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REPORT ON

DETAILED FOUNDATION INVESTIGATION AND DESIGN DETOUR #1, DETOUR #2 AND WIDENING OF MUNICIPAL ROAD 80 FROM HARRISON DRIVE TO HIGHWAY 17 INTERCHANGE G.W.P 99-98-00 MINISTRY OF TRANSPORTATION, ONTARIO DISTRICT 54, SUDBURY

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DETAILED
FOUNDATION INVESTIGATION AND DESIGN
DETOUR #1, DETOUR #2
AND WIDENING OF MUNICIPAL ROAD 80
FROM HARRISON DRIVE TO HIGHWAY 17 INTERCHANGE

G.W.P 99-98-00

MINISTRY OF TRANSPORTATION, ONTARIO

DISTRICT 54, SUDBURY

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Earth Tech (Canada) Inc. (EarthTech) to carry out a detailed foundation investigation for the proposed detour alignments and widening of Municipal Road 80 as part of the final design for the new interchange at Highway 17 and Sudbury Municipal Road 80 (Long Lake Road) being carried out for the Ministry of Transportation, Ontario (MTO). The proposed works consist of two detour alignments to maintain the traffic flow during the widening of Municipal Road 80 and during the construction of the overpass structure and the new N/S-E and W-N ramps that will cross the existing Highway 17. In addition, it has been proposed to widen Municipal Road 80 to four lanes from north of Harrison Drive southerly to the new Highway 17/MR 80 interchange. The general location of the Highway 17 and Municipal Road 80 (Long Lake Road) interchange is shown in plan on Figure 1.

The terms of reference for the scope of work are outlined in the following Golder letter proposals:

- "Additional Foundation Investigation, Widening of Regional Road 80, North/South of Harrison Drive" dated February 24, 2003

- "Additional Foundation Investigation, Widening of East Side of Municipal Road 80, From Harrison Drive to New Interchange" dated June 3, 2003

- "Additional Foundation Investigation, Detours #1 and #2 and Widening of West Side of Municipal Road 80 From Harrison Drive to New Interchange" dated June 26, 2003

The additional investigation is part of the Consultant's Agreement (Number P.O.5005 – A 000218) for this project. The work was carried out in a manner consistent with the Quality Control Plan for this project dated August 21, 2002. The plans and profiles detailing the proposed new detours and interchange alignments at Highway 17 and Municipal Road 80 were provided to Golder by EarthTech.

This report addresses the high fill embankment and swamp crossing areas along the proposed MR 80 widening, Detour #1 and Detour #2 alignments that have total lengths of about 800 m, 320 m and 320 m, respectively.

The foundation investigation and design recommendations for the proposed overpass structure and for the high embankment fills, deep cuts and swamp crossings for the new interchange roadway and ramp alignments have been provided in two previous foundation reports dated June 2003 and May 2003.

The purpose of this investigation is to establish the subsurface conditions at the areas of the proposed high embankments and swamp crossings associated with the widening of existing Municipal Road 80 and the new Detour #1 and Detour #2 roadway construction. The locations of the investigated sites within the project limits are shown in plan on Drawing 1.

2.0 SITE DESCRIPTION

The site is located near the intersection of the existing Highway 17 and Sudbury Municipal Road 80 (Long Lake Road) in Sudbury, Ontario. Highway 17 in this area of Northern Ontario is presently a two lane, rural King's highway with an at-grade, signalled intersection with Sudbury Municipal Road 80. Municipal Road 80 is currently a two lane secondary highway.

The site generally consists of rolling terrain including open fields, bush areas, swamp areas, and numerous rock outcrops at the ground surface. The ground surface within the limits of the project areas general lies between Elevation 269 m and 284 m.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for this investigation was carried out between March 17 and August 29, 2003 during which time a total of seventeen (17) sampled boreholes, eight (8) Dynamic Cones Penetration Tests (DCPTs), eight (8) Test Pits, and four (4) Probeholes were put down. The following table summarizes the locations of the areas investigated within the project limits as part of the foundation investigation.

Location	*Proposed Works*	*Chainage*	*Swamp Area*
East Side of MR 80 (widening north bound lanes and turning lanes at Harrison / Countryside Drive Intersection)	Swamp Crossing (fill up to 2 m)	9+000 to 9+340	A10
	Swamp Crossing (fill up to 4 m)	9+340 to 9+470	A12E
West Side of MR 80 and Detour #1 (widening south bound lanes)	Swamp Crossing (fill up to 4 m)	9+290 to 9+510	A12W
	Swamp Crossing (fill up to 2.5 m)	9+850 to 9+950	A2
Detour #2 (South Side of Hwy 17 / N/S-E Ramp)	High Fill (fill up to 9 m)	16+180 to 16+230	
	High Fill (fill up to 7 m)	16+230 to 16+430	
	Swamp Crossing (fill up to 8 m)	16+430 to 16+500	A13

The boreholes were advanced along the centreline (or within the footprint) of the proposed detour / highway widening roadways in the high fill and swamp areas. In the swamp areas, a combination of boreholes, DCPTs, and test pits were performed within the footprint of the proposed embankments. In areas where bedrock was expected to exist at shallow depths, probeholes were put down to confirm the bedrock surface. The location of the site area is shown on Figure 1. The locations of the investigated areas at the Highway 17/Municipal Road 80 interchange are shown on Drawing 1. Borehole Location and Soil Strata drawings for the individual areas investigated are shown on Drawings 2 to 7.

The borehole / DCPT field investigation was carried out using two, track-mounted CME 55 drill rigs supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario and Colbar Resources of Sudbury, Ontario. The boreholes put down with the drill rig were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers. Soil samples were obtained, where possible, continuously or at intervals ranging from 0.75 m to 1.5 m in depth, using a

50 mm O.D. split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Where access by conventional drill rig equipment was not feasible, shallow test pits were excavated using a CAT E120B backhoe supplied and operated by Interpaving Ltd. of Sudbury, Ontario. In areas where bedrock was assessed to exist at a shallow depth, probeholes were advanced using a 25 mm diameter steel rod advanced manually into the ground until effective refusal. Probeholes were advanced by members of our technical staff.

All boreholes were advanced to a depth ranging from 0.7 m to 6.7 m (generally to a depth equal to the height of new embankment fill, or to refusal to further penetration). The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in selected boreholes to permit monitoring of the groundwater level at these locations. The piezometers consist of a 25 mm outside diameter rigid PVC tubing with a 0.3 m long slotted tip that is installed at a selected depth within the boreholes. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report.

The field work was supervised throughout by members of our 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 samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and appropriate laboratory testing. Laboratory tests, including classification testing such as water content, grain size distribution, and Atterberg Limiting, were carried out to MTO and/or ASTM Standards as appropriate.

On completion of the fieldwork, all investigated boreholes (i.e. including DCPT's, test pit, and probehole locations) were located in relation to either the MR 80 centreline or the staked Detour #2 centreline. The chainage and offset locations for each borehole were sent to EarthTech, where they were converted into the NAD 83 MTM (Zone 12) co-ordinate system. EarthTech then provided us with northing, easting, and elevation (geodetic datum) for each as-drilled borehole location.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geology

From published geologic information, the site is located in the physiographic region known as the Laurentian Highlands that form the southernmost part of the Canadian Shield (Geology of Ontario; OGS Special Volume 4). The Laurentian Highlands comprise a southeast-trending and slightly elevated region that is underlain by Precambrian bedrock. To the south, the highlands gradually disappear beneath the cover sequences of Paleozoic strata and the southern Borderlands. These Precambrian rocks were eroded to a gently undulating land surface, before the deposition of Paleozoic strata and later erosion during glaciation left behind only scarred Precambrian rocks covered in a few places by flat-lying Paleozoic strata. The local physiography is generally characterized by variable overburden materials and an irregular, variable bedrock surface with rock outcrops.

