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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
DETAIL DESIGN
HIGHWAY 11, GRAVENHURST PATROL YARD
GRAVENHURST, ONTARIO
G.W.P 5420-02-00**

Submitted to:

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GEOCRES No. 31D-420

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PART A

**FOUNDATION INVESTIGATION REPORT
HIGHWAY 11, GRAVENHURST PATROL YARD
GRAVENHURST, ONTARIO
G.W.P. 5420-02-00**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) to provide foundation engineering services for the new Highway 11 Patrol Yard in Gravenhurst, Ontario, under G.W.P. 5420-02-00.

A geotechnical investigation was carried out at this site in January 2006 to establish the subsurface conditions at the proposed Patrol Yard structure locations by means of borehole drilling, bedrock coring, in situ testing, and subsequent geotechnical laboratory testing on selected samples.

All of the work was carried out in accordance with Golder's Proposal No. P51-1331, dated May 2005, which formed part of the Consultant's Agreement for this project, and the Quality Control Plan for this project dated September 2005.

2.0 SITE DESCRIPTION

The proposed Patrol Yard is located about 250 m northeast of the proposed Highway 11-Muskoka Road 169 underpass. The terrain at the site is undulating but generally slopes downwards from west to east, with the existing ground surface varying from about Elevation 257 m at the west side to about Elevation 254 m at the east side of the site. The site is covered with young to mature trees and shrubs; bedrock outcrops are present to the north and west of the proposed structure footprint as shown on Drawing 1.

3.0 INVESTIGATION PROCEDURES

3.1 Current Investigation

The current field investigation was carried out between January 19 and January 26, 2006, during which time ten boreholes were advanced at the site. Four boreholes were drilled within the proposed garage and office building area and six boreholes were advanced within the proposed truck wash bay and sand / salt storage area.

The boreholes were drilled using a CME-55 track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Barrie, Ontario. The boreholes were advanced using hollow stem augers to about 11 m depth or to auger/sampler refusal on inferred bedrock, where this was encountered at less than 11 m depth, with soil samples obtained at intervals of 0.75 m and 1.5 m depth, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedure (ASTM D1586-99). Approximately 3 m of NQ-size bedrock coring was carried out in four of the boreholes which were terminated at effective refusal to further auger penetration. Where refusal was not encountered at 11 m depth (i.e. at the locations of Boreholes F1, F9 and F10), Dynamic Cone Penetration Testing (DCPT) was carried out to effective refusal. The groundwater level in the open boreholes was observed and recorded throughout the drilling operations. Piezometers were installed in Boreholes F1 and F3 to monitor groundwater levels at the site.

The field work was supervised throughout by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's 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 (water contents, grain size distributions and Atterberg limits) was carried out on samples of the overburden soils. Strength testing (point load indices) was carried out on selected specimens from the rock core.

All investigated borehole locations were surveyed and referenced to the NAD83 MTM co-ordinate system and the geodetic datum. The locations of the boreholes were determined by MRC and Golder on a plan, and these locations were surveyed and staked by J.D. Barnes Ltd., Ontario Land Surveyors, prior to drilling. The borehole locations (referenced to the MTM NAD83 coordinate system) and ground surface elevations (referenced to the geodetic datum) are summarized in the following table and are shown on Drawing 1.

<i>Borehole Number</i>	<i>Borehole Location</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
F1	Salt Stockpile	4973532.1	316018.0	254.0
F2	Office Building	4973538.8	315978.0	255.6
F3	Garage	4973550.6	315919.7	257.2
F4	Garage	4973566.2	315920.8	256.8
F5	Garage	4973559.6	315956.7	256.3
F6	Truck Wash Bay	4973563.2	315996.7	254.9
F7	Truck Wash/ Sand Stockpile	4973583.8	316001.1	254.4
F8	Sand Stockpile	4973619.5	316010.0	254.9
F9	Sand Stockpile	4973615.0	316031.2	254.9
F10	Sand Stockpile	4973575.5	316023.3	254.2

3.2 Previous Investigation

Borehole information from a previous investigation carried out at the site by MRC and Ecoplans Limited has been included with this report. The report for the previous investigation is referenced as follows:

- Final Report entitled “Highway 11 Gravenhurst Patrol Yard Site Selection Study”, prepared by McCormick Rankin Corporation, dated February 2005.

The previous field work for the Site Selection Study investigation was carried out by EcoPlans Limited between May 17 and May 20, 2004, during which time nine boreholes were advanced in the vicinity of the proposed Patrol Yard. Five of these boreholes (denoted BH6, MW5, MW6, MW7 and MW8) are located within the site boundary and near the proposed structures and are considered relevant for the current investigation. The locations of the relevant previously drilled boreholes are shown on Drawing 1 and copies of the borehole logs from these previous investigations are provided in Appendix B.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

According to *The Physiography of Southern Ontario*, the site is mainly located within the physiographic region known as the “Number 11 Strip”, with portions of Highway 11 in contact with the “Georgian Bay Fringe” region. The Number 11 Strip is a narrow belt that extends from Gravenhurst to North Bay and is characterized by deposits of sand, silt and clay, together with more recent swamp deposits, between rock outcrops. The Georgian Bay Fringe is a broad belt characterized by shallow soil and bare bedrock knobs and ridges. The bedrock in the area is typically highly deformed gneiss of the Moon River Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province (Ontario Geological Society, 1991).

4.2 Subsurface Conditions at Patrol Yard Site

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the Record of Borehole and Drillhole sheets following the text of this report. The borehole records and results of laboratory testing from the previous investigation at the site are included in Appendix B.

The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The inferred stratigraphy as encountered in the boreholes and DCPTs at the proposed Patrol Yard site is shown on Drawing 2. The total overburden thickness ranged from approximately 1.8 m at the north side of the proposed garage building structure to about 19.5 m at the southeast corner of the sand/salt storage structure, as encountered in the boreholes. In general, the subsoils at the proposed Patrol Yard site consist of cohesionless soils, varying in composition from sand to silt, underlain by bedrock. The surficial sand to silt deposits contained trace quantities of organics, and the silt content of the cohesionless deposit generally decreased from the west side to the east side of the site. In one borehole (Borehole F7) advanced at the north entrance of the proposed wash bay area, a 0.3 m thick layer of clayey silt was encountered directly above the bedrock; a thin (0.1 m thick) layer of clayey silt was also encountered in one of the boreholes (Borehole MW5) advanced as part of the previous investigation by Ecoplans.

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 the ground surface in Boreholes F1 to F3 and F6 to F8. The topsoil ranged from about 300 mm to 400 mm in thickness.

4.2.2 Sand to Silt

Below the topsoil in Boreholes F1 to F3 and F6 to F8, and immediately below the ground surface in the remaining boreholes (Boreholes F4, F5, F9 and F10), lies a deposit of sand, to silty sand, to sand and silt, to sandy silt, to silt. The silt content within the cohesionless deposit is generally higher at the west side of the site, and decreases in an easterly direction resulting in predominantly sandy soils at the east side of the site, as encountered at the borehole locations. The cohesionless soils typically contain trace gravel and clay. Trace organics were encountered within the upper 0.3 m to 0.8 m below the ground surface. Cobbles were encountered within the 1.8 m of the cohesionless soil deposits in Boreholes F1, F3, F5, F6 and F10; cobbles and boulders should also be expected throughout the cohesionless soils, particularly near the soil/bedrock interface. Grain size distribution test results taken on selected samples of the sand to silty sand are presented on Figure A1 in Appendix A, and show the uniform grading of this material. Grain size distribution test results for the sand and silt to sandy silt are shown on Figure A2, and results for the silt are shown on Figure A3.

The top of the cohesionless soil deposit was encountered at depths ranging from ground surface to 0.4 m depth, and the measured thickness ranged from 1.8 m to greater than 11.3 m (i.e. the maximum sampled depth, below which DCPTs were advanced).

The measured Standard Penetration Test (SPT) “N” values in the cohesionless sand to silt soils generally range from 22 blows to greater than 50 blows per 0.3 m of penetration, indicating that this deposit has a generally compact to very dense relative density. A single SPT “N” value of 10 blows per 0.3 m of penetration was measured at a depth of about 10 m in Borehole F9; this value may be the result of disturbance due to groundwater inflow (“blowing” sands into the augers) during drilling and sampling. Typically, higher SPT “N” values were measured near the bedrock interface.

The natural water content measured on selected samples of the sand to silt soils generally ranged from 5 to 27 per cent; the higher water content values were generally measured on samples obtained below the water table, and on those samples with a higher silt content. A natural water content of 79 per cent was measured on a sample taken at the sand and silt / topsoil interface; this high value is attributed to the presence of organics and the high silt content in this particular sample.

4.2.3 Sand and Gravel to Gravelly Sand

In Borehole F3, a layer of sand and gravel was encountered below the sandy silt to silt layer at a depth of 5.8 m (Elevation 251.4 m). The sand and gravel, trace silt layer was 4.3 m thick and extended to the bedrock surface, which was encountered at a depth of 10.1 m. In Borehole F9, a layer of gravelly sand was encountered below the sand deposit at a depth of 10.7 m (Elevation 244.2 m). The gravelly sand, trace silt layer had a measured thickness of 0.6 m; however, this layer could extend to the inferred bedrock surface which was encountered at a depth of 11.3 m (Elevation 11.9 m), suggesting a thickness of about 1.2 m. Grain size distribution test results for selected samples of the sand and gravel to gravelly sand are shown on Figure A4 in Appendix A, and indicate the well-graded nature of this deposit. It is noted that cobbles and boulders should be expected within the sand and gravel to gravelly sand layers, particularly near the soil/bedrock interface, as was found in Borehole MW7 in the previous investigation at this site.

The measured SPT “N” values in the sand and gravel / gravelly sand layer ranged from 55 blows to greater than 100 blows per 0.3 m of penetration, indicating a very dense relative density.

The natural water content measured on selected samples of the sand and gravel / gravelly sand ranged from 9 to 13 per cent.

4.2.4 Clayey Silt

A 0.3 m thick layer of clayey silt containing some sand, as well as sand seams, was encountered in one borehole (Borehole F7) drilled during the current investigation. The clayey silt was encountered at a depth of 5.0 m (Elevation 249.4 m), and it underlies the sand deposit and directly overlies the inferred bedrock surface. A 0.1 m thick layer of clayey silt was also encountered in one borehole (Borehole MW5) drilled during the previous investigation at the site.

The measured SPT “N” values within the clayey silt layer are 12 and 13 blows per 0.3 m of penetration, indicating that this layer has a stiff consistency.

A natural water content measured on one sample of the clayey silt was 30 per cent. Atterberg limits testing carried out on a sample of the clayey silt measured a liquid limit of 25 per cent and a plastic limit of 16 per cent, corresponding to a plasticity index of 9 per cent. The Atterberg limits test results are shown on the Record of Borehole sheet and are plotted on a plasticity chart on Figure A5 in Appendix A; these results indicate a clayey silt of low plasticity.

4.2.5 Bedrock

Visible bedrock outcrops are present to the north and west of the site, beyond the proposed limits of the patrol yard structure, as shown on Drawing 1. Bedrock was encountered and cored for 3 m in Boreholes F3, F5, F6 and F8. The presence of bedrock was inferred from auger, sampler and/or DCPT refusal in all of the remaining boreholes advanced as part of the current investigation. The surface of the bedrock is variable, and was encountered in the boreholes between about 1.8 m and 19.5 m depth below ground surface, at elevations ranging from Elevation 254.7 m to 234.5 m. The depth to bedrock below ground surface (i.e. overburden thickness) and corresponding bedrock surface elevation encountered at each borehole location is summarized in the following table.

<i>Borehole Number</i>	<i>Borehole Location</i>	<i>Depth to Bedrock Surface</i>	<i>Bedrock Surface Elevation</i>	<i>Comments</i>
F1	Salt Storage	19.5 m	234.5 m	DCPT Refusal
F2	Office	3.7 m	251.9 m	Auger Refusal
F3	Garage	10.1 m	247.1 m	Bedrock Cored
F4	Garage	2.1 m	254.7 m	Auger Refusal
F5	Garage	1.8 m	254.5 m	Bedrock Cored
F6	Wash Bay/Sand Storage	4.3 m	250.6 m	Bedrock Cored
F7	Wash Bay/Sand Storage	5.3 m	249.1 m	Auger Refusal
F8	Sand Storage	5.5 m	249.4 m	Bedrock Cored
F9	Sand Storage	11.9 m	243.0 m	DCPT Refusal
F10	Sand Storage	12.5 m	241.7 m	DCPT Refusal

Based on the cored bedrock samples, the bedrock generally consists of biotite gneiss (in Boreholes F6 and F8) and granite gneiss (in Boreholes F3, F5 and F8).

In general, the biotite gneiss bedrock samples are described as slightly weathered to fresh, black and white, fine to medium grained, and strong to very strong. The bedrock samples typically contained distinct foliation planes and medium- to coarse-grained, quartz- and feldspar-rich veins/banding. The Rock Quality Designation (RQD) measured on the biotite gneiss core samples typically ranged from about 58 to 100 per cent, indicating a rock mass of fair to excellent quality.

The granite gneiss bedrock samples are described as slightly weathered to fresh, black, white and pink, fine to coarse-grained, and strong to very strong. The bedrock samples typically contained distinct foliation planes and fine- to medium-grained, biotite-rich bands/clusters and thinly banded quartz. The Rock Quality Designation (RQD) measured on the core samples typically ranged from about 58 to 98 percent, indicating a rock mass of fair to excellent quality.

Point load strength tests were performed on samples of the rock core. Diametral and axial point load strength index values are shown on the Record of Drillhole Sheets and on Table 1 following the text of this report. The point load index (Is_{50}) results from the laboratory tests on the gneissic bedrock range from approximately 4.1 MPa to 7.5 MPa with an average of about 5.7 MPa for diametral tests (i.e. testing carried out perpendicular to the core axis). Axial tests (i.e. testing carried out parallel to the core axis) performed on the gneissic bedrock range from approximately 4.1 MPa to 7.6 MPa with an average of about 5.3 MPa. Diametral and axial point load tests performed on the biotite gneiss bedrock samples typically gave lower point load test values compared to the granite gneiss bedrock with an average of about 4.9 MPa and 4.4 MPa for diametral and axial tests, respectively.

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

Borehole No.	Average Diametral Point Load Index, Is_{50} (MPa)		Average Axial Point Load Index, Is_{50} (MPa)	
	Biotite Gneiss	Granite Gneiss	Biotite Gneiss	Granite Gneiss
F3	-	5.1	-	5.2
F5	-	5.5	-	7.6
F6	4.9	-	4.1	-
F8	-	7.5	4.6	5.1

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

4.2.6 Groundwater Conditions

The water level was observed in the open boreholes at the time of drilling, and a standpipe piezometer was installed in Boreholes F1 and F3 to permit monitoring of water levels. The piezometer in Borehole F1 was sealed near the middle of the cohesionless sand deposit at a depth of about 4.6 m, and the piezometer in Borehole F3 was sealed within the sand and gravel layer directly above the bedrock surface at a depth of 8.2 m. Details of the piezometer installation are shown on the Record of Borehole Sheets following the text of this report.

