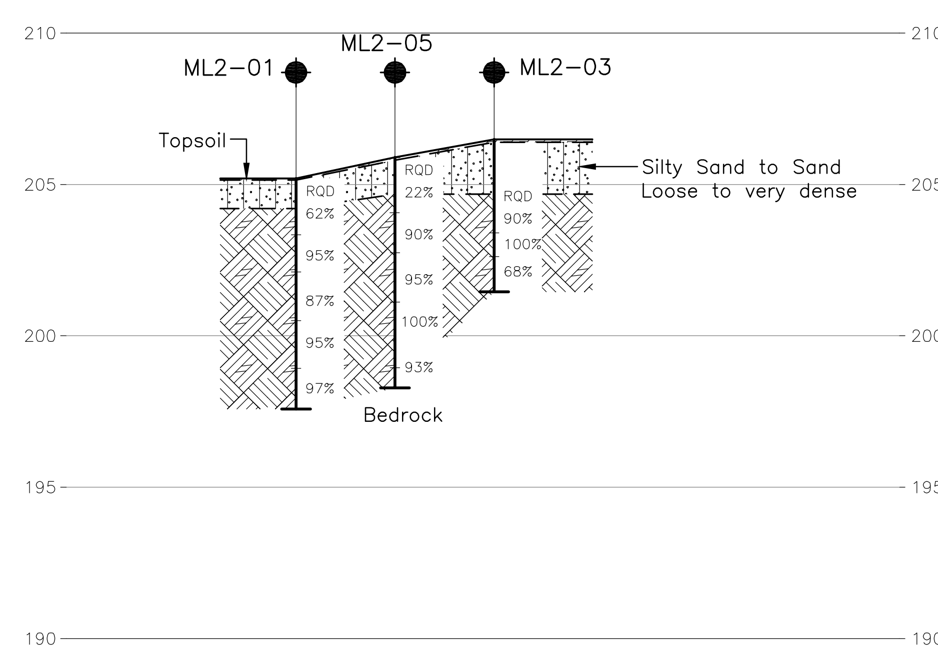
HIGHWAY 69
MARSH LAKE ROAD UNDERPASS
SOIL STRATA

SHEET

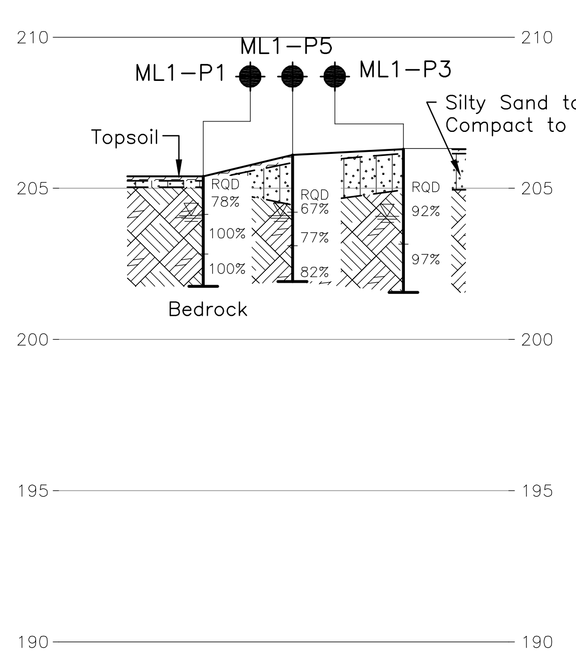
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