4.2 Subsurface Conditions and General Overview

The detailed subsurface soil and groundwater conditions as encountered in the boreholes and test pits completed during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole and Field Test Pit sheets following the text of this report. The thickness of the overburden in the swamp areas as inferred from the resistance to Dynamic Cone Penetration Test (DCPT) is shown on the Record of Penetration Test sheets following the text of this report. The depth to bedrock as inferred from the resistance to further penetration is shown on the Field Probehole Log sheets following the text of this report. The results from the laboratory testing are provided in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets and Field Test Pit Logs are inferred from non-continuous sampling, observations of drilling and excavation 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 and test pit locations.

The soil stratigraphy as encountered in the boreholes and test pits along the proposed MR 80 widening, Detour #1, and Detour #2 areas are shown on Drawings 2 to 7 inclusive.

In general, the stratigraphy encountered at the areas investigated is similar; however, the overburden thickness is variable ranging from no cover (i.e. quartzite bedrock outcrops present at ground surface) to about 6.7 m deep. At locations beyond the existing highway embankments, the stratigraphy generally consists of near surface layers of topsoil and/or fibrous peat overlying deposits of cohesive varved silty clay and clayey silt and non-cohesive layers such as silt, silty

sand to sandy silt, and sand. Embankment fill is present near the ground surface in selected areas within the project limits.

The natural water content determinations and Atterberg limits testing carried out on selected samples obtained in the boreholes and test pits are shown on the attached Record of Boreholes sheets and Field Test Pit Logs.

Detailed descriptions of the subsurface conditions at each site are provided in the following sections.

4.3 East Side of MR 80 (widening north bound lanes and turning lanes at Harrison / Countryside Drive Intersection)

The proposed widening of the East side of Municipal Road 80 (north bound lanes) crosses over Swamp A10, Swamp A12E, and bedrock outcrops as identified on Drawings 1, 2 and 3. Swamp A10 exists both to the north and south of the proposed Countryside drive intersection along MR 80. South of Swamp A10, the east side of MR 80 consists of shallow overburden in Swamp A12E and bedrock outcrops.

4.3.1 Swamp A10 (Sta. 9+000 to 9+340)

Plan and profile drawings along the new widening showing the borehole and test pit locations and interpreted stratigraphy between Stations 9+000 and 9+340 are shown on Drawings 2 and 3. A total of four boreholes (Boreholes 03-151 to 03-154 inclusive) and eight test pits (TP03-01 to TP03-08) were completed within the swamp area to investigate the subsurface conditions along the proposed new MR 80 widening alignment between Stations 9+000 and 9+340. All boreholes were terminated upon refusal to further auger advancement; whereas some test pits did not encounter bedrock upon completion of the test pit excavation.

In all boreholes effective refusal (inferred bedrock) was reached between Elevations 264.2 m and 267.0 m. Effective refusal (i.e. unable to advance backhoe bucket below inferred bedrock) was encountered in two test pit locations (TP03-04 and TP03-05) at Elevation 265.8 m and 267.8 m, respectively. Effective refusal was reached along the west side of Test Pit TP03-08 at a depth of about 0.5 m below ground surface (Elevation 269.1 m); however the east half of the test pit did not reach refusal at a depth of 2.9 m (Elevation 266.7 m), at which depth the test pit was terminated.

In general, the subsurface soils along the proposed new MR 80 widening alignment in this area consist of an organic layer of topsoil or fibrous peat underlain by deposits of silt / clayey silt over

varved silty clay to clayey silt over bedrock. Boreholes 03-151 and 03-152 and Test Pits 03-06 and 03-07 were advanced from the ice surface (approximately Elevation 269 m). The thickness of the ice was measured to range between 100 mm and 300 mm.

4.3.1.1 Topsoil

At the ground surface in Boreholes 03-154 (Elevation 269.3 m) and Test Pits 03-01, 03-04, 03-07 and 03-08 (Elevations 269.4 m, 269.0 m, 269.2 m and 269.6 m, respectively), a 100 mm to 300 mm thick layer of dark brown topsoil was encountered. At some test pit locations, the topsoil was mixed with variable amounts of fill near the toe of the existing MR 80 embankment slope (TP03-05 and TP03-07). The topsoil in test pit 03-07 was encountered below the ice surface.

4.3.1.2 Fibrous Peat

At the ground surface in Boreholes 03-151, 03-152, and 03-153 (Elevations 268.8 m, 268.7 m, and 269.2 m) and Test Pits 03-02, 03-03, 03-06 (Elevations 268.7 m, 268.7 m, and 269.1 m), an approximate 300 mm to 900 mm thick layer of brown to black fibrous peat was encountered. The fibrous peat was located beneath the ice surface in Boreholes 03-151 and 03-152, and in Test Pit 03-06. A 300 mm thick layer of fibrous peat was also encountered beneath the surficial embankment fill encountered in Test Pit 03-05.

4.3.1.3 Silt / Clayey Silt

Immediately beneath the topsoil / fibrous peat / embankment fill in all of the boreholes and test pit locations completed in this area, a layer of brown to grey silt, some clay, trace sand and gravel to clayey silt was encountered. The elevation of the top of this layer varied between 268.1 m and 269.4 m and the thickness ranges from 0.7 m to 2.8 m. In Borehole 03-153 and Test Pit 03-08, the silt deposit is about 2.4 m and 2.8 m thick, respectively, however it contains an interlayer of varved silty clay.

Standard Penetration Testing (SPT) carried out within the silt stratum measured 'N' values ranging from 4 blows to 8 blows per 0.3 m of penetration indicating a very loose to loose state of packing. Refusal to further split spoon sampling and augering was encountered in this stratum on probable bedrock at Elevation 267.0 m in Borehole 03-154. Refusal to further excavation was encountered in this stratum on probable bedrock at Elevation 267.8 m in Test Pit 03-05. Refusal to further backhoe bucket penetration was encountered on probable bedrock at Elevation 269.1 m on the west side of Test Pit 03-08. The east side of test pit 03-08 was terminated at a depth of 2.9 m (Elevation 266.7 m) within the silt deposit.

A grain size distribution for a sample of the clayey silt deposit is shown on Figure A5 of Appendix A (TP03-06, Sample 1). The natural water content measured on selected samples of this silty deposit ranged between 23 percent and 40 percent with an average of about 35 percent.

4.3.1.4 Varved Silty Clay to Clay / Clayey Silt

Varved silty clay to clay and varved clayey silt deposits with trace sand were encountered below or within the silt / clayey silt layers in Boreholes 03-151 to 03-153 and in Test Pits 03-01 to 03-04 and 03-06 to 03-08. This deposit typically contained sand seams and variable amounts of cobbles / boulders were encountered in Test Pit 03-06. The elevation of the top of this layer varied from 266.9 m to 268.6 m and the layer ranged in thickness from 0.3 m to 3.4 m. The varved silty clay to clay / clayey silt stratum extends to Elevation 264.2 m and 265.4 m in Boreholes 03-151 and 03-152; and extends to Elevation 265.8 m in Test Pit 03-04, where auger refusal or backhoe refusal on probable bedrock was reached.