The groundwater level typically varied from Elevation 250.3 m to 252.9 m (approximately 2 m to 5.5 m below ground surface at the borehole locations) during the borehole investigation carried out in January 2006. Water levels measured in the piezometers on February 6, 2006 varied from Elevation 250.3 m (about 3.7 m depth) to Elevation 252.0 m (about 5.2 m depth). The water levels measured in the piezometers are summarized in the following table:

<i>Borehole Number</i>	<i>Ground Surface Elevation (m)</i>	<i>Water Level Depth (m)</i>	<i>Water Level Elevation (m)</i>	<i>Date Measured</i>
F1	254.0	3.7	250.3	February 6, 2006
F3	257.2	5.2	252.0	February 6, 2006

Monitoring wells were also installed by MRC/EcoPlans in previous Boreholes MW5, MW6, MW7 and MW8 which are sealed within the cohesionless sand and silt deposit. Details of the well installations are shown on the Log of Borehole sheets contained in Appendix B. The groundwater level typically varied from Elevation 252.2 m to 253.7 m in the relevant boreholes drilled during the previous investigation in July 2004. The most recent water levels measured in these wells are summarized in the following table.

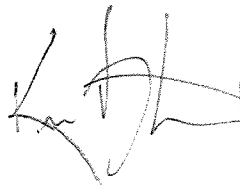
<i>Borehole Number</i>	<i>Ground Surface Elevation (m)</i>	<i>Water Level Depth (m)</i>	<i>Water Level Elevation (m)</i>	<i>Date Measured</i>
MW5	255.1	1.4	253.7	July 21, 2004
MW6	255.1	1.6	253.5	July 21, 2004
MW7	254.4	2.2	252.2	July 21, 2004
MW8	255.0	1.6	253.4	July 21, 2004

Based on the water level measurements made during the current and previous investigations at the site, the water level varies from about Elevation 250.3 m to 253.7 m, depending on the time of year. It should be noted that the groundwater levels at the site are anticipated to fluctuate as a result of seasonal variations in precipitation, runoff and temperature at the site.

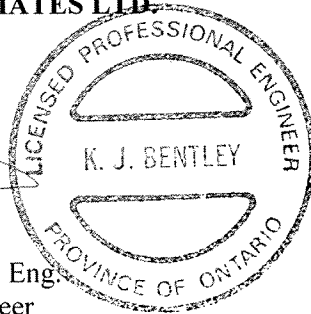
5.0 CLOSURE

The borehole investigation program for this project was supervised by Mr. Suresh Bainey, a Senior Technician with Golder. This Foundation Investigation Report was prepared by Ms. Shannon Palmer and Mr. Kevin J. Bentley, P.Eng., and was reviewed by Ms. Lisa C. Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder, carried out an independent quality control review.

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PART B

**FOUNDATION DESIGN REPORT
HIGHWAY 11, GRAVENHURST PATROL YARD
GRAVENHURST, ONTARIO
G.W.P. 5420-02-00**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides geotechnical/foundations recommendations for the proposed MTO Patrol Yard structures and engineered fill. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes advanced during the current and previous subsurface investigations carried out at the site.

The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

The new Patrol Yard structures will consist of a drive-through sand and salt containment area, a truck wash/bay area, an office area and a garage. The office, garage area, and truck wash/bay area will be heated, while the sand / salt storage areas will be unheated. Based on the preliminary design drawings (Drawing Nos. A-02 and A-03, dated December 15, 2005) provided to Golder by MRC, the proposed garage and office area will consist of an approximately 30 m by 130 m, single-storey steel structure with a concrete block wall exterior. The sand and salt storage area will be approximately 50 m to 60 m wide by 175 m with a maximum height of about 12.8 m; this structure will be constructed with 4 m high, buttress-supported, cast-in-place concrete walls around the perimeter, with pre-engineered timber walls and roof above.

The existing ground surface at the site varies from about Elevation 253 m to 259 m, sloping downward from the northwest (where exposed bedrock outcrops are present) to the southeast. Within the building footprint itself, the ground surface varies from about Elevation 257 m at the west end of the garage, to about Elevation 254 m to 254.5 m within the sand/salt storage area. Based on the proposed pavement elevations and drainage sketch provided by MRC (dated January 10, 2006), it is understood that the proposed top of the floor slab will be at about Elevation 257.1 m. In order to achieve this grade, up to about 3 m of engineered fill will be required to be placed within the structure footprints, as follows: approximately 2 m to 3 m of fill is required within the limits of the salt/sand storage structure; approximately 1.5 m to 2.5 m of fill is required in the office and wash bay areas; and approximately 0.5 m to 1.5 m of fill is required within the garage area.

The overburden soils at the site generally consist of compact to very dense sand, silty sand and sand and silt deposits, underlain by gneissic bedrock. There are typically trace to some amounts of organic material or topsoil present within about 0.3 m of the ground surface. The groundwater level in the sand and silt, silty sand and sand deposits was measured between about Elevation 250.9 m to 252.9 m; typically at slightly greater depth towards the southeast. The sand, silty sand to sand and silt overburden soils are typically underlain by strong to very strong biotite gneiss and granite gneiss bedrock of fair to excellent quality. In Boreholes F3 and F5, the upper portion (about 0.1 m to 0.15 m) of bedrock contains zones of broken rock of poor quality.

6.2 Site Preparation and Engineered Fill Construction

Any fill materials placed within the building envelope or within paved areas (i.e. excluding green spaces) should be carried out as an engineered fill.

Layers of topsoil approximately 0.3 m to 0.4 m in thickness were encountered at ground surface across the majority of the site. Native sandy soils containing trace amounts of topsoil/organics were present either at ground surface or beneath the topsoil extending to depths of 0.3 m to 0.8 m. The topsoil and any portions of the cohesionless deposit that are loose/disturbed or contain significant amounts of organics and/or other deleterious materials are not considered to be suitable for the subgrade support of building foundations, floor slabs, or other settlement-sensitive structures, or engineered fill materials that support these facilities. Following the stripping of the surficial topsoil, the exposed subgrade should be heavily proof-rolled under the supervision of experienced geotechnical personnel. Any softened/loosened or poorly performing areas of the subgrade soils should be subexcavated and replaced with engineered fill comprised of free-draining material, such as OPSS Select Subgrade Material or Granular B, Type I.

The prepared area should encompass the limits of the engineered fill. The engineered fill limits are defined such that the fill extends to at least 1 m beyond the outside edge of the founding level of any footing or other settlement-sensitive area and then downward and outward at a slope of one horizontal to one vertical (1H:1V).

Following proof-rolling and approval of the subgrade, engineer-approved fill should be placed in accordance with MTO's Special Provision SP105S10. Within building footprints, the fill should be compacted to 100 per cent of the material's Standard Proctor maximum dry density. Within paved areas, the fill should be compacted to 98 per cent of the material's Standard Proctor maximum dry density, except for the final lift which should be compacted to 100 per cent, or as required for the pavement design. Filling should continue until the design subgrade elevations are achieved, with full-time inspection and in-situ density testing carried out by a qualified geotechnical engineering firm during placement of engineered fill beneath the structure and settlement-sensitive areas.

As discussed in Sections 6.3 (Building and Perimeter Retaining Wall Foundations) and 6.7 (Settlement), the geotechnical recommendations provided for the design of shallow foundations and for settlement under the sand/salt pile loadings are based on the use of Granular “A” or Granular B Type II fill meeting the requirements of OPSS 1010 within the structure footprints. Outside of the structure and paved areas, native soils from cut operations at the site or other imported sand fill may be considered.

The final surface of the engineered fill should be protected as necessary from construction traffic, and should be sloped to provide positive drainage for surface water during the construction period. If the engineered fill materials will be left exposed (i.e. uncovered) during periods of freezing weather, consideration should be given to placing an additional soil cover above final subgrade to provide for frost protection.

6.3 Building and Perimeter Retaining Wall Foundations

The following foundation options have been considered for support of the proposed Patrol Yard structures:

- Strip/spread footings supported on the compact to very dense native sandy soils (where relatively little engineered fill is placed), on the engineered fill itself where greater thicknesses of fill are required (provided that OPSS Granular “A” or Granular “B” Type II material is used), or on the bedrock (where the engineered fill and native soils are relatively thin along the north side of the proposed garage).
- Caissons, which should be extended to the bedrock and not founded within the wet cohesionless soils above the bedrock surface, due to difficulties associated with disturbance of water-bearing sandy soils at the caisson base. The bedrock was encountered in the boreholes between 1.8 m and 19.5 m depth below existing ground surface, and the local slope of the bedrock surface typically ranges from 1.5H:1V to 3H:1V. The drilled piles/caissons would need to be advanced using a temporary liner to minimize disturbance to and loss of wet sandy soils, and then socketted into the bedrock to achieve a level founding surface at the base of the caisson and to minimize the potential for sliding along the inclined bedrock surface. In order to achieve a bedrock socket into the sloping bedrock, specialized equipment and drilling techniques would be required. Also, a special provision would be required to address varying lengths of caissons, and the presence of cobbles and boulders within the native soils that could affect the caisson installation.

- Steel H-piles driven to found on the bedrock, the surface of which varies between 1.8 m and 19.5 m depth below existing ground surface as encountered in the boreholes advanced as part of the current investigation. Driven piles would only be feasible where the thickness of the native soil plus the engineered fill is greater than about 5 m, and so could not be used for support of portions of the garage and office. In addition, cobbles and boulders within the native soils could affect the pile installation.

Based on the above considerations, shallow foundations (strip and spread footings) are considered to be the most economical, and the most practical from a foundations perspective, given the generally dense to very dense nature of the site soils. Caissons founded within the bedrock are not considered to be a practical option. Driven steel H-piles would be feasible for support of portions of the proposed structures; however, given that the native soils are generally dense to very dense and given that good quality engineered fill is required to achieve proper performance of the new raised floor slabs, such deep foundations are less practical and less economical than the shallow foundation option.

Recommendations for the preferred foundation option – strip and spread footings – and for the feasible but less practical option of steel H-piles are provided in the following sections.

6.3.1 Strip and Spread Footings

Strip or spread footings should be sized to comply with the minimum footing width given in the Ontario Building Code (OBC).

For spread footings or mass concrete founded on the properly prepared bedrock at this site, frost susceptibility is not an issue. However, where the founding subsoils consists of a native sandy deposit or engineered fill, all exterior footings and footings in unheated areas should be provided with a minimum of 1.7 m of soil cover for frost protection purposes. In addition, the bearing soil and fresh concrete should be protected from freezing during cold weather construction.

If stepped spread footings are constructed, the difference in elevation between individual footings should not be greater than one-half the clear distance between the footings. In addition, the lower footings should be constructed first so that if it is necessary to construct the lower footings at a greater depth than anticipated, the elevations of the upper footings can be adjusted accordingly. Stepped strip footings should be constructed in accordance with the Ontario Regulation 419/86, Section 9.15.3.12.

All foundation excavations should be inspected by qualified geotechnical personnel prior to placement of structural concrete to verify that the subgrade conditions are consistent with those assumed in the design, and free of loose/softened or excessively wet material.

6.3.1.1 Geotechnical Resistance

For strip or spread footings founded within the native compact to very dense sand to sand and silt, a factored geotechnical resistance at Ultimate Limit States (ULS) of 400 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 300 kPa may be used for design. These recommended resistances are based on a minimum footing width of 0.6 m.

Where the site grade will be raised by the placement of engineered fill materials, excavations to reach the compact to very dense native sand to sand and silt after fill placement may be as much as 3 m deep at some locations. Strip or spread footings having a minimum width of 0.6 m can also be supported on the properly placed and compacted engineered fill. Assuming that OPSS Granular “A” or Granular “B” Type II material is used for the engineered fill, and assuming that the fill is compacted to 100 per cent of the Standard Proctor maximum dry density within and beyond the building footprint, a factored geotechnical resistance at ULS of 400 kPa and at SLS of 300 kPa can be used for design of the foundations. The maximum total and differential settlements are expected to be less than 25 mm and 20 mm, respectively, for footings designed based on these recommended SLS values. This does not include the settlements due to the sand/salt stockpile loading, which are discussed in Section 6.7.2.

The following table provides the highest recommended founding levels at each borehole location for the underside of strip or spread footings, and/or for the underside of engineered fill, in order to achieve the above factored geotechnical resistances.

<i>Borehole No.</i>	<i>Highest Founding Elevation for Strip/Spread Footings and/or Highest Elevation for Underside of Engineered Fill</i>
F1	253.5 m
F2	255.2 m
F3	256.4 m
F4	256.0 m
F5	255.5 m
F6	254.2 m
F7	253.6 m
F8	254.4 m
F9	254.6 m
F10	253.5 m

The values provided above represent the anticipated maximum founding elevations. Regardless of the elevations given in the table above, the strip or spread footings must be provided with a minimum of 1.7 m of soil cover (or equivalent) to provide adequate protection against frost

penetration. Therefore, in many areas it will be necessary to extend the footings deeper than the elevations provided in the table.

Where the founding elevations for spread footings encounter a combination of native sandy soils and engineered fill across the width of the footing, the engineered fill should be subexcavated so that the footing is founded entirely within the native soils.

Bedrock was encountered at shallow depths of about 1.8 m and 2.1 m below ground surface along the north side of the proposed garage. Based on the required thickness of engineered fill in this area (less than about 1 m) and the required soil cover for frost protection purposes, the underside of the footings in this area will be at about Elevation 255.4 m, which is about 0.6 m above the bedrock surface. The footings in this area can be founded on the dense to very dense sand/silt deposit, as discussed above; however, it is noted that the subgrade soils in this area could be wet and it could be difficult to fully dewater the soil in close proximity to the bedrock. In this case, consideration could be given to extending shallow foundations for the north side of the garage deeper than required for frost protection purposes, and placing them directly on the bedrock; such excavations are expected to extend through water-bearing cohesionless soils immediately above the bedrock, and groundwater control will be required to maintain the stability of the excavation. If this option is adopted, following excavation to the bedrock surface, all loose, broken and/or fractured rock within the foundation footprint should be cleaned and scaled, and mass concrete placed as necessary to create a level founding area, in accordance with MTO's Special Provision SP902S01. Strip or spread footings placed on the bedrock, or on a mass concrete pad on the bedrock, may be designed based on a factored geotechnical resistance at ULS of 6,000 kPa; this resistance assumes 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 bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

The geotechnical resistances provided in this section 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.