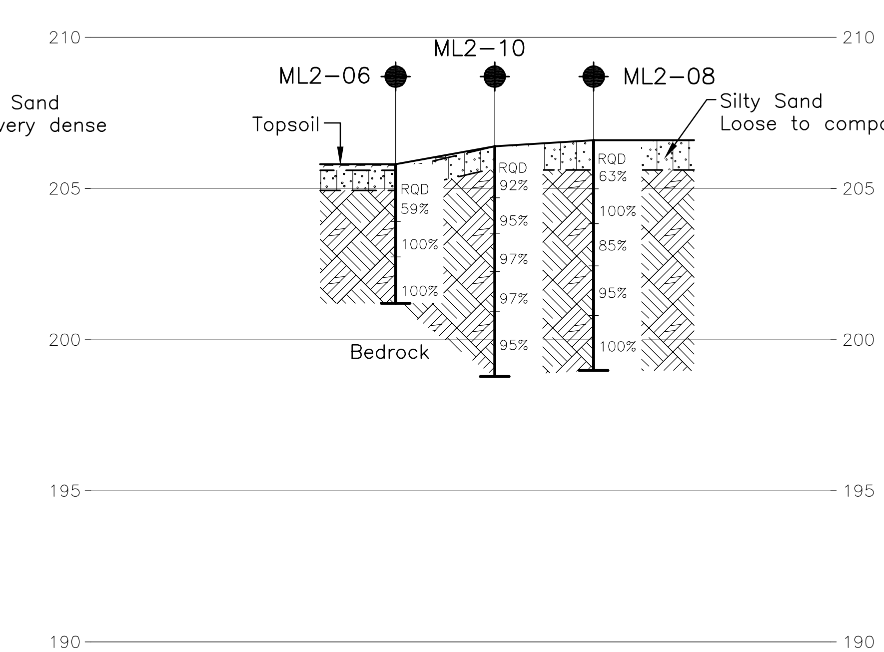
B-B' 1A OUTER SBL ABUTMENT
HORZ. SCALE 5 0 5 10 m
VERT. SCALE 2.5 0 2.5 5 m



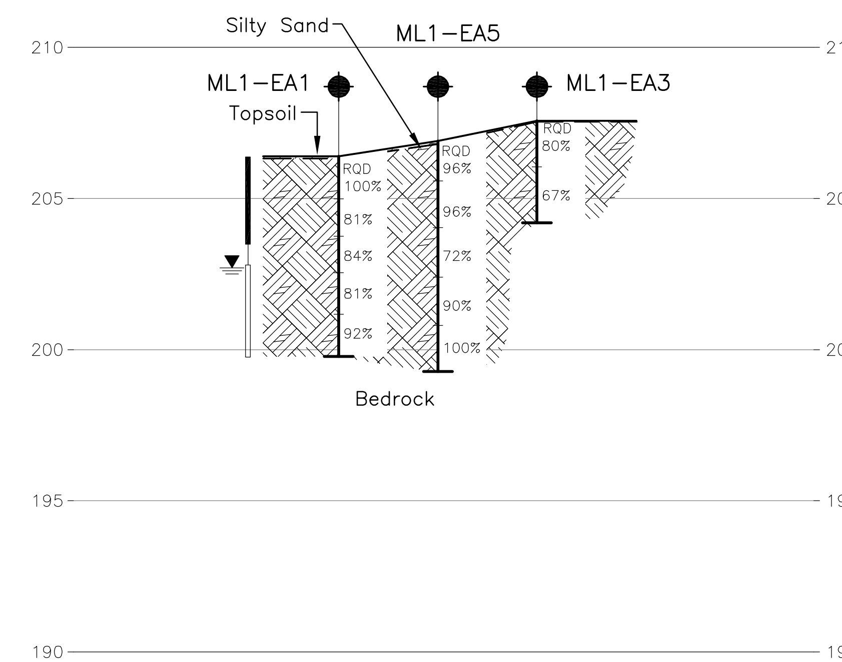
C-C' 1A INNER SBL ABUTMENT
HORZ. SCALE 5 0 5 10 m
VERT. SCALE 2.5 0 2.5 5 m



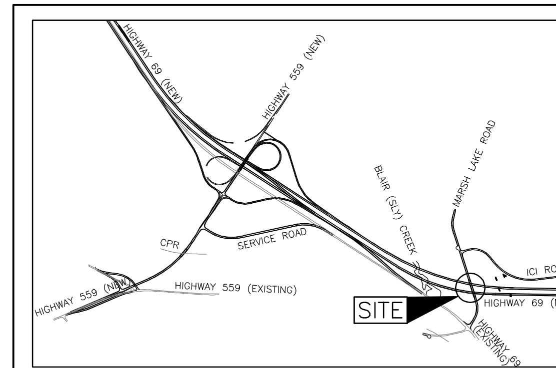
D-D' 1A MEDIAN
HORZ. SCALE 5 0 5 10 m
VERT. SCALE 2.5 0 2.5 5 m