Standard Penetration Testing (SPT) measured 'N' values typically ranged between 3 blows and 7 blows per 0.3 m of penetration, indicating a soft to firm consistency within this deposit. In situ vane testing carried out within the deposit measured undrained shear strengths of 29 kPa and 45 kPa; one test gave a value of greater than 100 kPa (i.e. the upper measurable limit of the vane). These values indicate the deposit typically has a firm to stiff consistency. Pocket penetrometer testing carried out on samples of the deposit typically ranged from less than 12 kPa to 86 kPa; two tests gave a value of greater than 100 kPa. The results of the pocket penetrometer testing indicates the deposit has a soft to stiff consistency, which is generally consistent with the in situ vane test results. The remoulded in situ vane values indicates that the sensitivity of the deposit is about 6.9 and 2.6, implying the deposit is medium sensitive to sensitive based on the classification system provided in the CFEM (1992).

Laboratory Atterberg Limits testing was carried out on four samples of the varved silty clay to clay / clayey silt deposit. The test results are summarized in the following table.

Borehole / Test Pit	*Sample*	*Elevation (m)*	*Liquid Limit (%)*	*Plastic Limit (%)*	*Plasticity Index (%)*
TP03-02	2	266.3 – 267.2	52	18	34
TP03-03	2	266.2 – 267.0	64	25	39
TP03-06	3	265.6 – 267.2	32	21	11
TP03-07	4	265.7 – 266.2	56	23	33
TP03-08	1	268.1 – 268.6	47	23	24
Average	-	-	50	22	28

The results of the Atterberg limits testing is shown on the plasticity chart on Figure A6 in Appendix A. A grain size distribution for one sample of the varved clay from this deposit is shown on Figure A4 of Appendix A. A grain size distribution for one sample of the varved clayey silt from this deposit is shown on Figure A5 of Appendix A (TP03-06, Sample 3). The natural water content measured on selected samples of this cohesive deposit ranged between 30 percent and 54 percent, with an average of 42 percent.

4.3.1.5 Sandy Silt (Till)

About 150 mm of sandy silt till with some gravel was encountered immediately beneath the silt deposit in Borehole 03-153. The top of this layer was encountered at Elevation 265.9 m.

Refusal to further split spoon sampling and augering was encountered on probable bedrock at Elevation 265.7 m.

4.3.1.6 Groundwater Conditions

In general, the samples taken in the boreholes were noted to be moist to wet. Details of the groundwater conditions and water levels observed in the open boreholes and test pits at the time of drilling and excavation are summarized on the Record of Borehole sheets and Field Test Pit Logs following the text of this report. A standpipe piezometer was installed in BH 03-151 to allow groundwater level monitoring within the Swamp A10 area. The water level in the piezometer was measured at Elevation 265.8 m (about 3.0 m below the ground surface), upon completion of installation on March 21, 2003; however, there was approximately 100 mm of ice present at the surface of the borehole (Elevation 268.8 m). It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

4.3.2 Swamp A12E (Sta. 9+340 to 9+470)

Plan and profile drawings along the proposed MR 80 widening showing the borehole and DCPT locations and interpreted stratigraphy between Stations 9+340 and 9+470 are shown on Drawing 3. A total of two boreholes (Boreholes 03-184 and 03-185) and two DCPT's (DC03-60 and DC03-61) were completed within the marshy area to investigate the subsurface conditions along the east side of the proposed new MR 80 widening alignment between Stations 9+340 and 9+470. All boreholes and DCPT's were terminated upon refusal to further auger or dynamic cone advancement.

In all boreholes and DCPT's, effective refusal was reached between Elevations 267.5 m and 269.3 m.

In general, the subsurface soils along the proposed new MR 80 widening alignment in this area consist of an organic layer of fibrous peat underlain by deposits of silt / clayey silt over varved silty clay / clayey silt over bedrock.

4.3.2.1 Bedrock Outcrop

Bedrock outcrops were visually identified in this area from approximate Station 9+330 to Station 9+370, offset approximately 12 m east of the MR 80 centreline (see Drawing 3).

4.3.2.2 Fibrous Peat

At the ground surface in Boreholes 03-184 and 03-185 (Elevations 269.9 m and 270.0 m), an approximate 0.5 m and 0.3 m thick layer of brown to black fibrous peat was encountered.

The natural water content measured on a selected sample of the fibrous peat layer was 44 percent.

4.3.2.3 Silt / Clayey Silt

Immediately beneath the fibrous peat at most borehole locations completed in this area, a layer of brown to grey silt, some clay, trace sand and organics was encountered. A layer of clayey silt with trace to some organics, trace cobbles, trace rock fill was encountered below the fibrous peat in Borehole 03-185. The elevation of the top of this silt / clayey silt layer varied between 269.4 m and 269.7 m and the thickness ranges from 0.4 m to 0.6 m.

Standard Penetration Testing (SPT) carried out within the clayey silt and silt stratum measured 'N' values of 4 blows and 19 blows per 0.3 m of penetration indicating a soft to firm consistency

for the clayey silt and compact state of packing for the silt. Refusal to further split spoon sampling and augering was encountered on probable bedrock at Elevation 269.3 m in Borehole 03-185.

The natural water content measured on selected samples of this silty deposit ranged between 22 percent and 31 percent with an average of about 27 percent.

4.3.2.4 Varved Clayey Silt

A varved clayey silt deposit with trace sand was encountered below the silt layer in Borehole 03-184. The elevation of the top of this deposit was at 268.6 m and it was about 1.1 m thick. The stratum extends to Elevation 267.5 m in Boreholes 03-184 where auger refusal on probable bedrock was reached.

A Standard Penetration Testing (SPT) measured 'N' value was 18 blows per 0.3 m of penetration, indicating a stiff consistency within this deposit.

Laboratory Atterberg Limits testing was carried out on one sample of the varved silty clay / clayey silt deposit. The test results are summarized in the following table.

Borehole / Test Pit	*Sample*	*Elevation (m)*	*Liquid Limit (%)*	*Plastic Limit (%)*	*Plasticity Index (%)*
BH03-184	3	267.8 – 268.4	28	18	10

The results of the Atterberg limits testing is shown on the plasticity chart on Figure A6 in Appendix A. The natural water content measured on a selected sample of this cohesive deposit was 24 percent.

4.3.2.5 Groundwater Conditions

In general, the samples taken in the boreholes were noted to be wet. Details of the groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. Based on observations made in the open boreholes, the groundwater level is inferred to be at the existing ground surface. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

4.4 West Side of MR 80 and Detour #1 (widening south bound lanes)

The proposed widening of the west side of MR 80 and Detour #1 alignment partially crosses over Swamp A12W and Swamp A2. Between the two swampy areas, the alignment in this area crosses numerous interspersed, low lying bedrock outcrops and shallow overburden.

4.4.1 Swamp A12W (Sta 9+290 to 9+510)

Plan and profile drawings along the proposed MR 80 widening and Detour #1 alignment showing the borehole locations and interpreted stratigraphy between Stations 9+290 and 9+510 are shown on Drawing 4. A total of five boreholes (Boreholes 03-179 to 03-183 inclusive) and four Dynamic Cone Penetration Tests (DCPT 03-56 and 03-59) were put down within the swamp area to investigate the subsurface conditions along the proposed new alignment. All boreholes and DCPTs were terminated upon refusal to further auger or cone advancement.