If the concrete for the footings on the native or engineered fill soil cannot be poured immediately after excavation and inspection, it is recommended that a working mat of lean concrete be placed in the excavation to protect the integrity of the bearing stratum. A Non-Standard Special Provision should be included in the Contract Documents in this regard.

Concrete used for the foundations should be designed to resist the anticipated harsh environmental conditions (i.e. foundations directly in contact with salt, detergents in wash bay, etc.).

6.3.1.2 Resistance to Lateral Loads

The following table provides coefficients of friction, $\tan \delta$, that should be used for design, assuming cast-in-place footings placed on properly prepared subgrade materials. These represent unfactored values; a factor of 0.8 is to be applied in calculating the horizontal resistance.

<i>Subgrade Material</i>	<i>Coefficient of Friction, $\tan \delta$</i>
Native sand/silt soils (value provided for siltier soils, since composition and distribution of soils is variable)	0.45
Engineered fill (Granular “A” or “B” Type II)	0.58
Bedrock or mass concrete	0.7

For footings supported on the bedrock, the sliding resistance can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the intact rock mass is essentially as strong as or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the unfractured (intact) bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. For uplift of the dowels, a factored value of 700 kPa may be assumed for the grout-to-rock bond stress for ULS design, and the upper 0.5 m of the bond length should be ignored in the calculation of required bond length since the rock near surface is more likely to be weathered or disturbed. The dowel design should also be checked to ensure that the conical rock mass mobilized around the dowel is sufficient to support the design loading. The actual bond stress along the rock-grout interface may vary from the design value given and it should, therefore, be verified in the field by pull-out testing. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Document to specify the installation, materials and testing of the dowels.

For the buttressed crash walls around the perimeter of the sand/salt storage building, it is understood that tie-rods are being considered beneath the slab-on-grade, extending east-west to connect opposing buttress walls, to resist the lateral pressures exerted on the crash walls by the sand/salt stockpiles. If tie-rods are used for the lateral support system, the tie-rods will need to be designed to accommodate the anticipated deformations due to the sand/salt stockpile loading (see Section 6.7.2) and the presence of the proposed subfloor drainage collection system (i.e. drainage pipes and geomembrane).

As an alternative to the use of tie-rods, the following suggestions / recommendations are provided for consideration in the design of the lateral support system for the crash walls, particularly at the north wall of the sand/salt storage building where the distance to the opposing wall will make the use of tie-rods less practical:

- Increase the wall foundation depth (i.e. extend deeper than the minimum 1.7 m depth for frost protection purposes). This will increase the overburden pressure, and hence sliding resistance, and allow for some contribution from passive resistance in the zone below the frost penetration depth. (Currently, it is recommended that any contribution from passive resistance be neglected within the frost action zone.)
- Place fill soils or insulation against and extending out from the exterior of the crash wall between buttress walls for frost protection purposes, thereby reducing the depth of frost penetration in the soils adjacent to the wall and allowing for an increase in the passive resistance that can be taken into account in the design. If this option is considered, weepers or a drainage collection system on the exterior wall would be required if fill soils are present above the final floor slab elevation.
- Increase the size and frequency of buttress walls.
- Incorporate tie-back anchors, micropiles or helical piles installed within the underlying granular soil or bedrock to resist lateral loads.

6.3.2 Steel H-Pile Foundations

Steel H-piles may be considered for support of structures where the depth to bedrock (including the thickness of both the engineered fill and the native soils) is greater than approximately 5 m; this will allow for a minimum pile length of 3.3 m, based on the pile cap being constructed with a minimum cover of 1.7 m below lowest surrounding grade for frost protection purposes. Steel H-piles are not feasible in the northern portion of the garage, where the bedrock is relatively shallow, unless blasting and subexcavation of the bedrock is carried out in this area; such operations would be expensive and are considered unnecessary since the proposed engineered fill, native soils and bedrock at the site are suitable for support of spread footings.

Based on the subsurface conditions encountered during the investigation, the pile length is expected to vary from about 3 m to 19 m below the pile cap level. Provision would be required in the contract for dealing with varying pile lengths due to the variable bedrock surface.

Based on the investigation the bedrock typically slopes downward from the northwest to the southeast, typically varying from about 1.5H:1V to 3H:1V; however, more steeply sloping bedrock should be expected. Consideration must be given to potential difficulties driving the piles due to the presence of cobbles and boulders within the native deposits and the sloping bedrock at this site. As a result, piles should be fitted with appropriate driving shoes (i.e. Titus

“Rock Injector Design” or equivalent). If piles are adopted, a Non-Standard Special Provision should be included in the Contract Documents to warn the contractor of the sloping bedrock and of the presence of cobbles and boulders at the site.

6.3.2.1 Axial Geotechnical Resistance

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

6.3.2.2 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, which is likely required at this site. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

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

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of horizontal subgrade reaction (MPa/m);} \\ z \text{ is the depth (m); and} \\ B \text{ is the pile diameter (m).} \end{array}$$

The following table provides the recommended range for the value of n_h to be used in the structural analysis. The range in values reflects the variability in the subsurface conditions and values used will depend on the design elevation of the pile cap. Design values are provided for the full stratigraphic sequence at the site.

<i>Soil Unit</i>	<i>Approximate Depth</i>	<i>n_h</i>
Above the Water Table: Engineered fill above existing ground surface	Above 0 m	10 MPa/m
Compact to very dense native sandy soils	0 – 3 m	15 MPa/m
Below the Water Table: Compact to very dense sand to silt soils	Below 3 m	10 MPa/m

A maximum factored lateral resistance of 60 kN at ULS and 30 kN at SLS is recommended for vertical HP 310x110 piles driven with a minimum embedment length of 3 m within the existing compact to dense sandy soils or within compacted granular fill placed after bedrock sub-excavation. Higher lateral capacities can be achieved for greater pile embedment depths.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R, as follows:

<i>Pile Spacing in Direction of Loading d = Pile Diameter</i>	<i>Reduction Factor (R)</i>
8d	1.0
6d	0.7
4d	0.4
3d	0.25

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

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

6.4 Lateral Earth Pressures for Design of Retaining Walls

The following information is provided concerning the design of walls at the site that will be required to support unbalanced lateral earth pressures (i.e. retaining walls or combined foundation/retaining walls).

- Backfill to the foundation/retaining walls should consist of granular fill meeting the specifications for Ontario Provincial Standard Specification (OPSS) Granular “A” or Granular “B” Type II, but with less than 5 per cent passing the No. 200 sieve. As a minimum requirement, the granular backfill should be placed in the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the structure's footing. Filtered longitudinal drains should be installed at the base of the fill to provide positive drainage of the granular backfill.
- The backfill should be placed and compacted in accordance with MTO's Special Provision SP105S10. Heavy compaction equipment should not be used within the lateral distance behind any structure equal to the current height of the fill above the base of the structure.
- Combined foundation/retaining walls that are part of the building's structural support and that are horizontally restrained are not anticipated to yield sufficiently for active earth pressure conditions to occur. In this case, the appropriate geotechnical design parameters (for a triangular lateral earth pressure distribution) are as follows:

$$\text{Unit weight of granular backfill} = \gamma = 21 \text{ kN/m}^3$$

$$\text{Unit weight of water} = \gamma_w = 9.8 \text{ kN/m}^3$$

$$\text{Lateral earth pressure coefficient} = K_o = 0.5$$

$$\begin{array}{l} \text{Coefficient of friction between} \\ \text{concrete footings and native soils} \\ \text{or engineered fill} \end{array} = \mu = 0.58$$

$$\begin{array}{l} \text{Coefficient of friction between} \\ \text{concrete footings and bedrock} \\ \text{or mass concrete} \end{array} = \mu = 0.7$$

- Active earth pressure design parameters would be applicable where a retaining wall/structure is unrestrained and free to rotate about its base. For these conditions, an active lateral earth pressure coefficient (K_a) of 0.3 can be used in design.
- If the wall support and structure allow lateral yielding of the wall, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. If the wall movement will exceed the following criteria, active earth pressures will develop and an unrestrained structure should be assumed:
 - Rotation of approximately 0.002 about the base of a vertical wall;
 - Horizontal translation of 0.001 times the height of the wall; or
 - A combination of both.

- The at-rest and active earth pressure coefficients provided above assume that the walls are backfilled with compacted Granular “A” or “B” as identified above, and that the backfill surface is horizontal (i.e. not sloping).
- Any anticipated surcharge loading on the floor slab adjacent to the retaining wall should be included in the design.

6.5 Earthquake Considerations

The proposed structures should be designed to resist a minimum earthquake (seismic) force “V” which, according to the Ontario Building Code (OBC), is equal to the product of:

$$V = v S I F W$$

The parameters that are related to geotechnical considerations are the zonal velocity ratio, v , and the foundation factor, F . From Table 2.5.1.A of the OBC, the zonal velocity ratio for the Gravenhurst area is 0.05. The foundation factor for bearing soils at this site may be taken to be 1.0 from Table 4.1.9.C of the OBC. These parameters should be reviewed by the structural engineer.

6.6 Floor Slabs

It is understood that concrete floor slabs may be adopted within the garage, wash bay, office building and in the drive-through portion of the sand/salt structure; the majority of the sand/salt structure, however, will be asphalt-paved. According to the designer, the finished floor slab elevation will be approximately 257.1 m. The building floor slabs are anticipated to be supported on native soil deposits or on engineered fill materials that should be placed and compacted as described in Section 6.2. The final lift of granular fill beneath the floor slabs should consist of a minimum thickness of 150 mm of OPSS Granular “A” material, uniformly compacted to 100 percent of its Standard Proctor maximum dry density.

The floor slabs should be structurally separate from the foundation walls and columns and saw-cut control joints should be provided at regular intervals and along column lines to minimize shrinkage cracking and to allow for normal differential settlement of the floor slabs.

Where the interior ground floor slab / pavement is at or above the level of the exterior final grade, no perimeter drainage at the footing level is required.

Based on conversations with the designer, a permanent sub-floor drainage system is recommended to collect salt-bearing water and convey it to a holding tank. It is anticipated that the drainage system would consist of a system of floor drains and collection pipes draining to

designated holding tanks for treatment and/or disposal. In order to minimize contamination into the native soils by run-off water containing a high concentration of salts, a barrier is recommended below the sand/salt storage area, and below other structure areas as may be required by the hydrogeological studies for the site. Consideration has been given to the use of compacted low-permeability clay (i.e. bentonite) and to the use of geosynthetics (i.e. geosynthetic clay liner, geomembrane, etc.). The use of a geomembrane is recommended for this site over compacted clay products in order to improve the performance of the barrier and floor slab system. Details of the geomembrane specifications, installation and quality control / assurance procedures should be provided by the designer of the drainage collection system. Geotechnical recommendations related to the installation of the geomembrane / drainage system are provided below.

The geomembrane should be installed below the floor slab / pavement structure and above any lateral restraint (e.g. tie-rods) system. A minimum 75 mm layer of sand fill should be placed below the geomembrane and above the subgrade in order to protect the geomembrane from angular gravel/cobble pieces that may be contained in the native soils or engineered fill. A minimum 300 mm thick layer of sand should be placed directly on top of the geomembrane in order to protect it from the overlying pavement structure. This sand fill should be in accordance with OPSS 1004 – Winter Sand, and may be available from the existing MTO Patrol Yard south of the site. The use of a geotextile (instead of sand) adjacent to the geomembrane was considered; however, given the low interface strength between the smooth geomembrane and geotextile; the factor of safety against global instability due to the sand stockpile for this configuration was inadequate.

Special care must be taken when placing the sand on top of the geomembrane, including spreading and compacting the sand with a low ground-pressure bulldozer (maximum ground pressure of 35 kPa) to at least 95 per cent of the material's Standard Proctor maximum dry density. A minimum 300 mm thick separation distance must be maintained between the bulldozer tracks and the top of the geomembrane. Truck traffic should be restricted to thickened areas of the cover soil layer by providing a minimum separation distance of 900 mm between truck tires and the underlying geomembrane. An example of an NSSP for placement and compaction of soils above the geomembrane is included in Appendix C.

6.7 Stability and Settlement in Sand/Salt Storage Area

6.7.1 Stability

Stability analyses have been performed for the maximum height sand/salt storage pile of 12.8 m, using the commercially available program SLOPE/W produced by Geo-Slope International Ltd. Effective stress parameters were employed in the analysis, based on the results of the Standard

Penetration Tests tempered by engineering judgment based on precedent experience in similar soils. The following table summarizes the strength parameters and unit weights employed for the different materials.

<i>Material</i>	<i>Unit Weight (kN/m³)</i>	<i>Strength Parameters</i>
Fill (sand and gravel)	21 to 22	$c' = 0$ kPa $\phi' = 32^\circ$ to 35°
Geomembrane	19	$c' = 0$ kPa $\phi' = 16^\circ$
Compact to very dense sand to silt	18	$c' = 0$ kPa $\phi' = 35^\circ$

NOTE: The internal friction angle for the geomembrane is based on information supplied by Layfield Geosynthetics.

The stability analyses assume that all topsoil and native soils containing organics have been removed prior to construction, that the concrete side walls in the sand/salt storage area have a minimum founding depth of 1.7 m, that the proposed geomembrane / sand interface has a minimum internal angle of friction of 16 degrees, and that there is a minimum granular sub-base thickness of 0.8 m between the slab / pavement structure and the geomembrane layer.

The results of the stability analyses are as follows:

- A factor of safety of greater than 1.5 is obtained for a deep-seated, global type failure surface that could impact the stability of the perimeter walls for the sand/salt storage structure. The result from a selected stability analysis is presented on Figure 1.
- A factor of safety of 1.3 is obtained for the stockpiled sand sliding along the geomembrane layer. The result from a selected stability analysis is presented on Figure 2.

6.7.2 Settlement

Settlement analyses have been performed for the sand/salt stockpile loading, assuming a maximum stockpile height of 12.8 m. The bedrock surface in the sand/salt storage area varies from about 5.8 m to 12.8 m depth below the existing ground surface, as encountered at the borehole locations. The site soils are cohesionless, with the exception of a thin (0.3 m thick) layer of stiff clayey silt that was encountered in Boreholes F7 and MW5. Based on the generally cohesionless nature of the soils at the site, significant long-term consolidation settlements are not anticipated.

It is noted that settlement will occur within the native soils as a result of placement of the engineered fill at the site; this will be limited to less than about 5 mm to 10 mm (depending on the thickness of the engineered fill and of the native soils) and will occur rapidly during placement of the engineered fill.

The elastic compression of the compact to very dense sand, silty sand and sand and silt subsoils under the sand/salt stockpile loading has been modelled using elastic moduli of deformation based on the measured SPT “N” values and correlations proposed by Bowles (1984) and CHBDC. The stockpile loadings have been assumed based on a maximum sand pile height of approximately 12.8 m at the centre and 4.5 m along the sides. The following table presents the estimated settlement of the foundation soils and the backcalculated moduli of vertical subgrade reaction (k_s) values due to the stockpile loading.