E-E' 1A INNER NBL ABUTMENT
HORZ. SCALE 5 0 5 10 m
VERT. SCALE 2.5 0 2.5 5 m



F-F' 1A OUTER NBL ABUTMENT
HORZ. SCALE 5 0 5 10 m
VERT. SCALE 2.5 0 2.5 5 m



KEY PLAN
SCALE 500 0 500 1000m

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on Nov. 14, 2004
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
W ML-APP 1	207.0	5032715.6	256878.8
ML2-01	205.2	5032769.0	256854.8
ML2-02	205.2	5032770.5	256854.3
ML2-03	206.5	5032774.5	256866.7
ML2-04	206.6	5032773.0	256867.2
ML2-05	205.9	5032771.7	256860.8
ML2-06	205.8	5032782.3	256850.6
ML2-07	205.9	5032783.8	256850.1
ML2-08	206.6	5032787.8	256862.4
ML2-09	206.7	5032786.3	256862.9
ML2-10	206.4	5032785.1	256856.5
ML1-P1	205.4	5032775.7	256856.9
ML1-P2	206.2	5032780.5	256855.3
ML1-P3	206.3	5032782.0	256860.1
ML1-P4	206.0	5032777.3	256861.6
ML1-P5	206.1	5032778.9	256858.5
ML1-EA1	206.4	5032828.0	256835.9
ML1-EA2	206.4	5032829.5	256835.4
ML1-EA3	207.6	5032833.5	256847.8
ML1-EA4	207.4	5032832.0	256848.3
ML1-EA5	206.9	5032830.8	256841.8
ML1-WA1	205.6	5032727.1	256868.3
ML1-WA2	205.8	5032728.6	256867.8
ML1-WA3	207.4	5032732.6	256880.2
ML1-WA4	207.4	5032731.1	256880.7
ML1-WA5	206.9	5032729.8	256874.2
E ML-APP 2	206.9	5032845.1	256837.2

REFERENCE
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Geocres No. 41H-56

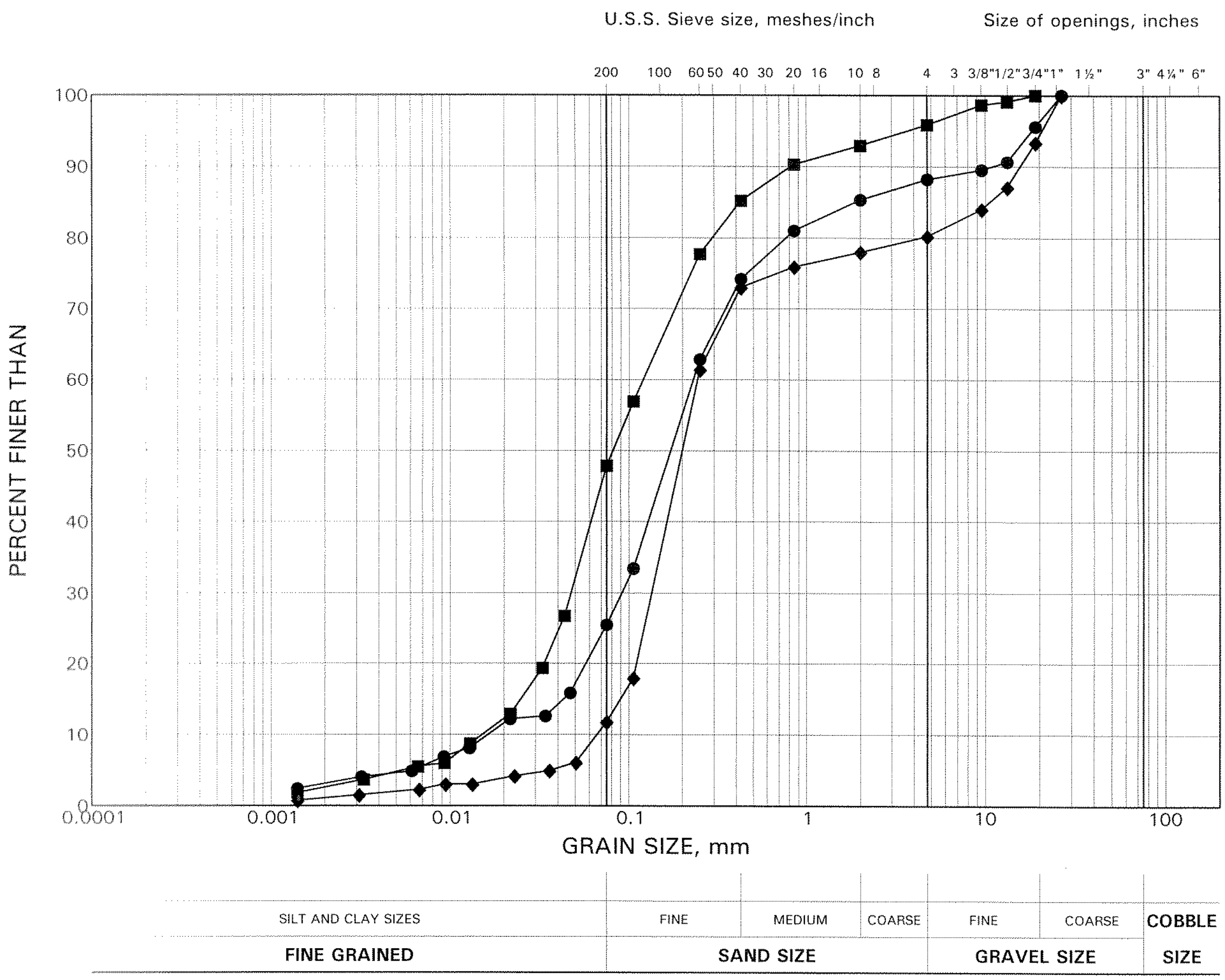
HWY. 69	PROJECT NO. 03-1111-028		DIST. 52
SUBM'D. CMG	CHKD. CMG	DATE: MARCH, 2006	SITE:
DRAWN: JFC	CHKD. ASP	APPD. FJH	DWG. 1B