The boreholes and DCPTs in this section were advanced to depths ranging between 0.9 m and 8.0 m below ground surface (Elevation ranging between 261.3 m and 268.6 m), all of which were terminated on inferred bedrock. The presence of bedrock was inferred from either refusal to further drilling, refusal of sampling spoon advance during SPT testing, and/or refusal to dynamic cone advance in the DCPT holes.

In general, the subsurface soils in this area consist of an organic layer of fibrous peat underlain by deposits of silt and sand, underlain by varved silty clay to clayey silt, underlain by silt and sand deposits over bedrock. Boreholes 03-179, 03-180, 03-181, and 03-182 were drilled with up to 0.3 m of standing water at the borehole locations (approximate standing water Elevation 269.7 m).

4.4.1.1 Fibrous Peat

Below the standing water surface in Boreholes 03-179 to 03-182 (inclusive) and surficially in Borehole 03-183, a brown to black fibrous peat layer was encountered. The elevation of the top of this layer varied from 269.0 m to 269.8 m and the layer ranged in thickness from 150 mm to 600 mm.

The natural water content measured on selected samples of this deposit ranged between 25 percent and 59 percent.

4.4.1.2 Silt

Immediately below the fibrous peat layer in Boreholes 03-179 and 03-181, a deposit of mottled greyish brown silt containing some clay, trace to some sand, trace gravel was encountered. The elevation of the top of this layer was 268.8 m and 268.9 m and the layer was about 0.3 m thick for Boreholes 03-181 and 03-179, respectively. A silt stratum containing trace to some sand and frequent sand seams was also encountered at depth in Borehole 03-182. The top of this layer was at Elevation 268.3 m and it was approximately 0.7 m thick.

Standard Penetration Testing (SPT) measured 'N' values ranging from 0 blows (weight of sampling hammer) to 9 blows per 0.3 m of penetration, indicating a very loose to loose state of packing for this deposit.

The natural water content measured on selected samples of this deposit ranged between 26 percent and 30 percent.

4.4.1.3 Silty Sand / Sand

A silty sand to sand layer containing trace clay and gravel was encountered in Boreholes 03-182 and 03-183, immediately below the fibrous peat layer. The top of this layer was encountered at Elevation 269.2 m and 268.3 m and was 0.9 m and 0.6 m thick for Boreholes 03-182 and 03-183, respectively.

Standard Penetration Testing (SPT) carried out within this stratum measured an 'N' value of 25 blows per 0.3 m of penetration indicating a compact state of packing.

A grain size distribution for one sample of the silty sand deposit is shown on Figure A3 of Appendix A. The natural water content measured on selected samples of this sandy deposit ranged between 23 percent and 27 percent.

4.4.1.4 Clayey Silt

Immediately beneath the silt layer in Borehole 03-182, a layer of grey clayey silt with trace to some sand and sand seams was encountered. The elevation of the top of this layer was 267.6 m and it was 0.8 m thick.

Standard Penetration Testing (SPT) carried out within the clayey silt stratum measured an 'N' value of 7 blows per 0.3 m of penetration indicating a firm consistency.

The natural water content measured on a selected sample of this deposit was 41 percent.

4.4.1.5 Varved Silty Clay to Clay

Varved silty clay to clay deposits were encountered below the fibrous peat, silt, clayey silt, silty sand and sand layers in all of the boreholes put down in this area (Boreholes 03-179 to 03-183, inclusive). This deposit typically contained sand seams and trace organics, and cobbles were encountered in Borehole 03-181. The elevation of the top of this layer varied from 266.8 m to 269.1 m and the layer ranged in thickness from 1.0 m to 3.4 m. The varved silty clay to clay stratum extends to Elevation 268.2 m in Borehole 03-180 where auger refusal on probable bedrock was reached.

Standard Penetration Testing (SPT) measured 'N' values typically ranged between 0 blows (weight of sampling hammer) and 9 blows per 0.3 m of penetration (with one value measuring 24), indicating a predominantly very soft to stiff consistency within this deposit.

The results of field vane testing carried out within the deposit are summarized in the following table.

Borehole	*Elevation/Depth (m)*	*Undrained Shear strength (kPa)*	*Remoulded Shear Strength (kPa)*	*Sensitivity*
BH03-179	267.5 / 1.7	51.7	2.9	18.0
BH03-179	267.2 / 2.0	> 98	11.5	8.5
BH03-181	267.3 / 2.4	23.0	2.9	8.0
BH03-181	267.0 / 2.7	57.5	8.6	6.7
BH03-181	265.7 / 4.0	11.5	8.6	1.3
BH03-181	265.4 / 4.3	25.9	8.6	3.0
BH03-182	266.5 / 3.2	37.3	5.7	6.5
BH03-182	266.2 / 3.5	63.2	10.1	6.3
BH03-182	265.0 / 4.7	28.7	3.6	8.0
Average	-	44.0	6.9	7.4

In situ vane testing carried out within the deposit measured undrained shear strengths ranging between 12 kPa and 63 kPa; one test gave a value of greater than 98 kPa (i.e. the upper measurable limit of the vane). These values indicated the deposit typically has a very soft to stiff consistency, which is consistent with the SPT test results. The remoulded in situ vane values indicates that the sensitivity of the deposit ranged between 1.3 and 18.0, implying the deposit

ranges from low to extra sensitive, with one value indicating quick clay based on the classification system provided in the CFEM (1992).

Laboratory Atterberg Limits testing was carried out on three samples of the varved silty clay to clay deposit. The test results are summarized in the following table.

Borehole	*Sample*	*Elevation (m)*	*Liquid Limit (%)*	*Plastic Limit (%)*	*Plasticity Index (%)*
BH03-179	2	267.8 – 268.4	44	22	22
BH03-181	3	267.6 – 268.2	57	22	35
BH03-181	4	266.0 – 266.7	57	21	36
Average	-	-	53	22	31

The results of the Atterberg limits testing are shown on the plasticity chart on Figure A6 in Appendix A. The natural water content measured on selected samples of this varved silty clay to clay deposit ranged between 32 percent and 56 percent, with an average of 46 percent.

4.4.1.6 Silty Sand / Silt / Sand / Sandy Silt

Underlying the varved silty clay to clay, deposits of silty sand, silt, sandy silt and sand were encountered in Boreholes 03-179, 03-181, 03-182, and 03-183. The deposits contained variable amounts of clay, trace gravel and organics. The elevation of the top of this layer varied from 264.7 m to 266.8 m and the layer ranged in thickness from 0.1 m to 0.9 m.

Standard Penetration Testing (SPT) carried out within the silty sand, silt, sand, and sandy silt stratum measured 'N' values of 0 blows (i.e. weight of sampling hammer) and 11 blows per 0.3 m of penetration indicating a very loose to compact state of packing. Refusal to further split spoon sampling and augering was encountered below this stratum on probable bedrock at elevations ranging between 263.9 m and 266.7 m in Boreholes 03-179, 03-181, 03-182, and 03-183.

The natural water content measured on selected samples of this sandy and silty deposit ranged between 22 percent and 30 percent.