<i>Proposed Structure Area</i>	<i>Assumed Contact Stress</i>	<i>Interpolated Overburden Thickness</i>	<i>Estimated Settlement of Foundation Soils</i>	<i>Moduli of Vertical Subgrade Reaction (k_s) Value</i>
West Edge of Sand Storage Area	80 kPa	6 m	10 mm	8.0 kPa/mm
Centre of Sand Storage Area	230 kPa	8.5 m	30 mm	7.7 kPa/mm
East Edge of Sand Storage Area	80 kPa	12 m	20 mm	4.0 kPa/mm

These elastic movements are expected to occur rapidly in response to changes in the stockpile height, based on the granular nature of the native soils.

Since k_s is roughly inversely proportional to the loading width, these values should not be used for areas / configurations of different sizes, nor should they be used in different areas with different contact stresses.

6.8 Construction Considerations

6.8.1 Temporary Excavations

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. It is anticipated that the majority of the excavations will be associated with footing construction and will therefore extend to about 1.7 m depth within either native soils or engineered fill; for this depth, the excavations are expected to be maintained above the groundwater level. Deeper excavations (such as for installation of wash bay tanks) may extend below the groundwater level at the site. The typically compact to very dense sandy soil above the water table is classified as Type 2 soil according to the OHSA; however, all fill materials are classified as a Type 3 soil, and

the native soils below the water table would be classified as Type 4 soil unless a suitable dewatering system is installed to lower the water level below the base of the excavation. Temporary excavations above the water table may be made with side slopes no steeper than 1H:1V. Where excavations extend below the groundwater table at the site, the temporary side slopes will have to be formed at 3H:1V unless proper groundwater control is implemented.

It should be noted that the water levels in this area have been known to fluctuate up to 2 m depending on the time of year. It is recommended that excavations for foundations and/or the wash bay tank be carried out in late summer when water levels are anticipated to be lower.

If temporary excavation support is required at this site, it should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 105S19.

6.8.2 Groundwater Control

The subsurface conditions at the site generally consist of fine sand to silt above an undulating bedrock surface. If shallow foundations are adopted, it is expected that the foundation excavations will be terminated within the native sandy soil or within engineered fill above the groundwater table at the site. Where excavations extend to more than 2 m below the original ground surface at the site, there is potential to intercept the water table (particularly during wetter periods of the year). In these areas, groundwater control, such as a well-point or vacuum well-point dewatering systems, will be required in order to control the excavation base and side slopes.

Excavations that extend to or near to the surface of the bedrock may also encounter groundwater seepage from water perched above the bedrock surface. Groundwater control, such as the use of perimeter trenches and sumps within the excavation, will be required in order to achieve a dry and stable subgrade and/or to maintain stable excavation sides.

Perimeter drainage should be provided for the underground wastewater tank. According to the designer, the base of the tank is about 4 m below finished grade and based on the water levels observed in the boreholes, the base of the tank is below the groundwater table. It should be noted that water levels will fluctuate with seasonal variation and may periodically rise higher than the levels presented in this report. As an alternative to perimeter drainage, a structural anchorage system could be designed to account for the fluctuating water levels.

6.8.3 Obstructions

The soils at the site may contain cobbles and boulders, particularly near the soil/bedrock interface. Conventional excavation equipment should be suitable for the majority of excavation

through the on-site soils; however, the presence of rock fragments or boulders may interfere with or slow the progress of excavation at some locations. The presence of cobbles and boulders could affect the installation of steel H-piles, if these are adopted for support of the structures, and it is recommended that an NSSP be included in the Contract Documents to warn the Contractor of this condition.

6.8.4 Summary of Required OCs / NSSPs

To summarize the preceding discussions, it is recommended that the following Operational Constraints (OCs) or Non-Standard Special Provisions (NSSPs) be provided in the Contract Documents to address geotechnical/foundations concerns during construction at this site:

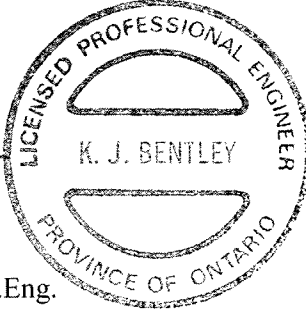
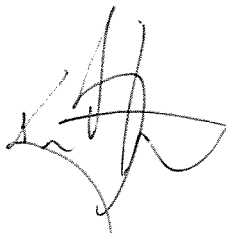
- ☐ NSSP regarding placement of a lean concrete working mat on the foundation subgrade immediately following inspection of the prepared subgrade, to protect the sand/silt soils from disturbance and degradation.
- ☐ NSSP concerning dewatering of the native soils during excavation and foundation construction.
- ☐ NSSP for supply and installation of sand fill above the geomembrane, and to warn the Contractor of restricted construction activities above the geomembrane.
- ☐ NSSP to address installation, materials and testing of dowels, for spread footings dowelled into the bedrock. (A draft NSSP has been prepared for this item, but it is understood from the designers that dowelling of spread footings into the bedrock is not planned at this time.)
- ☐ NSSP to warn the contractor of the sloping bedrock and the presence of cobbles and boulders within the native sandy soils, if H-pile foundations are adopted at this site. (A draft NSSP has been prepared for this item, but it is understood from the designers that H-pile foundations will not be adopted at this site.)

Draft OCs or NSSPs for are included in Appendix C. All OCs/NSSPs will be revised as the design and planning of construction staging progress.

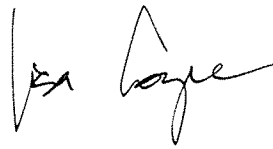
7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Shannon Palmer, EIT, and Mr. Kevin Bentley, P.Eng., and was reviewed by Ms. Lisa Coyne, P.Eng., and Mr. Kevin Nelson, P.Eng., both Associates and geotechnical engineers with Golder. Mr. Fintan Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted a quality control review of the report.

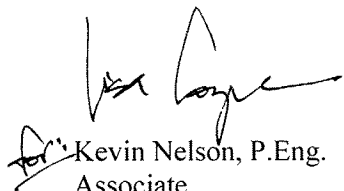
GOLDER ASSOCIATES LTD.




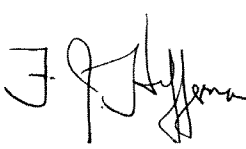
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TABLE 1
SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO.:05-1111-029

TITLE: MTO Patrol Yard, Gravenhust

DATE: March 2, 2006

Borehole Number	Sample Number	Sample Depth (m)	Bedrock Type	Test Type	Core Length (mm)	Core ⁽²⁾ Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	Is (50mm) (MPa)	Approx. ⁽¹⁾ UCS (MPa)
F-3	1	10.8	Granite Gneiss	D	71.2	63.1		15079.4	14.58		3.660	4.065	93
	2	11.0	Granite Gneiss	A	72.3	62.7	76.00	25801.1	24.95	4.320		5.215	120
	3	11.3	Granite Gneiss	D	79.0	62.7		22801.8	22.05		5.602	6.205	143
F-5	1	3.8	Granite Gneiss	A	67.4	62.8	73.44	35660.9	34.49	6.394		7.602	175
	2	4.5	Granite Gneiss	D	103.5	62.9		20423.0	19.75		4.986	5.530	127
F-6	1	5.1	Biotite Gneiss	A	66.3	62.8	72.79	19161.2	18.53	3.498		4.142	95
	2	5.7	Biotite Gneiss	D	159.0	62.7		18099.4	17.50		4.447	4.925	113
F-8	1	6.1	Granite Gneiss	D	129.0	62.7		27662.7	26.75		6.797	7.528	173
	2	6.3	Granite Gneiss	A	71.1	63.1	75.58	23139.6	22.38	3.917		4.718	109
	3	6.4	Granite Gneiss	A	65.4	63.1	72.52	25159.9	24.33	4.627		5.470	126
	4	6.7	Biotite Gneiss	A	63.2	60.9	70.01	20023.1	19.36	3.951		4.597	106

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Consistency

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 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.

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

- Notes:** 1 $\tau = c' + \sigma' \tan \phi'$
 2 shear strength = (compressive strength)/2
 * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

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

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

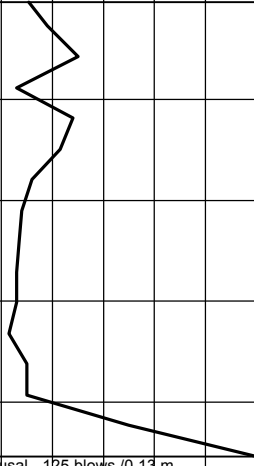
B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To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

PROJECT 05-1111-029				RECORD OF BOREHOLE No F1				1 OF 2 METRIC						
W.P. 5420-02-00				LOCATION N 4973532.1 ; E 316018.0				ORIGINATED BY SB						
DIST 52 HWY 11				BOREHOLE TYPE Power Auger, 108 mm I.D. Hollow Stem Auger				COMPILED BY SP						
DATUM Geodetic				DATE January 23, 2006				CHECKED BY KJB						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
254.0	GROUND SURFACE													
0.0	Topsoil													
253.6														
0.4	Silty SAND, some gravel, containing cobbles Very dense Brown Moist		1	SS	50/2		253							
252.5														
1.5	SAND, trace to some gravel and silt Compact to very dense Brown Moist		2	SS	70		252							
			3	SS	50		251							
			4	SS	55		250							
			5	SS	52		249							
			6	SS	54		248							
			7	SS	38		247							
			8	SS	41		246							
			9	SS	26		245							
			10	SS	31		244							
242.7	End of Borehole Start of Dynamic Cone Penetration Test (DCPT)						243							
11.3							242							
							241							
							240							

Continued Next Page

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

MIS-MTO 001 051111029.GPJ GAL-MISS.GDT 3/8/06 DD

PROJECT <u>05-1111-029</u>										RECORD OF BOREHOLE No F1										2 OF 2 METRIC									
W.P. <u>5420-02-00</u>					LOCATION <u>N 4973532.1 ; E 316018.0</u>					ORIGINATED BY <u>SB</u>																			
DIST <u>52</u> HWY <u>11</u>					BOREHOLE TYPE <u>Power Auger, 108 mm I.D. Hollow Stem Auger</u>					COMPILED BY <u>SP</u>																			
DATUM <u>Geodetic</u>					DATE <u>January 23, 2006</u>					CHECKED BY <u>KJB</u>																			
SOIL PROFILE					SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT					UNIT WEIGHT					REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION				STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)					γ kN/m ³	GR SA SI CL							
										20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 10 20 30														
234.5	--- CONTINUED FROM PREVIOUS PAGE ---									238																			
19.5	End of DCPT Notes: 1. Water level at 3.1 m depth below ground surface upon completion of drilling. 2. Water level in piezometer at 3.1 m depth (Elevation 250.8m) on Jan. 24, 2006. 3. Water level in piezometer at 3.7m depth (Elevation 250.3m) on February 6, 2006.									235	Refusal - 125 blows / 0.13 m																		

PROJECT		5420-02-00		LOCATION		N 4973538.8 ; E 315978.0		ORIGINATED BY		SB								
DIST		52		HWY		11		BOREHOLE TYPE		Power Auger, 108 mm I.D. Hollow Stem Auger								
COMPILED BY		SP		DATE		January 23, 2006		CHECKED BY		KJB								
SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	20			40	60	80	100	20					
255.6	GROUND SURFACE																	
0.0	Topsoil																	
255.3																		
0.3	SAND, trace to some gravel, trace silt and organics Very dense Brown Moist		1	SS	55/0.15													
254.1																		
1.5	Sandy SILT, trace to some gravel, trace clay Dense to very dense Brown Moist to wet		2	SS	45													
			3	SS	33													
	Grey at 3.1 m depth																	
251.9			4	SS	69/0.22													
3.7	End of Borehole Auger Refusal																	
	Note: 1. Borehole dry upon completion of drilling operations.																	

PROJECT 05-1111-029			RECORD OF BOREHOLE No F3			1 OF 1 METRIC											
W.P. 5420-02-00			LOCATION N 4973550.6; E 315919.7			ORIGINATED BY SB											
DIST 52 HWY 11			BOREHOLE TYPE Power Auger, 108 mm I.D. Hollow Stem Auger			COMPILED BY SP											
DATUM Geodetic			DATE January 25, 2006			CHECKED BY KJB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ	GR SA SI CL
257.2	GROUND SURFACE							20 40 60 80 100									
0.0	Topsoil																
256.8																	
256.4	SAND, trace gravel, roots and organics, contains cobbles Dark brown Moist		1	SS	50												
0.8	SANDY SILT to SILT, trace to some sand, trace gravel and clay Compact to very dense Brown to grey Moist to wet		2	SS	50												
			3	SS	39												0 10 85 5
			4	SS	37												
			5	SS	31												
			6	SS	28												
251.4																	
5.8	SAND and GRAVEL, trace silt and clay Very dense Brown Moist to wet		7	SS	64												
			8	SS	84												33 60 (7)
			9	SS	100/25												
247.1	GRANITE GNEISS (BEDROCK) Slightly weathered to fresh, strong to very strong, Fine to coarse crystalline, Foliated Black/White/Pink. Bedrock cored from 10.1m to 13.0m. For bedrock coring details refer to Record of Drillhole F3.																
10.1																	
244.2	End of Borehole																
13.0	Note: 1. Water level in piezometer at 5.5 m depth below ground surface after installation. 2. Water level measured in piezometer at 5.2m depth (Elevation 252.0m) on February 6, 2006.																

PROJECT: 05-1111-029

RECORD OF DRILLHOLE: F3

SHEET 1 OF 1

LOCATION: N 4973550.6 ;E 315919.7

DRILLING DATE: January 25, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLLOID % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate										BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage										PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular										PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break										BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
									RECOVERY					R.Q.D. %	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY K, cm/sec					Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
									TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT INDEX PER 0.3 m	B Angle			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	K, cm/sec	K, cm/sec	K, cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
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DEPTH SCALE

1 : 50





LOGGED: SB

CHECKED: KJB

MIS-RCK 004 051111029.GPJ GAL-MISS.GDT 3/8/06 DD

PROJECT		RECORD OF BOREHOLE No F4				1 OF 1 METRIC											
W.P.		LOCATION				ORIGINATED BY											
DIST		BOREHOLE TYPE				COMPILED BY											
DATUM		DATE				CHECKED BY											
05-1111-029		N 4973566.2; E 315920.8				SB											
5420-02-00		Power Auger, 108 mm I.D. Hollow Stem Auger				SP											
52 HWY 11		January 26, 2006				KJB											
Geodetic																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30	kN/m ³			
256.8	GROUND SURFACE																
0.0	SAND and SILT, trace clay, gravel, roots and organics Dark brown Moist																
256.1																	
0.8	SANDY SILT to SILT, some sand, trace clay, containing roots above 1.5 m Dense to very dense Brown Moist		1	SS	45		256										
			2	SS	71		255										0 20 78 2
254.7	Auger Refusal End of Borehole																
2.1	Note: 1. Borehole dry upon completion of drilling operations.																