APPENDIX A

LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION
Silt and Sand to Sand

FIGURE A-1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	ML1-P3	2	205.2
■	ML1-WA5	2	206.0
◆	ML2-04	3	204.8

Date 1/18/2005

Project 03-1111-028

Golder Associates

Prepared by LG

Checked by __________

APPENDIX B

SAMPLE NON-STANDARD SPECIAL PROVISIONS

MASS CONCRETE – Item No.

Non-Standard Special Provision

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the outer East/West and/or inner East/West abutment footings.

Construction

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

Basis of Payment

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

DOWELS Into Rock – Item No.

Non-Standard Special Provision

Scope of Work

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

Construction

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

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

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

Rock Dowel Testing

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

Performance Tests

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

Bridge	Foundation	Number of Dowels for Performance Testing
Highway 69 / Marsh Lake Road Underpass	Outer East Abutment	2
Highway 69 / Marsh Lake Road Underpass	Inner East Abutment	2
Highway 69 / Marsh Lake Road Underpass	Inner West Abutment	2
Highway 69 / Marsh Lake Road Underpass	Outer West 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.

Non-Standard 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.

<u>ROCK DOWELS FOR ROCK CUT FACES</u> - Item No.

Non-Standard Special Provision

Scope

Work under this item is for the installation of rock dowels for the stabilization of rock cut faces.

Construction

Install fully cement grouted, hot-dip galvanized deformed rock dowels at locations shown in the contract package and as directed by the Contract Administrator. The rock dowels are to consist of 2.0 or 3.0 m long 25 mm diameter (minimum) deformed rebar grade (minimum yield strength 400 MPA) bars as shown on contract drawings.

Drill the rock dowel holes at the diameter and inclination shown on the drawings or as otherwise directed by the Contract Administrator. Drill to a measured depth so that when bolts are fully inserted in the completed drill hole they are flush with the rock or shotcrete surface. Clean the holes using compressed air from the drill or a compressed air blowpipe, min. 500 cfm. .

Cement grout for dowels shall be pre-mixed, non-metallic shrinkage compensating grout placed according to the manufacturer's specifications. Water for use in grout mixes shall be clean and free of deleterious substances. The water shall be filtered if necessary to reduce the suspended solids to less than 500 mg/litre.

All dowels shall be fully cement grouted using a water:cement ratio of 0.35 to 0.40.

Dowels shall be installed and grouted at least 48 hours prior to any rock excavation in any areas within 5 m of the area where the dowels are to be installed.

Corrosion protection (approved by the Contract Administrator) is to be applied to all exposed surfaces not already protected.

Each rock dowel not fully grouted (drill hole not completely filled with hardened grout) or which protrudes by an amount greater or less than the specified amount is to be replaced at the Contractor's expense by another installed alongside.

The Contractor shall submit the following information at least 2 weeks prior to doing the work under this item for approval by the Contract Administrator.

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.

Rock Dowels: Rock dowel and grout supplier; type of grout; drill hole diameter and installation methodology.

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

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

ROCK DOWELS FOR ROCK CUT FACES - Item No.

Non-Standard Special Provision

Measurement for Payment

Measurement shall be for each rock dowel installed as specified.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all rock dowels, fittings, grout, corrosion protection and other materials, provision of cranes, lift equipment, scaffolding and other means of access, and labour and materials for drilling and installation.

WARRANT: Always with this tender item.