4.4.1.7 Groundwater Conditions

In general, the samples taken in the boreholes were noted to be wet. Details of the groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. Based on observations made in

the open boreholes, the groundwater level is inferred to be at the existing ground surface. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

4.4.2 Swamp A2 (Sta. 9+850 to 9+950)

A plan and profile drawing along the proposed Detour #1 alignment showing the borehole locations and interpreted stratigraphy in Swamp A2 between Stations 9+850 and 9+950 are shown on Drawing 5. A total of two boreholes (Boreholes 03-173 and 03-174) and two Dynamic Cone Penetration Tests (DCPT 03-54 and 03-55) were put down within the swamp area during the current investigation to investigate the subsurface conditions along the proposed new alignment.

The boreholes and DCPTs in this section (BH 03-173, BH 03-174, DCPT 03-54, and DCPT 03-55) were advanced to depths ranging between 2.4 m and 4.6 m below ground surface (Elevation ranging between 274.3 m and 276.7 m), all of which were terminated on inferred bedrock. The presence of bedrock was inferred from either refusal to further drilling, refusal of sampling spoon advance during SPT testing, and/or refusal of dynamic cone advance in the DCPT holes.

In general, the subsurface soils along the proposed new Detour #1 alignment in this area consist of an organic layer of fibrous peat underlain by a layer of clayey silt, underlain by deposits of silt and silty sand to sand over inferred bedrock.

4.4.2.1 Fibrous Peat

A surficial layer of dark brown to black fibrous peat was encountered in Boreholes 03-173 and 03-174 put down in this area. The elevation of the top of this layer was 279.1 m and 278.9 m (for Boreholes 03-173 and 03-174, respectively) and the layer was about 200 mm thick.

4.4.2.2 Clayey Silt

Immediately beneath the fibrous peat, a layer of brown clayey silt with trace to some sand, trace organics, trace gravel, containing sand seams was encountered. The elevation of the top of this layer was 278.9 m and 278.7 m and the layer was 1.2 m and 1.3 m thick for Boreholes 03-173 and 03-174, respectively.

Standard Penetration Testing (SPT) carried out within the clayey silt stratum measured 'N' values of between 3 blows to 15 blows per 0.3 m of penetration indicating a soft to stiff consistency.

A grain size distribution for one sample of the clayey silt layer is shown on Figure A1 of Appendix A. The natural water content measured on selected samples of the clayey silt varied between 17 percent and 25 percent, with an average of about 21 percent.

4.4.2.3 Silt

Underlying the clayey silt layer, a stratum of brown to grey silt containing trace to some sand, trace gravel, and trace organics was encountered in both boreholes put down in this area. Occasional sand seams were encountered within the silt layer. The elevation of the top of this layer was 277.7 m and 277.4 m and the thickness of the layer was 1.7 m and 1.9 m for Boreholes 03-173 and 03-174, respectively.

Standard Penetration Testing (SPT) carried out within the silt deposit measured 'N' values of between 13 blows to 55 blows per 0.3 m of penetration, which indicates a compact to very dense state of packing.

The natural water content of two selected samples of the silt was measured at 13 percent and 20 percent.

4.4.2.4 Silty Sand to Sand

A deposit of brown silty sand to sand was encountered beneath the silt layer in both boreholes put down in this area. The silty sand to sand deposit contained trace to some gravel and occasional cobbles. The top of this deposit was at Elevation 276.1 m and 275.6 m and the thickness of the deposit was 0.9 m and 1.2 m for Boreholes 03-173 and 03-174, respectively.

Standard Penetration Testing (SPT) carried out within the silty sand to sand deposit measured 'N' values of between 7 blows and 47 blows per 0.3 m of penetration indicating a loose to dense state of packing. Refusal to further split spoon sampling and augering was encountered below this deposit on probable bedrock at elevations of 275.1 m and 274.3 m in Boreholes 03-173 and 03-174, respectively.

The natural water content measured on selected samples of this silty sand to sand deposit ranged between 12 percent and 24 percent.

4.4.2.5 Groundwater Conditions

In general, the samples taken in the boreholes were noted to be wet. Details of the groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized

on the Record of Borehole sheets following the text of this report. Based on observations made in the open boreholes during mid-summer, the groundwater level is 1.2 m and 1.3 m below ground surface (Elevation 277.9 m and 277.6 m) at Boreholes 03-173 and 03-174, respectively. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

4.5 Detour #2 (South Side of Hwy 17 / N/S-E Ramp)

The proposed Detour #2 alignment generally extends from the proposed N/S-E ramp, starting near the east limit of Swamp area A8, and crosses over shallow overburden and several bedrock outcrops and continues east crossing Swamp area A13 as shown on Drawing 1, 6, and 7.

4.5.1 High Fill (Sta. 16+180 to 16+230)

A plan and profile drawing along the proposed Detour #2 alignment showing the borehole location and interpreted stratigraphy between Stations 16+180 and 16+230 is shown on Drawing 6. One borehole (BH 03-175) was advanced to determine the subsurface conditions in this area.

The borehole was advanced to a depth of 5.6 m (Elevation 272.4 m) and was terminated on inferred bedrock (auger and sampling spoon refusal).

In general, the subsurface conditions along the proposed Detour #2 alignment in this area consist of a surficial deposit of fibrous peat, underlain by silt, underlain by sand over inferred bedrock. These subsoils are consistent with those encountered in boreholes BH 03-30 and 03-32 put down to investigate the east side of Swamp A8 as presented in our previous foundation report dated May 2003.

4.5.1.1 Fibrous Peat

A surficial layer of dark brown to black fibrous peat was encountered in BH 03-175 that was 100 mm thick.

4.5.1.2 Silt

A layer of brown silt containing some sand and trace organics was encountered below the fibrous peat. The top of this layer was at Elevation 277.9 and the layer was 0.7 m thick.

Standard Penetration Testing (SPT) carried out within the silt deposit measured an 'N' value of 2 blows per 0.3 m of penetration, which indicates a very loose state of packing.

The natural water content of a selected sample of the silt was measured at 28 percent.

4.5.1.3 Sand

A deposit of brown to grey sand was encountered beneath the silt layer in Borehole 03-175 put down in this area. The sand deposit contained trace to some silt, trace clay, and occasional silt interlayers. The deposit contained trace to some gravel below a depth of about 4 m below ground surface (Elevation 274.0). The top of this sandy deposit was at Elevation 277.2 m and the deposit was 4.8 m thick.

Standard Penetration Testing (SPT) carried out within the sand deposit measured 'N' values of between 9 blows and 45 blows per 0.3 m of penetration indicating a loose to dense state of packing. Refusal to further split spoon sampling and augering was encountered at the bottom of this deposit on probable bedrock at Elevation 272.4 m.

A grain size distribution for one sample of the sand deposit is shown on Figure A2 of Appendix A The natural water content measured on selected samples of this sand deposit ranged between 20 percent and 22 percent.

4.5.1.4 Groundwater Conditions

Details of the groundwater conditions and water level observed in the open borehole at the time of drilling are summarized on the Record of Borehole sheet following the text of this report. Based on observations made in the open borehole during mid-summer, the groundwater level is 2.1 m below ground surface (Elevation 275.9 m). It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

4.5.2 High Fill (Sta. 16+230 to 16+430)

Plan and profile drawings along the proposed Detour #2 alignment showing the borehole locations and interpreted stratigraphy between Stations 16+230 and 16+430 are shown on Drawing 6. A total of four probeholes (Probeholes PH03-01 to PH03-04 inclusive) were advanced within the high fill area to estimate the depth to bedrock (overburden thickness) along the proposed new alignment.