PROJECT <u>05-1111-029</u>		RECORD OF BOREHOLE No F5				1 OF 1 METRIC											
W.P. <u>5420-02-00</u>		LOCATION <u>N 4973559.6 ; E 315956.7</u>				ORIGINATED BY <u>SB</u>											
DIST <u>52</u> HWY <u>11</u>		BOREHOLE TYPE <u>Power Auger, 108 mm I.D. Hollow Stem Auger</u>				COMPILED BY <u>SP</u>											
DATUM <u>Geodetic</u>		DATE <u>January 25, 2006</u>				CHECKED BY <u>KJB</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
256.3	GROUND SURFACE							20	40	60	80	100					
0.0	SAND, some gravel, containing roots, organics and cobbles																
255.7	Dark brown Moist																
0.6	SAND and SILT, trace gravel and clay, containing cobbles		1	SS	67												
	Very dense Brown to grey Moist																
254.5			2	SS	80/0.25												
1.8	GRANITE GNEISS (BEDROCK) Slightly weathered to fresh Very strong Fine to coarse crystalline, Foliated Black/white/pink Bedrock cored from 1.8 m to 4.7 m. For bedrock coring detail see Record of Drillhole F5.																
251.6	End of Borehole																
4.7	Note: 1. Borehole dry upon completion of drilling operations.																

PROJECT: 05-1111-029

RECORD OF DRILLHOLE: F5

SHEET 1 OF 1

LOCATION: N 4973559.6 ;E 315956.7

DRILLING DATE: January 25, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	CORRELATION										HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q/ AVG.	NOTES WATER LEVELS INSTRUMENTATION						
									JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate				BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage				PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular						PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break		BR - Broken Rock			
									RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA				TYPE AND SURFACE DESCRIPTION						DIP w.r.t. CORE AXIS		NOTE: For additional abbreviations refer to list of abbreviations & symbols.			
									TOTAL CORE %	SOLID CORE %			B Angle															
		CONTINUED FROM PREVIOUS PAGE		254.50 1.80																								
2		GRANITE GNEISS (BEDROCK) Slightly weathered to fresh Very strong Fine to coarse grained crystalline Foliated Black/white/pink																										
3																												
4																												
5		End of Drillhole		251.60 4.70																								
6																												
7																												
8																												
9																												
10																												
11																												

DEPTH SCALE

1 : 50



LOGGED: SB

CHECKED: KJB

MIS-RCK 004 051111029.GPJ GAL-MISS.GDT 3/8/06 DD

PROJECT		05-1111-029		RECORD OF BOREHOLE No F6		1 OF 1 METRIC												
W.P.		5420-02-00		LOCATION		N 4973563.2 ; E 315996.7												
DIST		52		HWY		11												
BOREHOLE TYPE		Power Auger, 108 mm I.D. Hollow Stem Auger		COMPILED BY		SP												
DATUM		Geodetic		DATE		January 24, 2006												
CHECKED BY		KJB																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30	kN/m ³				
254.9	0.0	GROUND SURFACE																
254.6	0.3	Topsoil																
254.3	0.6	SAND, trace gravel and roots, containing cobbles Dark brown		1	SS	79		254										
		SAND, trace to some silt and clay, trace gravel, containing cobbles in upper 1.2 m Dense to very dense Brown Moist		2	SS	62		253										
				3	SS	56		252										
		Wet at 3.1 m depth		4	SS	37		251										
				5	SS	50/15		250										
250.6	4.3	Auger Refusal																
		BIOTITE GNEISS (BEDROCK) containing quartz-rich banding Slightly weathered to fresh Strong to very strong Foliated Black/white																
		Bedrock cored from 4.3 m to 7.3 m.																
		For bedrock coring detail see Record of Drillhole F6.																
247.6	7.3	End of Borehole																
		Note: 1. Water level inside casing at 2.0 m below ground surface upon completion of drilling.																

PROJECT: 05-1111-029

RECORD OF DRILLHOLE: F6

SHEET 1 OF 1

LOCATION: N 4973563.2 ; E 315996.7

DRILLING DATE: January 24, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate										BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage										PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular										PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break										BR - Broken Rock										NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
								RECOVERY										R.Q.D. %	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY K, cm/sec										Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
								TOTAL CORE %	SOLID CORE %	R.O.D. %	B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	10	10	10	10			10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10			10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10		10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	

DEPTH SCALE

1 : 50



LOGGED: SB

CHECKED: KJB

MIS-RCK 004 051111029.GPJ GAL-MISS.GDT 3/8/06 DD

PROJECT 05-1111-029			RECORD OF BOREHOLE No F7			1 OF 1 METRIC		
W.P. 5420-02-00			LOCATION N 4973583.8 ; E 316001.1			ORIGINATED BY SB		
DIST 52 HWY 11			BOREHOLE TYPE Power Auger, 108 mm I.D. Hollow Stem Auger			COMPILED BY SP		
DATUM Geodetic			DATE January 26, 2006			CHECKED BY KJB		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
254.4	GROUND SURFACE							PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 10 20 30
0.0	Topsoil							78.5
254.0								
253.7	SAND, trace to some silt, trace gravel containing roots and organics							
0.8	Dark brown Moist		1	SS	72			
	SAND, trace to some silt, trace clay, containing organics							
	Compact to very dense		2	SS	65			
	Brown to grey Moist							
			3	SS	45			
			4	SS	30			
			5	SS	33			
249.4			6	SS	13			
249.1	CLAYEY SILT, some sand, containing sand seams							
5.3	Stiff Brown Moist							
	Auger Refusal							
	End of Borehole							
Notes:								
1. Borehole caved to a depth of 3.3 m below ground surface.								
2. Water level in caved borehole at 2.4 m depth below ground surface.								

PROJECT		RECORD OF BOREHOLE		No F8		1 OF 1		METRIC										
W.P.		LOCATION		ORIGINATED BY		DIST		HWY										
BOREHOLE TYPE		COMPILED BY		DATE		CHECKED BY												
05-1111-029		N 4973619.6 ; E 316010.0		SB		52		11										
Geodetic		January 19, 2006		KJB														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR	SA	SI	CL
254.9	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										
0.0	Topsoil																	
254.5																		
0.3	SAND, trace to some silt, trace clay, containing wood fragments Compact to very dense Brown Moist to wet		1	SS	28		254											
			2	SS	46		253											
			3	SS	46		252											
			4	SS	57		251											
			5	SS	67		250											
			6	SS	80		249											
249.4	GRANITE/BIOTITE GNEISS (BEDROCK) Slightly weathered to fresh Very strong Fine to coarse crystalline Black/white/pink Bedrock cored from 5.5 m to 8.5 m. For bedrock coring detail see Record of Drillhole F8.						248											
5.5							247											
246.3	End of Borehole																	
8.5	Note: 1. Water level inside hollow stem augers at 2.9 m depth upon completion of drilling operations.																	

PROJECT: 05-1111-029

RECORD OF DRILLHOLE: F8

SHEET 1 OF 1

LOCATION: N 4973619.6 ;E 316010.0

DRILLING DATE: January 19, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate										BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage										PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular										PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break										BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.										NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
									RECOVERY					R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY K, cm/sec					Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
									TOTAL CORE %	SOLID CORE %	R.O.D. %	B Angle	DIP w.r.t. CORE AXIS			TYPE AND SURFACE DESCRIPTION										10 10 10 10	10 10 10 10																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
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DEPTH SCALE

1 : 50



LOGGED: SB

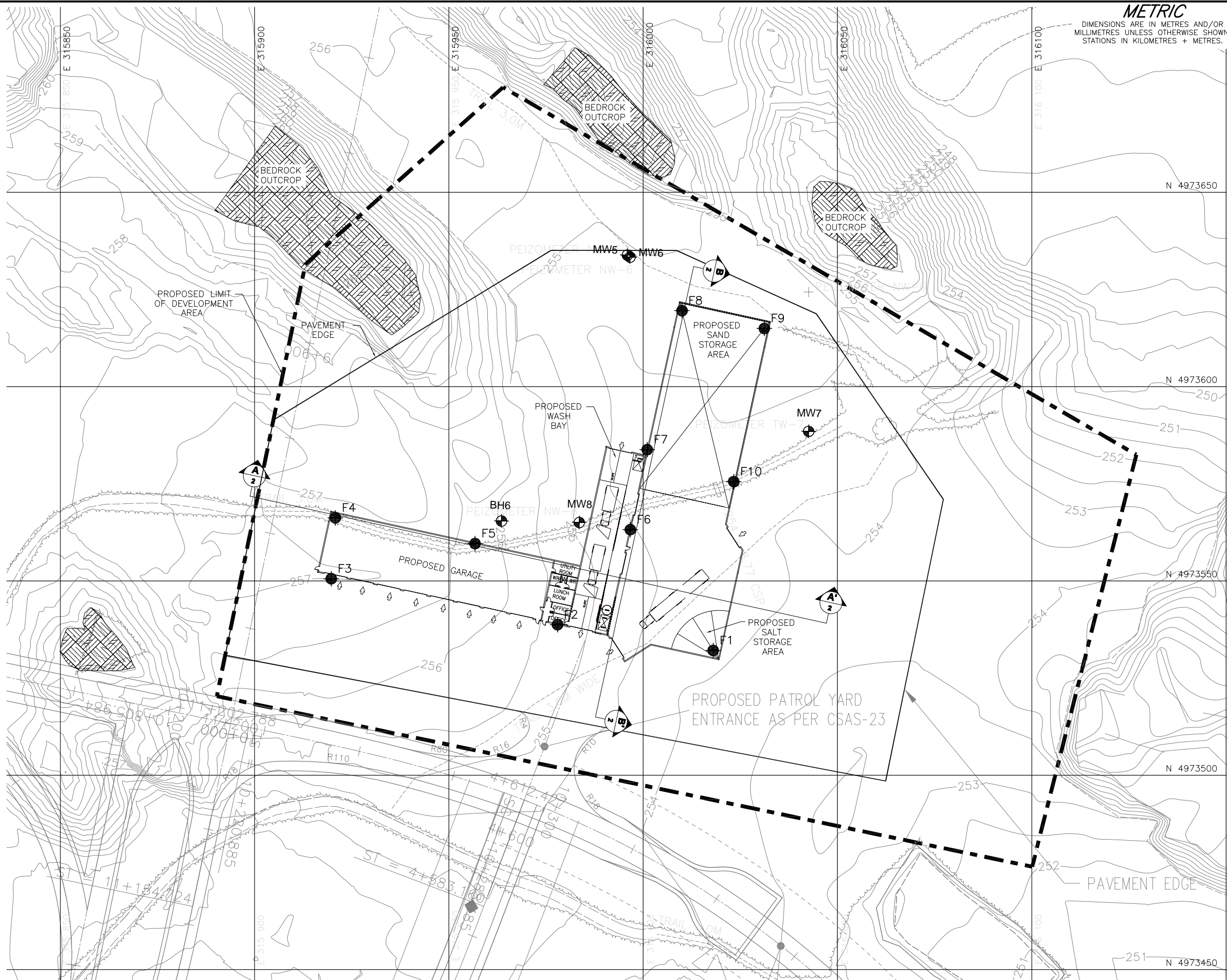
CHECKED: KJB

MIS-RCK 004 051111029.GPJ GAL-MISS.GDT 3/8/06 DD

PROJECT 05-1111-029			RECORD OF BOREHOLE No F9			1 OF 1 METRIC							
W.P. 5420-02-00			LOCATION N 4973615.0; E 316031.2			ORIGINATED BY SB							
DIST 52 HWY 11			BOREHOLE TYPE Power Auger, 108 mm I.D. Hollow Stem Auger			COMPILED BY SP							
DATUM Geodetic			DATE January 20, 2006			CHECKED BY KJB							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
254.9	GROUND SURFACE						20 40 60 80 100						
0.0	SAND, trace silt and roots Compact to very dense Brown to grey Moist		1	SS	22								
			2	SS	58								
			3	SS	49								
	Wet at 2.9 m depth		4	SS	33								
			5	SS	50								
			6	SS	64								
			7	SS	43								
			8	SS	28								
			9	SS	10								
244.2	GRAVELLY SAND, trace silt Very dense Brown Wet		10	SS	55								
10.7	End of Borehole												
243.6	Start of Dynamic Cone Penetration Test (DCPT)												
11.3	End of DCPT												
243.0	Note: 1. Water level inside caved borehole at 2.9 m depth upon completion of drilling operations.												
11.9													

PROJECT 05-1111-029			RECORD OF BOREHOLE No F10			1 OF 1 METRIC					
W.P. 5420-02-00			LOCATION N 4973575.5 ; E 316023.3			ORIGINATED BY SB					
DIST 52 HWY 11			BOREHOLE TYPE Power Auger, 108 mm I.D. Hollow Stem Auger			COMPILED BY SP					
DATUM Geodetic			DATE January 24, 2006			CHECKED BY KJB					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	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 (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
254.2	GROUND SURFACE						254				
0.0	SAND, trace silt and gravel, containing organics, roots and cobbles Very dense Brown Moist		1	SS	51		253				
252.6											
1.5	SAND, trace gravel, silt and clay Compact to very dense Brown to grey Moist		2	SS	62		252				
			3	SS	50		251				
	Wet at 3.1 m depth		4	SS	47		250				
			5	SS	37		249				
			6	SS	40		248				
			7	SS	56		247				
			8	SS	32		246				
			9	SS	26		245				
			10	SS	25		244				
242.9	End of Borehole Start of Dynamic Cone Penetration Test (DCPT)						243				
11.3							242				
241.7	End of DCPT										
12.5	Note: 1. Water level inside hollow stem augers at 3.0 m below ground surface during drilling operations.										