The probeholes in this section were advanced to depths ranging between 0.5 m and 0.9 m below ground surface; with one probehole achieving refusal at ground surface (i.e. on a bedrock outcrop). All of the probeholes were terminated on inferred bedrock, which was taken to be

refusal to further advancement of the probe. The inferred bedrock surface ranged from Elevation 280.1 m to 283.4 m.

The surficial soils, where present, at the probehole locations generally consisted of silty sand with cobbles and boulders.

4.5.3 Swamp A13 (Sta. 16+430 to 16+500)

A plan and profile drawing along the proposed Detour #2 alignment showing the borehole locations and interpreted stratigraphy in Swamp A13 between Stations 16+430 and 16+500 are shown on Drawing 7. A total of three boreholes (Boreholes 03-176, 03-177, and 03-178) were put down within the swamp area to investigate the subsurface conditions near the proposed new Detour #2 embankment toe alignment.

The boreholes in this section were advanced to depths ranging between 1.2 m and 6.7 m below ground surface (Elevation ranging between 270.2 m and 275.7 m), all of which were terminated on inferred bedrock. The presence of bedrock was inferred from either refusal to further drilling or refusal of sampling spoon advance during SPT testing.

In general, the subsurface soils along the proposed new Detour #2 toe alignment in this swamp area consist of an organic layer of fibrous peat underlain by a deposit of silty sand to sand, underlain by a deposit of varved silty clay, underlain by deposits of silt and sand over inferred bedrock.

4.5.3.1 Fibrous Peat

A surficial layer of dark brown to black fibrous peat was encountered in all of the boreholes put down in this area. In Borehole 03-177, the fibrous peat was encountered below 0.3 m of standing water. The fibrous peat ranges in thickness from 0.2 m to 0.3 m.

4.5.3.2 Silty Sand to Sand

A deposit of brown and grey silty sand to sand, some silt was encountered beneath the fibrous peat in all of the boreholes put down in this area. The silty sand to sand contained trace gravel, clay and cobbles, and contained silty clay interlayers. The elevation of the top of this layer varied between 276.3 m and 276.7 m and the thickness ranged from 0.9 m to 2.1 m.

Standard Penetration Testing (SPT) carried out within the silty sand to sand deposit typically measured 'N' values of between 16 blows and 25 blows per 0.3 m of penetration indicating a

compact state of packing. Refusal to further split spoon sampling and augering was encountered at the bottom of this deposit on probable bedrock at Elevation 275.7 m in Borehole 03-176.

The natural water content measured on selected samples of this silty sand to sand deposit ranged between 22 percent and 30 percent.

4.5.3.3 Varved Silty Clay

Varved silty clay deposits containing sand seams were encountered below the fibrous peat, silty sand, and sand layers in Boreholes 03-177 and 03-178. The elevation of the top of this layer was 274.6 m to 274.8 m and the layer was 1.4 m and 1.5 m thick in Boreholes 03-177 and 03-178, respectively.

Standard Penetration Testing (SPT) measured 'N' values were 1 blow and 4 blows per 0.3 m of penetration, indicating a very soft to firm consistency within this deposit.

The results of field vane testing carried out within the deposit are summarized in the following table:

Borehole	*Elevation/Depth (m)*	*Undrained Shear strength (kPa)*	*Remoulded Shear Strength (kPa)*	*Sensitivity*
BH03-177	273.7 / 3.2	28.7	4.3	6.7
BH03-177	273.4 / 3.5	34.5	7.2	4.8
BH03-177	272.3 / 4.6	28.7	7.2	4.0
BH03-178	273.7 / 3.2	17.2	7.2	2.4
BH03-178	273.4 / 3.5	8.6	4.3	2.0
Average	-	23.5	6.0	4.0

In situ vane testing carried out within the deposit measured undrained shear strengths ranging between 9 kPa and 35 kPa; with an average value of 24 kPa. These values indicated the deposit has a very soft to firm consistency, which is consistent with the SPT test results. The remoulded in situ vane values indicates that the sensitivity of the deposit ranged between 2.0 and 6.7, implying the deposit is medium sensitive to sensitive based on the classification system provided in the CFEM (1992).

Laboratory Atterberg Limits testing was carried out on two samples of the varved silty clay deposit. The test results are summarized in the following table.

Borehole	*Sample*	*Elevation (m)*	*Liquid Limit (%)*	*Plastic Limit (%)*	*Plasticity Index (%)*
BH03-177	3	274.0 – 274.6	48	19	29
BH03-178	4	274.0 – 274.6	46	18	27
Average	-	-	47	19	28

The results of the Atterberg limits testing are shown on the plasticity chart on Figure A6 in Appendix A. The natural water content measured on selected samples of this varved silty clay deposit were 49 percent and 53 percent.

4.5.3.4 Silty Sand, Sand, and Silt

Underlying the varved silty clay, deposits of silty sand, sand, and silt were encountered in Boreholes 03-177 and 03-178. The deposits contained trace to some clay and variable amounts of gravel and cobbles. The elevation of the top of this layer was at 273.4 m and 273.1 m and it was 3.2 m and 1.2 m thick in Boreholes 03-177 and 03-178, respectively.

Standard Penetration Testing (SPT) carried out within the silty sand, sand, and silt stratum measured 'N' values ranging from 2 blows to greater than 100 blows per 0.3 m of penetration indicating a very loose to very dense state of packing. The SPT sample associated with the 'N' values that indicate a very dense state of packing typically contained gravel sizes and/or pieces of larger particles. Refusal to further split spoon sampling and augering was encountered below this stratum on probable bedrock at Elevations 270.2 m and 272.0 m in Boreholes 03-177 and 03-178, respectively.

The natural water content measured on selected samples of this silty sand, sand, and silt stratum ranged between 18 percent and 23 percent.

4.5.3.5 Groundwater Conditions

Based on observations made in the open boreholes upon completion of drilling and the fact that there was approximately 300 mm of standing water in Borehole 03-177, the groundwater level is inferred to be at the existing ground surface in this area. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

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PART B

DETAILED
FOUNDATION INVESTIGATION AND DESIGN
DETOUR #1, DETOUR #2
AND WIDENING OF MUNICIPAL ROAD 80
FROM HARRISON DRIVE TO HIGHWAY 17 INTERCHANGE
G.W.P 99-98-00
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 54, SUDBURY

Golder Associates

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides our interpretation of the factual geotechnical data obtained during the investigation and our recommendations on the foundation aspects of design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

5.1 General

The overall project involves the design of a new Highway 17 interchange at Sudbury Municipal Road 80 (Long Lake Road) in Sudbury, Ontario. This report provides foundation recommendations for the areas of high embankments and swamp crossings that are associated with two proposed detour alignments and the widening of Municipal Road 80 to four lanes.

The following table summarizes the locations of the areas investigated within the project limits requiring foundation design.