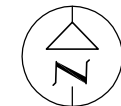
MIS-MTO 001 051111029.GPJ GAL-MISS.GDT 3/8/06 DD



PLAN
SCALE
10 0 10 20 m

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

CONT No.
WP No. 5420-02-00

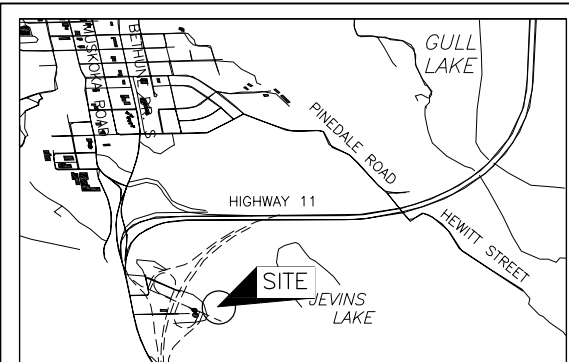


HIGHWAY 11 PATROL YARD
GRAVENHURST
BOREHOLE LOCATIONS

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



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

LEGEND

- Borehole - Current Investigation (Golder, 2006)
- Borehole - Investigation by others (Ecoplans, 2004)
- Approximate location of bedrock outcrops

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
F1	254.0	4973532.1	316018.0
F2	255.6	4973538.8	315978.0
F3	257.2	4973550.6	315919.7
F4	256.8	4973566.2	315920.8
F5	256.3	4973559.6	315956.7
F6	254.9	4973563.2	315996.7
F7	254.4	4973583.8	316001.1
F8	254.9	4973619.6	316010.0
F9	254.9	4973615.0	316031.2
F10	254.2	4973575.5	316023.3
BH6	257.0	4973565.4	315963.6
MW5	255.1	4973633.9	315995.9
MW6	255.1	4973633.3	315996.6
MW7	254.4	4973588.5	316042.6
MW8	255.0	4973565.0	315983.6

NOTES

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

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

REFERENCE

Base plans provided in digital format by MRC (Drawing File Nos. 6187-sp contour oct14-05.dwg; 05799-XA1-PROP.REQUEST.dwg; 05799-XB1-PROP.REQUEST.dwg; 05799-XN1-PROP.REQUEST.dwg and R04-0198-BH AND TP.dwg, received January 12, 2006).

NO.	DATE	BY	REVISION
Geocres No. 31D-420			
HWY. 11	PROJECT NO. 05-1111-029		DIST. 52
SUBM'D.	CHKD. KJB	DATE: OCT 2006	SITE:
DRAWN: MSM	CHKD. KJB	APPD. LCC	DWG. 1

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

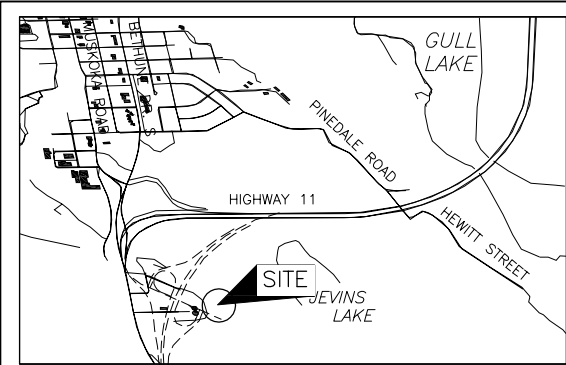
CONT No.
WP No. 5420-02-00

HIGHWAY 11 PATROL YARD
GRAVENHURST
SOIL STRATA

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



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

LEGEND

- Borehole - Current Investigation
- Borehole - Investigation by others
- 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 MMM DD, YYYY
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
F1	254.0	4973532.1	316018.0
F2	255.6	4973538.8	315978.0
F4	256.8	4973566.2	315920.8
F5	256.3	4973559.6	315956.7
F6	254.9	4973563.2	315996.7
F7	254.4	4973583.8	316001.1
F8	254.9	4973619.5	316010.0

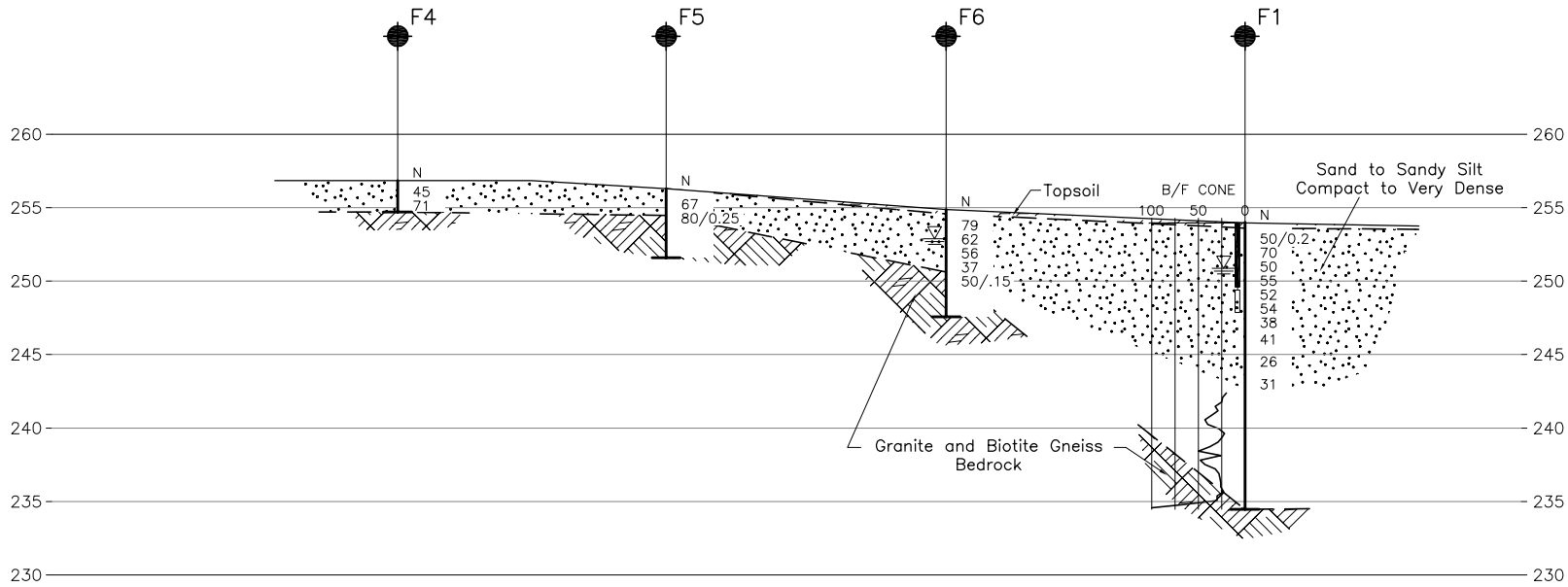
NOTES

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

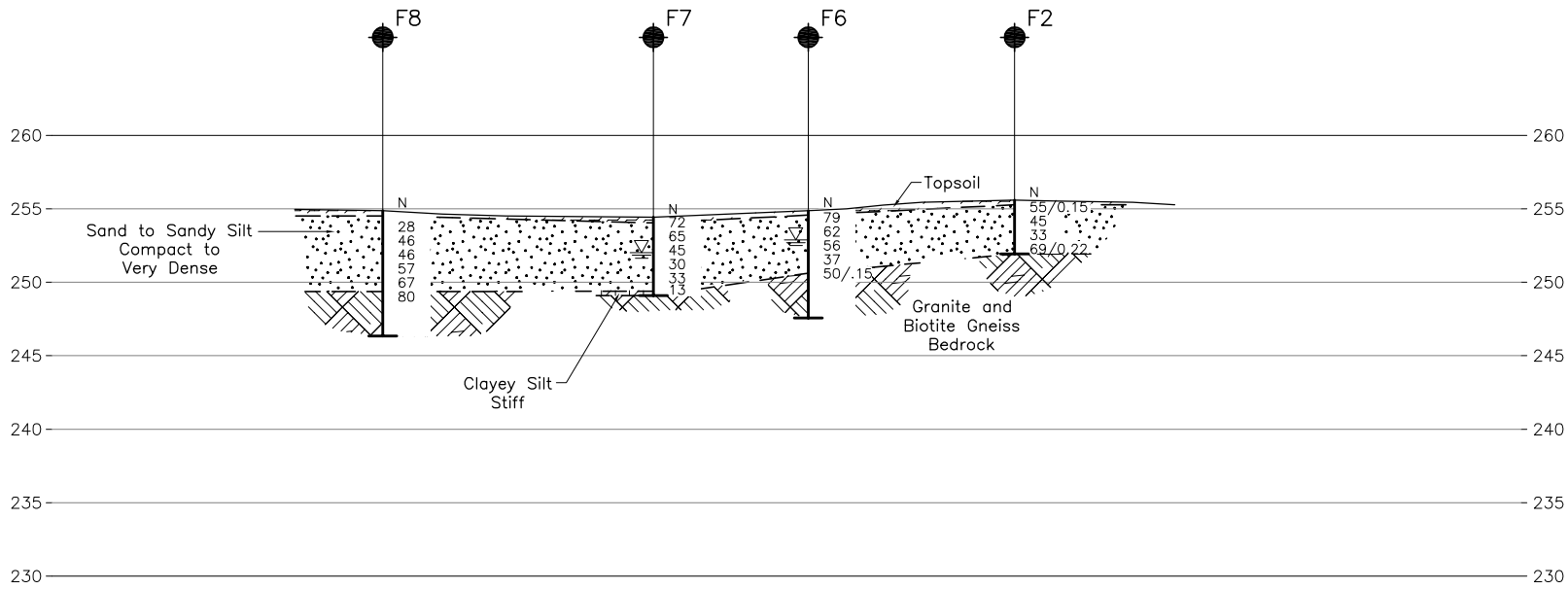
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE

Base plans provided in digital format by MRC (Drawing File Nos. 6187-sp contour oct14-05.dwg; 05799-XA1-PROP.REQUEST.dwg; 05799-XB1-PROP.REQUEST.dwg; 05799-XN1-PROP.REQUEST.dwg and R04-0198-BH AND TP.dwg, received January 12, 2006).



A-A'



B-B'

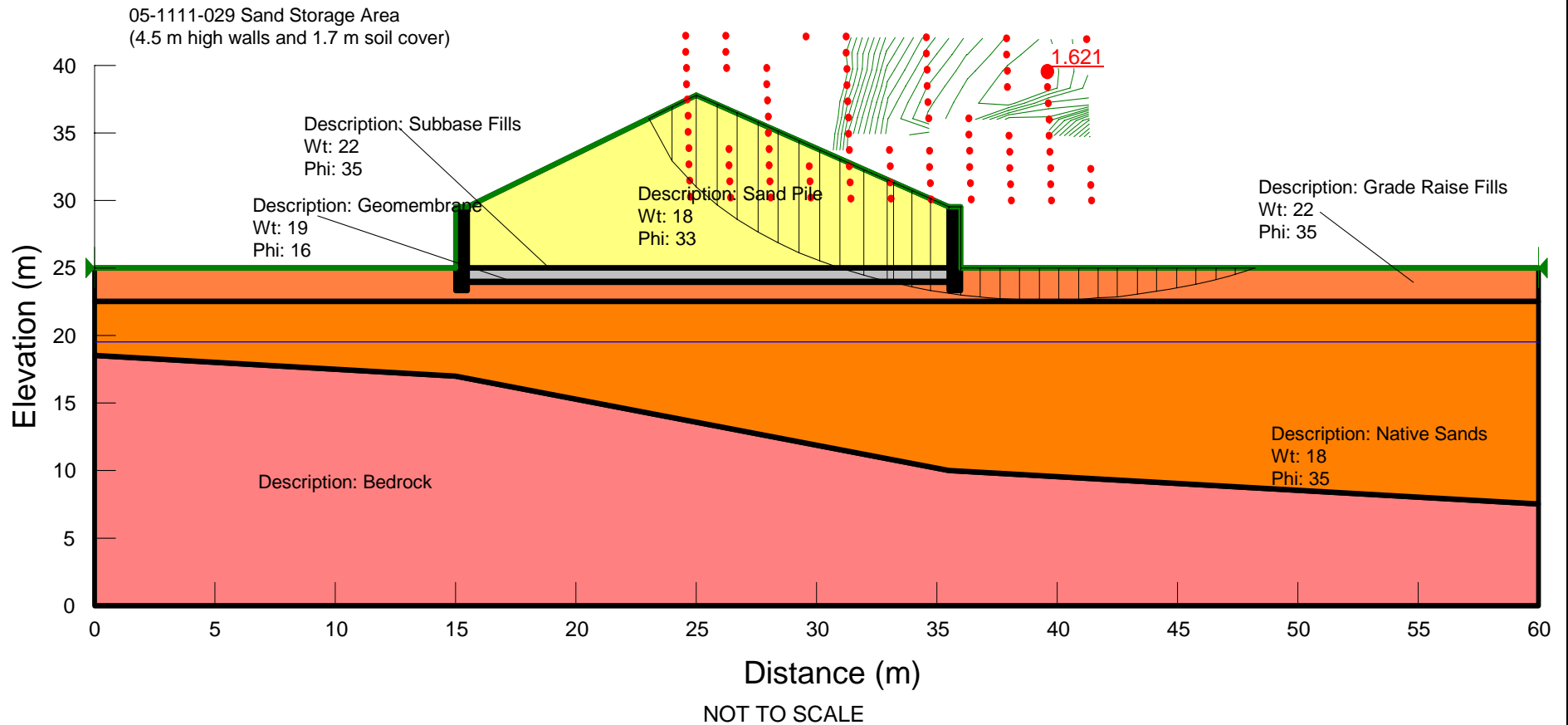
SECTIONS



NO.	DATE	BY	REVISION
Geocres No. 31D-420			
HWY. 11	PROJECT NO. 05-1111-029		DIST. 52
SUBM'D.	CHKD. KJB	DATE: OCT 2006	SITE:
DRAWN: MSM	CHKD. KJB	APPD. LCC	DWG. 2