Location	*Proposed Works*	*Chainage*	*Swamp Area*
East Side of MR 80 (widening north bound lanes and turning lanes at Harrison / Countryside Drive Intersection)	Swamp Crossing (fill up to 2 m)	9+000 to 9+340	A10
	Swamp Crossing (fill up to 4 m)	9+340 to 9+470	A12E
West Side of MR 80 and Detour #1 (widening south bound lanes)	Swamp Crossing (fill up to 5.5 m)	9+290 to 9+510	A12W
	Swamp Crossing (fill up to 2.5 m)	9+850 to 9+950	A2
Detour #2 (south side of Hwy 17 / N/S-E Ramp)	High Fill (fill up to 9 m)	16+180 to 16+230	
	High Fill (fill up to 7 m)	16+230 to 16+430	
	Swamp Crossing (fill up to 8 m)	16+430 to 16+500	A13

Golder Associates Ltd. (Golder) has been retained by Earth Tech (Canada) Inc. (EarthTech) to provide recommendations on the foundation aspects related to the final design and construction of

the high fill embankments and swamp crossings at the areas noted above. The scope of work includes stability and settlement analysis, recommendations for stable embankment geometry, fill materials used, implementation of ground improvement techniques that may be required and means to minimize settlements (where required). It also includes addressing specialized construction concerns and potential geotechnical problems including sub-excavating soft / organic materials and placement of fill materials. These requirements are addressed in the following sections.

5.2 High Fill and Swamp Embankments

The existing Municipal Road 80 embankments in the area of the proposed widening and Detour #1 alignment range in height from about 1.5 m to 4.5 m and have side slope profiles of approximately 1.25H:1V to 2H:1V. These embankments appear to be primarily composed of rock fill and/or sand and gravel fill. The existing Highway 17 embankments in the area of the Detour #2 alignment range in height from about 3 m to 8 m and have side slope profiles of approximately 1.25 horizontal (H): 1 vertical (V). These embankments appear to be primarily composed of rock fill.

Based on profiles of the MR 80 widening and detour alignments provided to us by EarthTech, it is our understanding that the new MR 80 roadway and detour grades will require fill embankments ranging in height from about 2 m to 5.5 m, and 1 m to 9 m, respectively. A portion of the MR 80 widened roadway alignment will pass through existing rock outcrops requiring cuts up to about 2.5 m. Recommendations on the excavation through rock are provided in Section 5.6.

In the following sections, the results of stability and settlement analysis for the new embankments in the critical areas are presented. Recommendations on mitigation of time dependent settlements in the areas where the subsoils contain fine-grained cohesive deposits are provided in subsequent sections.

5.2.1 Stability

Stability analyses were performed on the critical sections of the proposed fill embankments. For this report, critical sections are assumed to correspond to the greatest new embankment height and steepest embankment sections. In all areas where cohesive strata where encountered in the subsoils, the stability of the proposed new or widened embankment sections were analysed using the limit equilibrium method. In areas where the subsoils consisted of cohesionless soils only, the stability of the proposed embankment section was assessed based on precedent experience in similar soil conditions. Deposits of soft to stiff, clayey silt to silty clay to clay were encountered in the following areas :

Location	*Proposed Works*	*Chainage*	*Swamp Area*
East Side of MR 80 (widening north bound lanes and turning lanes at Harrison / Countryside Drive Intersection)	Swamp Crossing (fill up to 2 m)	9+000 to 9+340	A10
	Swamp Crossing (fill up to 4 m)	9+340 to 9+470	A12E
West Side of MR 80 and Detour #1 (widening south bound lanes)	Swamp Crossing (fill up to 5.5 m)	9+290 to 9+510	A12W
	Swamp Crossing (fill up to 2.5 m)	9+850 to 9+950	A2
Detour #2 (south side of Hwy 17 / N/S-E Ramp)	Swamp Crossing (fill up to 8 m)	16+430 to 16+500	A13

All slope stability analyses were performed using the commercially available program SLOPE/W (Version 5.13), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target factor of safety of 1.3 is normally used for the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at these sites considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the various embankment heights, widenings and geometries.

The subsoils encountered in the various areas are composed of cohesionless soils or a combination of cohesive and cohesionless soils. For cohesionless layers, effective stress parameters were employed in the analysis assuming drained conditions for the soils. The effective stress parameters for the cohesionless soils were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al. (1974), Schmertmann (1975) and US Navy (1971) were employed, and the results tempered by engineering judgement considering experience in similar soil conditions.

For cohesive layers, total stress parameters were employed in the analysis. The total stress parameters (i.e. average operative undrained shear strength – s_u) for the cohesive soils were

assessed based on the results of field vane tests and estimated from correlations with the SPT results and other laboratory test data.

A site specific correlation was developed relating natural moisture content to undrained shear strength as measured with the field vane. This correlation was utilized to estimate shear strengths in thin clayey layers where it was not possible to carry out in situ field vane tests.

Where varved clayey soils were encountered, a 0.75 correction factor was applied to the undrained shear strength estimated from the above methods in order to account for the effect of the weak clay layers interspersed between the stronger silt or clayey silt layers.

At all areas, the analyses assume that the organic soils (encountered at or below the ground surface during drilling operations) have been removed prior to construction of the new embankments. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and measured in the standpipe installations. In general, the groundwater level is located at about the elevation of the existing ground surface.

The following tables summarize the simplified stratigraphy and the associated strength and unit weight employed for the different soil types for each of the critical areas.

Swamp Area A10 (East Side MR 80 STA 9+000 to 9+340)

Soil	*Unit Weight (kN/m^3)*	*Strength*
Rock Fill	18	$c' = 0$ kPa, $\phi' = 38^\circ$
Sand and Gravel Fill	20	$c' = 0$ kPa, $\phi' = 35^\circ$
Fibrous Peat (beyond toes)	12	$c_u = 16$ kPa
Silt	19	$c' = 0$ kPa, $\phi' = 32^\circ$
Varved Silty Clay	18	$c_u = 20$ kPa

Swamp Area A12E (East Side MR 80 STA 9+340 to 9+470)

Soil	*Unit Weight (kN/m^3)*	*Strength*
Rock Fill	18	$c' = 0$ kPa, $\phi' = 38^\circ$
Sand and Gravel Fill	20	$c' = 0$ kPa, $\phi' = 35^\circ$
Fibrous Peat (beyond toes)	12	$c_u = 16$ kPa
Silt	19	$c' = 0$ kPa, $\phi' = 32^\circ$
Varved Silty Clay	20	$c_u = 16$ kPa

Swamp Area A12W (West Side MR 80 / Detour #1 STA 9+290 to 9+450)

Soil	*Unit Weight (kN/m^3)*	*Strength*
Rock Fill	18	$c' = 0$ kPa, $\phi' = 38^\circ$
Sand and Gravel Fill	20	$c' = 0$ kPa, $\phi' = 35^\circ$
Fibrous Peat (beyond toes)	12	$c_u = 16$ kPa
Silt to Silty Sand	19	$c' = 0$ kPa, $\phi' = 32^\circ$
Varved Silty Clay	18	$c_u = 16$ kPa
Silty Sand	20	$c' = 0$ kPa, $\phi' = 33^\circ$

Swamp Area A12W (West Side MR 80 / Detour #1 STA 9+450 to 9+510)

Soil	*Unit Weight (kN/m^3)*	*Strength*
Rock Fill	18	$c' = 0$ kPa, $\phi' = 38^\circ$
Sand and Gravel Fill	20	$c' = 0$ kPa, $\phi' = 35^\circ$
Fibrous Peat (beyond toes)	12	$c_u = 16$ kPa
Silt to Silty Sand	19	$c' = 0$ kPa, $\phi' = 32^\circ$
Varved Silty Clay	18	$c_u = 22$ kPa
Silty Sand	20	$c' = 0$ kPa, $\phi' = 33^\circ$

Swamp Area A2 (West Side MR 80 / Detour #1 STA 9+850 to 9+950)