Gravenhurst Patrol Yard Global Stability of Perimeter Walls

Figure 1



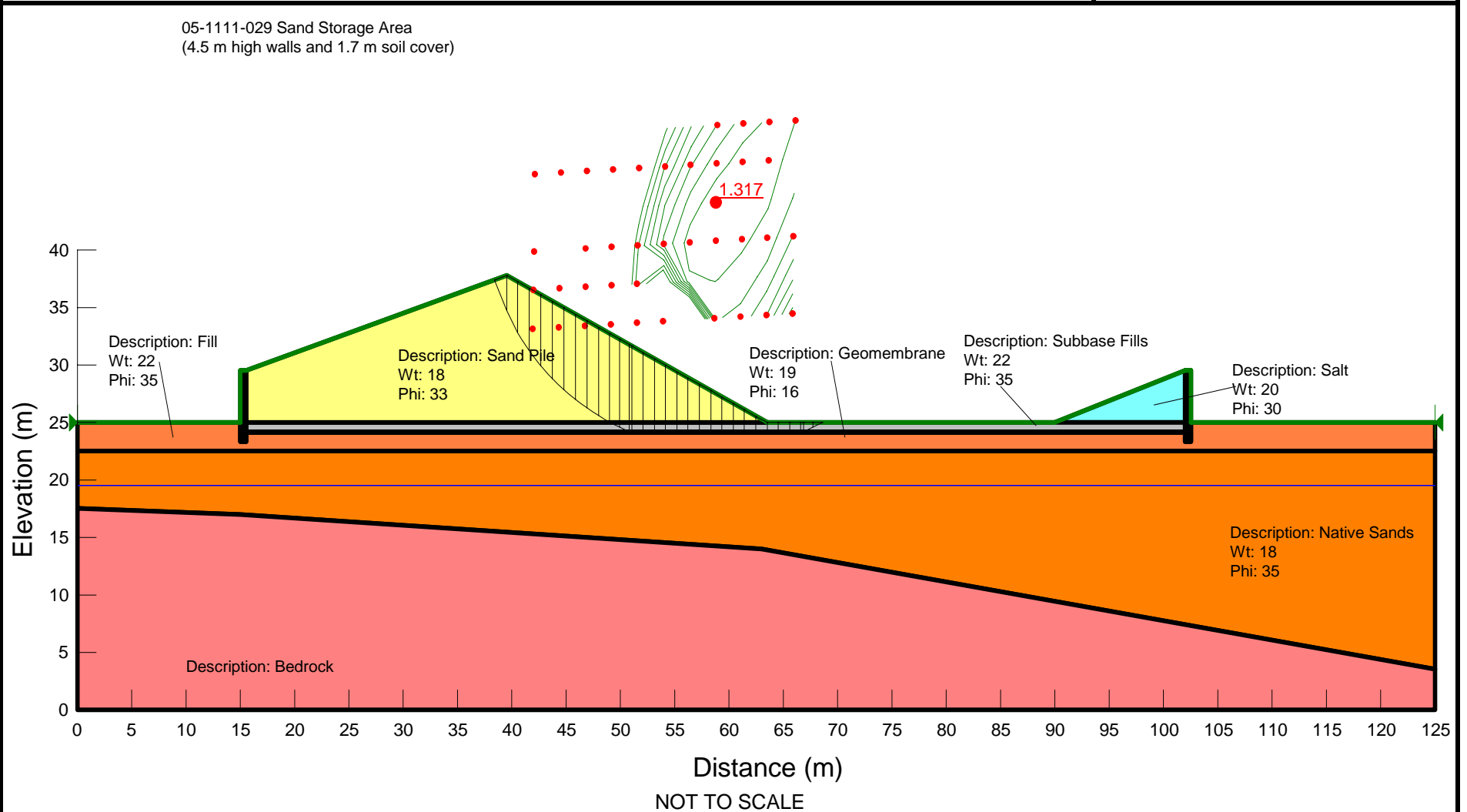
Date: June 2006
Project: 05-1111-029

Golder Associates

Drawn: SLP
Checked: KJB

Gravenhurst Patrol Yard Stability of Sand / Salt Stockpile

Figure 2



Date: May 2006
Project: 05-1111-029

Golder Associates

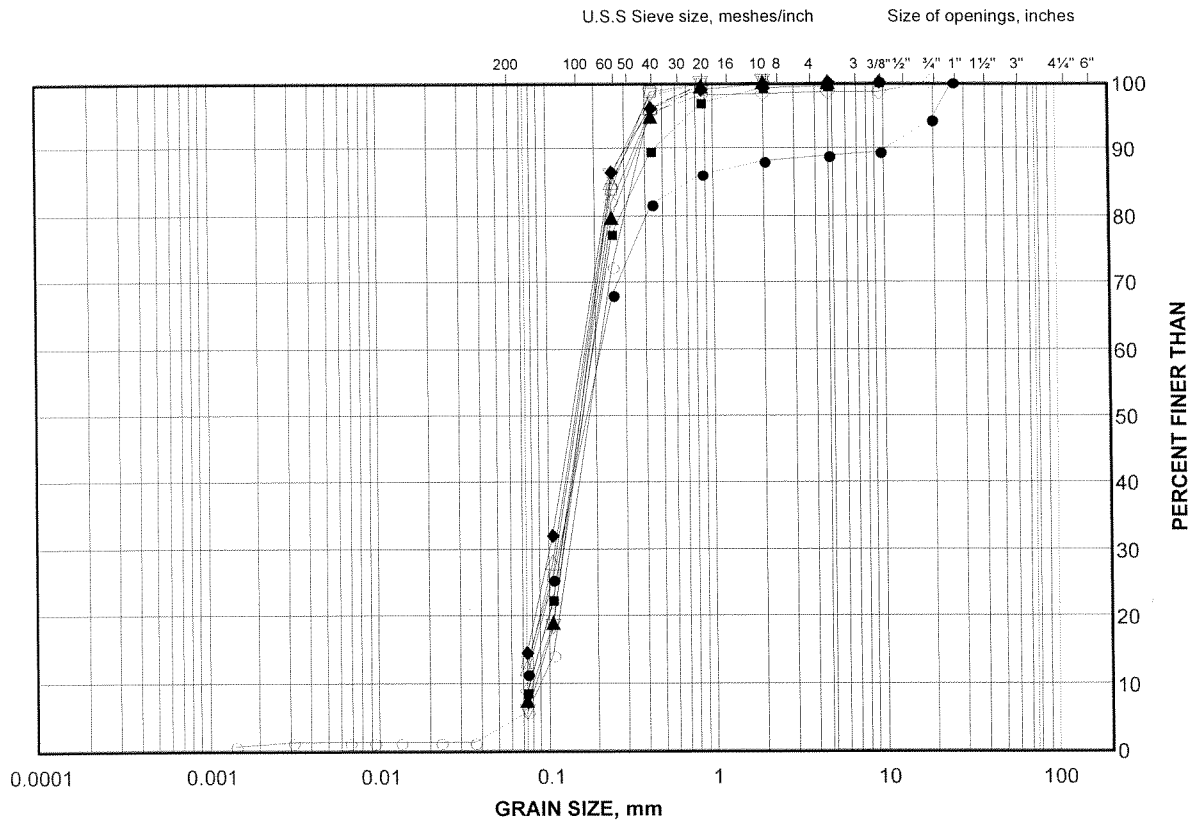
Drawn: KG
Checked: KJB

APPENDIX A
LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand to Silty Sand

FIGURE A1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F-1	4	250.6
■	F-1	9	244.5
◆	F-7	2	252.6
▲	F-8	6	250.2
▽	F-9	5	250.8
○	F-10	7	247.8
□	F-8	4	251.5
△	F-10	2	252.4
▽	F-6	2	253.1

Project Number: 05-1111-029

Checked By: KJB

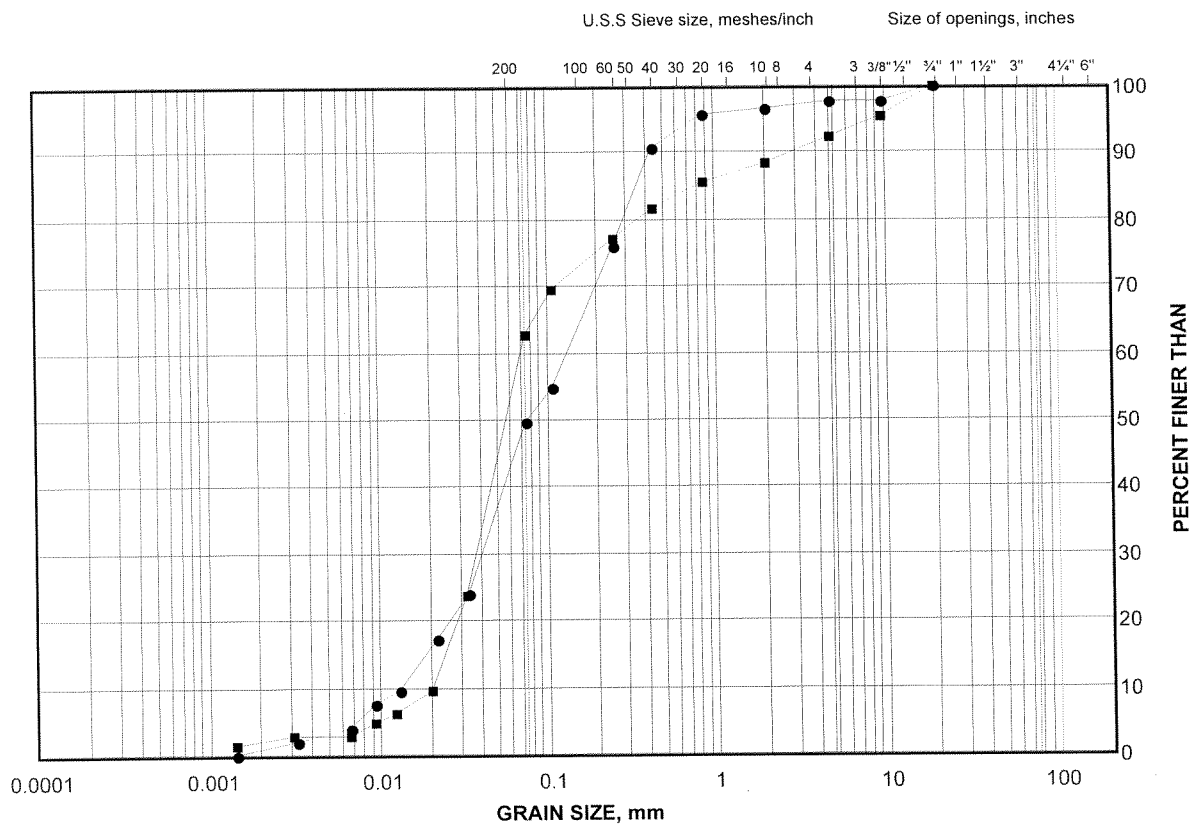
Golder Associates

Date: 31-Mar-06

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Silt to Sandy Silt

FIGURE A2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F-5	1	255.2
■	F-2	4	252.4

Project Number: 05-1111-029

Checked By: KJB

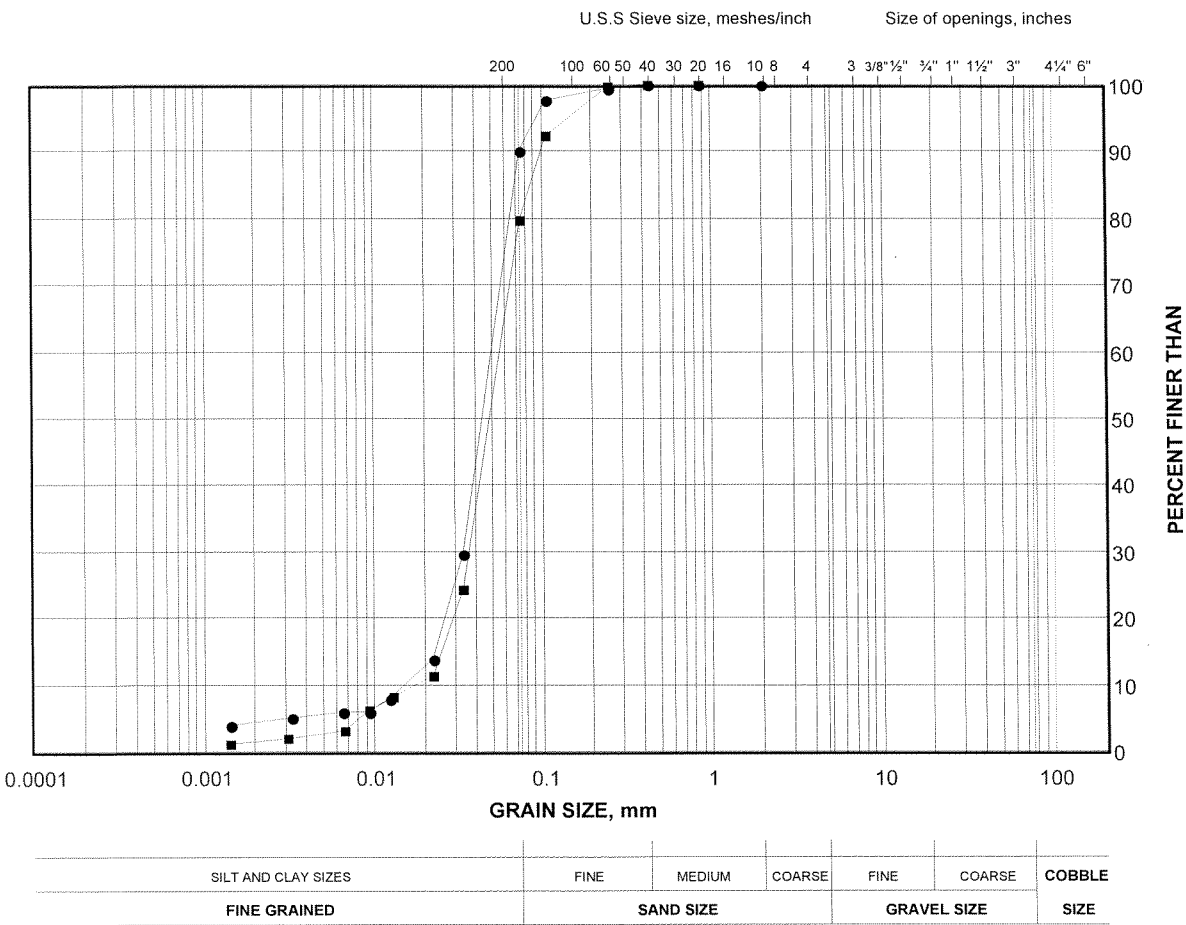
Golder Associates

Date: 31-Mar-06

GRAIN SIZE DISTRIBUTION TEST RESULTS

Silt

FIGURE A3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F-3	3	254.6
■	F-4	2	255.0

Project Number: 05-1111-029

Checked By: KTB

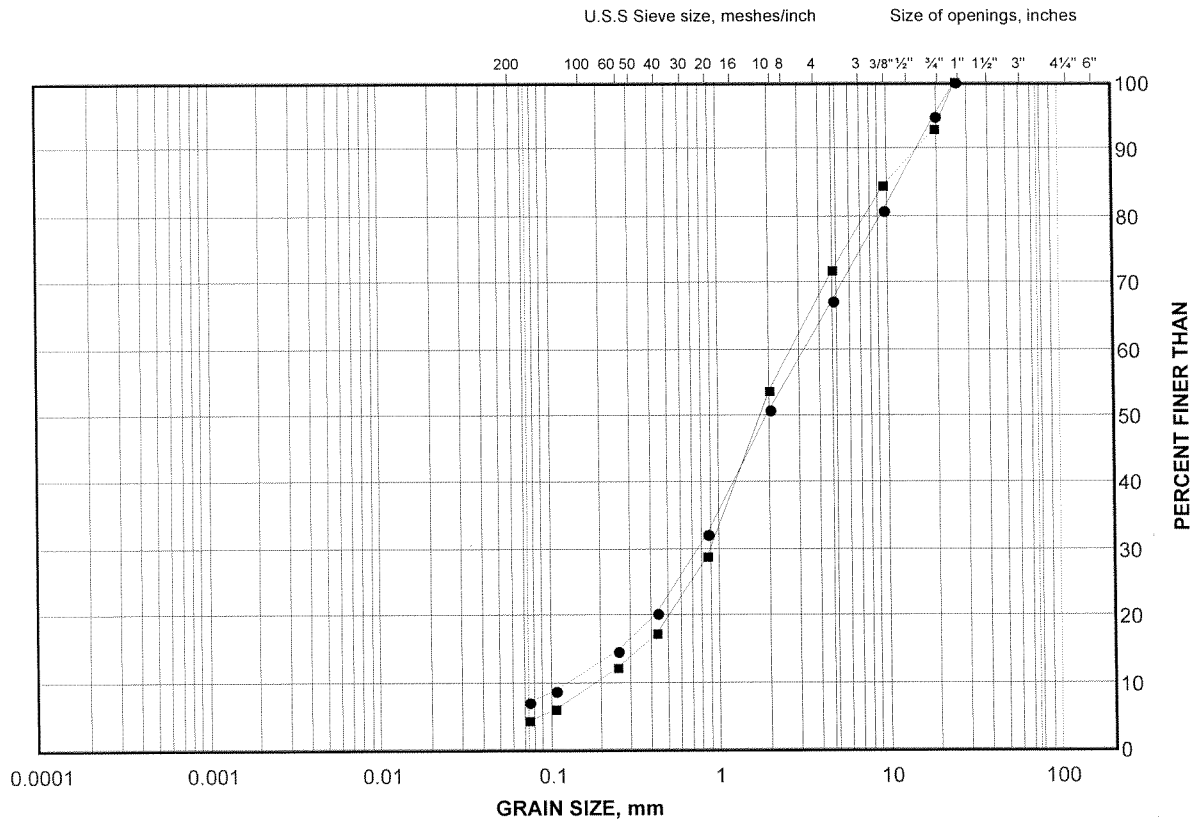
Golder Associates

Date: 31-Mar-06

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel to Gravelly Sand

FIGURE A4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F-3	8	249.5
■	F-9	10	243.8

Project Number: 05-1111-029

Checked By: KJB

Golder Associates

Date: 31-Mar-06

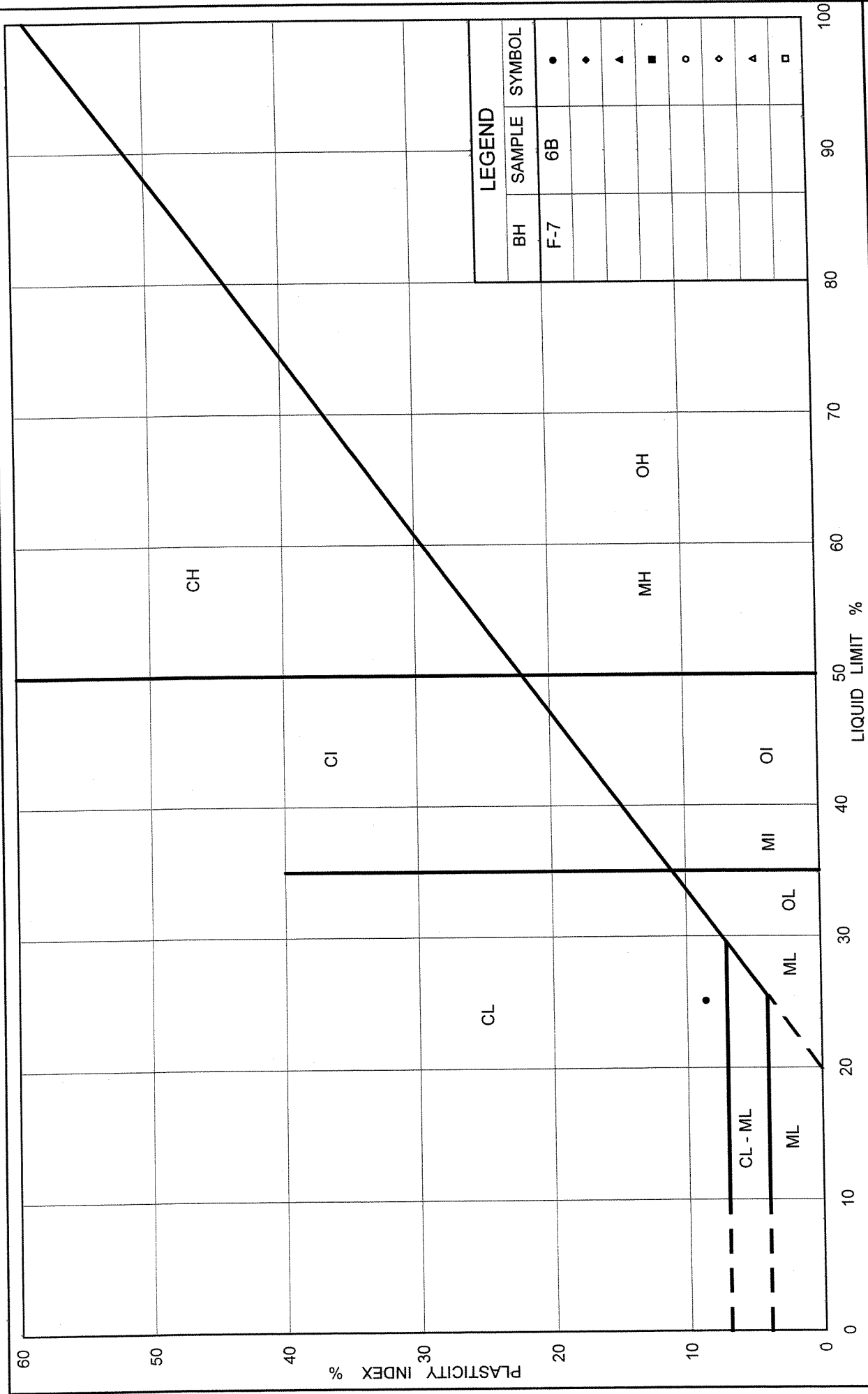


FIG No. A5

PLASTICITY CHART

Clayey Silt

Ministry of Transportation

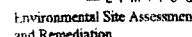
Project No. 05-1111-029



Ontario

APPENDIX B

RECORD OF BOREHOLES – PREVIOUS INVESTIGATION



Sheet: 1 of 1

Originated By: I. Khan

Compiled By: I. Khan

Checked By: D. Stewart

[illegible]

Log of Borehole: BH4 (MW5)

Sheet: 1 of 1

WP: 5420-02-00

Co-ordinates: 4 973 442.9 N, 629 615.7 E

Originated By: I. Khan

Project: Gravenhurst Patrol Yard Class EA

Borehole Type: Hollow Stew Augers

Compiled By: I. Khan

Datum: Geodetic

Date: May 19, 2004

Checked By: D. Stewart

SUBSURFACE PROFILE			SAMPLE				Well Details	Water Levels	Moisture Content	Grain Size Distribution
Elevation/ Depth (m)	Description	Strata Plot	Number	Type	"N" Value	TOV Reading				
						(ppm) 5				
255.1	Ground Surface									
254.9	Topsoil									
0.1	Dark brown, silty sand with organics, detritus and rootlets, very loose, moist		1	SS	1	<2				
	SAND									
	Brown and fine to medium-grained, very loose to compact, moist to moist-wet		2	SS	8	<1				
	-becoming wet									
	-becoming fine-grained		3	SS	19	<1				
			4	SS	17	<1				
251.6						<1				
251.4	Clayey SILT		5	SS	12	<1				
3.6	Brown with grey interspersions, firm, dry									
	Sandy SILT		6	SS	12	<1				
	Brown to greyish brown with fine-grained sand, compact, wet									
250.2						<1				
4.9	SAND		7	SS	50	<1				
	Brown to greyish brown and medium to coarse-grained with some gravel and cobbles, very dense									
			8	SS	50/13cm	<1				
249.4										
5.6	End of Borehole									

Environmental Site Assessment
and Remediation

Log of Borehole: BH4A (MW6)

Sheet: 1 of 1

WP: 5420-02-00

Co-ordinates: 4 973 442.4 N, 629 616.4 E

Originated By: I. Khan

Project: Gravenhurst Patrol Yard Class EA

Borehole Type: Hollow Stem Augers

Compiled By: I. Khan

Datum: Geodetic

Date: May 19, 2004

Checked By: D. Stewart

[illegible]

Log of Borehole: BH5 (MW7)

Sheet: 1 of 2

WP: 5420-02-00

Co-ordinates: 4 973 398.4 N, 629 663.2 E

Originated By: I. Khan

Project: Gravenhurst Patrol Yard Class EA

Borehole Type: Hollow Stem Augers

Compiled By: I. Khan

Datum: Geodetic

Date: May 19, 2004

Checked By: D. Stewart

SUBSURFACE PROFILE			SAMPLE				Well Details	Water Levels	Moisture Content (%) 20 40 60	Grain Size Distribution (%) GR SA SI CL
Elevation/ Depth (m)	Description	Strata Plot	Number	Type	"N" Value	TOV Reading (ppm) 5				
254.4	Ground Surface									
0.0 254.3 0.1	Topsoil Dark brown, silty sand with organics, loose, moist SAND Brown and fine-grained, trace gravel, loose to compact, dry-moist -becoming fine to medium-grained, moist to wet with depth, no gravel		1	SS	4	<1 ▲				
			2	SS	15	<2 ▲				
			3	SS	19	<1 ▲				
			4	SS	25	<2 ▲				
			5	SS	11	<1 ▲				
			6	SS	25	<1 ▲				
			7	SS	22	<1 ▲				
			8	SS	34	<1 ▲				
			9	SS	27	<1 ▲				

Sheet: 1 of 1

Co-ordinates: 4 973 373.9 N, 629 604.6 E

Originated By: I. Khan

Borehole Type: Hollow Stem Augers

Compiled By: I. Khan

Date: May 20, 2004

Checked By: D. Stewart

[illegible]

APPENDIX C

DRAFT OPERATIONAL CONSTRAINTS AND NON-STANDARD SPECIAL PROVISIONS

LEAN CONCRETE (MUD MAT) – Item No.

Special Provision

Scope of Work

The scope of work for the above noted tender item includes supply and installation of the lean concrete (i.e. mud mat) to prevent erosion and/or disturbance to the foundation soils, if required. If the concrete for the footings on the native or engineered fill soil cannot be poured immediately after excavation and inspection, a working mat of lean concrete should be placed in the excavation to protect the integrity of the bearing stratum.

Construction

Lean concrete shall have a compressive strength of at least 5 MPa and will be placed in accordance with OPSS 904. A minimum thickness of 75 mm is recommended.

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.

Dewatering For Foundation and Wash Bay Tank Excavations - Item No.

Non-Standard Special Provision

Scope

The contractor shall be alerted that the soils at the Highway 11 Gravenhurst Patrol Yard site consist of water-bearing sand and silts containing cobbles. Foundation and wash bay tank construction below the groundwater level must be carried out in the dry. The excavation shall be kept stable during the work.

It should be noted that water levels within the area are known to fluctuate by up to about 2 m. As a result, it is recommended that excavation for the foundations and wash bay tank be performed in mid to late summer.

Basis of Payment

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

Sand Fill (Cover Soil) Above Geomembrane – Item No.

Special Provision

Scope of Work

The scope of work for the above noted tender item includes the supply and placement of sand fill above the geomembrane. This specification is also to alert the contractor that specialized equipment or construction techniques are required for placing and compacting all fill materials above the geomembrane.

Materials

Sand fill placed directly above and below the geomembrane will meet OPSS 1004 (Table 9) – Winter Sand or OPSS 1102 (Table 1) – Concrete Sand.

At least two weeks prior to delivery of the cover soil to the site, the Contractor shall submit to the Contract Administrator the results of a grain size distribution analysis (sieve and hydrometer) performed on a representative sample of the cover soil material.

The source and quality of the sand fill material must be approved by the Contract Administrator prior to delivery of the material to site.

Construction

The Contractor must submit a list of equipment and proposed methods for placement of the sand fill (cover soil) above the geomembrane to the Contract Administrator at least two weeks prior to commencement of work. If equipment and/or methods prove unsatisfactory, the contractor will implement changes required to ensure proper completion of work.

The sand fill must be spread with a low ground-pressure bulldozer or equivalent (maximum ground pressure of 35 kPa). A minimum of 300 mm separation distance must be maintained between the bulldozer tracks and the top of the geomembrane. The compacted density of the cover soil shall be great than or equal to 95% of the Standard Proctor Maximum Dry Density.

Truck traffic will be restricted to temporarily thickened areas of the cover soil layer providing a minimum separation distance of 900 mm between the truck tires and the underlying geomembrane.

Vary the bulldozer traffic path and operate equipment with care and under controlled speed, keeping turning radii as large as possible.

Sand Fill (Cover Soil) Above Geomembrane – Item No.

Special Provision

Repair any damage (i.e. tears, punctures) to the geomembrane liner caused during placement of the cover soil layer. The liner repair work shall involve uncovering the damaged areas and patching to a minimum distance of 1 m all around the tear or puncture (refer to the geomembrane specifications for repair requirements).

Construction Quality Assurance

Samples of the sand fill received at the site will be taken for analysis of grain size distribution by the CQA Consultant. The sampling frequency will be one sample per 500 m³ of cover soil received at the site or one sample if the source changes. The results of the analyses shall meet the requirements given in this specification. Material not meeting the project specification must be removed from the site by the Contractor at no cost to the Owner.

The CQA Consultant will inspect the placement of the cover soil, with particular attention given to the thickness of the cover soil layer and the action of the spreading and hauling equipment on the construction surface.

The CQA Consultant will check the thickness of the cover soil layer and carry out testing for percent compaction using a Nuclear Density Gauge at a minimum frequency of 1 test per 500 m².

Basis of Payment

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

DOWELS Into Rock – Item No.

Special Provision

Scope of Work

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

Materials and Installation

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

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

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

Rock Dowel Testing

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

Performance Tests

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

Building Foundation Area	Number of Dowels for Performance Testing
Garage	2
Office	2
Sand / Salt Stockpile	2

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

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

DOWELS Into Rock – Item No.

Special Provision

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.

Sloping Bedrock - Item No.

Non-Standard Special Provision

Scope

The contractor shall be alerted that the bedrock surface at the Highway 11 Gravenhurst Patrol Yard site is variable and steeply sloping. Any foundations designed on bedrock should account for the varying founding elevations, pile lengths, etc.

Basis of Payment

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

Cobbles/Boulders During Excavation, Pile Installation, Etc. - Item No.

Special Provision

The overburden soils at the site consist of water-bearing sand and silt containing gravel, cobbles and boulders. In addition, the soils will be susceptible to cave-in, sloughing and boiling.

Appropriate equipment and procedures will be required to penetrate/remove cobbles/boulders that are encountered during excavation and/or pile installation.

Basis of Payment

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

END OF SECTION