Soil	*Unit Weight (kN/m^3)*	*Strength*
Rock Fill	18	$c' = 0$ kPa, $\phi' = 38^\circ$
Sand and Gravel Fill	20	$c' = 0$ kPa, $\phi' = 35^\circ$
Fibrous Peat (beyond toes)	12	$c_u = 16$ kPa
Clayey Silt	19	$c_u = 30$ kPa
Silt	20	$c' = 0$ kPa, $\phi' = 32^\circ$
Sand	20	$c' = 0$ kPa, $\phi' = 33^\circ$

Swamp Area A13 (Detour #2 STA 16+430 to 16+500)

Soil	*Unit Weight (kN/m^3)*	*Strength*
Rock Fill	18	$c' = 0$ kPa, $\phi' = 38°$
Sand and Gravel Fill	20	$c' = 0$ kPa, $\phi' = 35°$
Fibrous Peat (beyond toes)	12	$c_u = 16$ kPa
Sand	19	$c' = 0$ kPa, $\phi' = 33°$
Varved Silty Clay	18	$c_u = 10$ kPa
Silty Sand	20	$c' = 0$ kPa, $\phi' = 33°$

In the analyses, two different types of fill (i.e. rock fill and earth fill) were considered for the required fill embankments and widenings. The fill alternatives provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to native subsoils), construction cost and time, and ease of construction / availability. A brief description of each alternative is described below.

Earth Fill (Sand and Gravel) Embankment

This option assumes that conventional earth fill is used for the construction of the embankment. The main advantage of this option is the ease of construction and the lack of post-construction settlements within the fill embankment itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes tend to be flatter than rock fill slopes. For this project, acceptable earth fill is considered to be suitable locally available and/or imported, free-draining granular material.

Based on our observations during the field investigation, the existing Highway 17 roadway embankments are composed mainly of rock fill. The existing MR 80 roadway embankments are composed of earth fill (between about Station 9+000 to 9+300) and rock fill (between about Station 9+300 to 9+950). In most places (i.e. for the MR 80 widening, Detour #1 and where Detour #2 ties in to the existing Highway 17), the fill will be required to be placed immediately adjacent to the existing MR 80 and Highway 17 embankments.

It is recommended that sand and gravel (earth fill) be used where the existing embankments are composed of earth fill, such as for the widening of MR 80 between Station 9+000 to 9+300. If sand and gravel (earth fill) is used in areas where the adjacent embankments are composed of rock fill, then provision must be made to have a separator (either a geotextile filter or a graded granular filter) between the dissimilar fills to avoid loss of fines from the new earth fill embankment into the existing rock fill embankment. Otherwise, consideration could be given to

using earth fill where the new roadway embankments are located away from the existing rock fill embankments (i.e. Detour #2 from 16+180 to 16+230).

Rock Fill Embankment

This option would employ rock fill rather than earth fill for the embankment. The main advantage of this material is the ability to achieve steeper embankment side slopes. This is useful in areas with limited right-of-ways. In addition, the surplus rock fill available from the deep cuts through the existing bedrock outcrops on the project, would have a cost advantage. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first year of construction.

The use of rock fill for embankment widening is considered most suitable in areas where the existing embankment fill is composed primarily of rock fill. If rock fill is used for embankment widening where the existing or new adjacent embankment is composed of earth fill, provision must be made to have a separator (either a geotextile filter or a graded granular filter) between the dissimilar fills to avoid loss of fines from the earth fill embankment into the rock fill embankment.

Although rock fill is subject to settlement due to compression of the rock fill itself (as discussed in Section 5.2.2.3) the use of similar adjacent fill materials will preclude problems caused by the migration of fines between dissimilarly graded fill types as well as potential variation in thermal effects related to different materials. For this reason, we recommend the use of rock fill for the construction of the new embankments in all areas where the existing roadway embankments are constructed of a similar material.

The results of the stability analyses for the two options are summarized in the following table. At each area, the highest (i.e. most critical) embankment section has been analysed. A recommended side slope profile and the associated factor of safety for each of the critical areas on the project (where the subsoils contain cohesive soil strata), is presented below.

Location	*Embankment Height at Critical Section (m)*	*Earth fill Option*		*Rock fill Option*	
		Recommended Side Slope Profile	*Minimum Factor of Safety*	*Recommended Side Slope Profile*	*Minimum Factor of Safety*
Swamp A10 East Side of MR 80 9+000 to 9+340 *(Figure 2)*	2	2H : 1V	≥ 1.3	1.25H : 1V	≥ 1.3
Swamp A12E East Side of MR 80 9+340 to 9+470 *(Figure 3)*	4				
Swamp A12W West Side of MR 80 / Detour #1 9+290 to 9+400 *(Figure 4)*	2.5				
Swamp A12W West Side of MR 80 / Detour #1 9+400 to 9+450 *(Figure 5)*	5	(see Section 5.2.1.1)			
Swamp A12W West Side of MR 80 / Detour #1 9+450 to 9+510 *(Figure 6)*	6	(see Section 5.2.1.2)			
Swamp A2 West Side of MR 80 / Detour #1 9+850 to 9+950 *(Figure 7)*	2.5	2H : 1V	≥ 1.3	1.25H : 1V	≥ 1.3
Swamp A13 Detour #2 16+430 to 16+500 *(Figure 8)*	8	(see Section 5.2.1.3)			

The minimum factor of safety for each area summarized above is based on a deep-seated, global trial failure surface that would impact the operation of the roadway. The results of the limit equilibrium stability analysis for each of the above areas are presented on Figures 2 to 8.

The incorporation of a 2 m wide bench (or berm) into the uniform side slope profile is required at certain sections of the proposed fill embankments as per OPSD – 202.010 and MTO Northern

Region guidelines. The presence of a berm will increase the internal and surficial stability of the embankment and aid in surface water control on the slope. The presence of this berm has been incorporated in the stability analysis, where required.

In areas where earth fill embankments will exceed a height of 8 m, a 2 m wide mid-height berm is required and will be suitable for the proposed roadway embankments. In areas where rock fill embankments will exceed a height of 6 m, 2 m wide berms (or successive benches) are required so that the uninterrupted rock fill slope never exceeds a height of 6 metres (as per MTO Northern Region guidelines) and will be suitable for the proposed upgrading and widening of the highway.

The recommended embankment side slope profiles for all of the high fill and swamp crossing areas associated with the MR 80 widening and detour alignments (including those founded on cohesionless subsoils) are summarized in Table 1. Included on this table are the recommended requirements for side slope berms and measures to mitigate the embankment problems as indicated in the following sections.

5.2.1.1 Swamp A12W – West Side of MR 80 (Station 9+400 to 9+450)

On the Detour #1 / west side widening of MR 80 alignment between Station 9+400 to 9+450 (Swamp Area A12W), a very soft to firm varved silty clay layer was encountered in boreholes BH 03-181 and 03-182. The elevation of the top of this stratum is at about Elev. 268.5 m, which is about 2.5 m below the existing ground surface. The soft clayey stratum has an average total thickness of about 3.4 m and operative undrained shear strengths as low as about 16 kPa.

To achieve a Factor of Safety (FoS) = 1.3 for the up to 5 m high embankment fill in this area, it would be necessary to construct earth fill or rock fill berms at the west toe of this embankment. Stability analyses indicate that toe berms approximately 6 m wide by 2 m high would be necessary to achieve an adequate FoS at this location (see Figure 5).

The size of the berm required for embankment stability at this location could make this option impractical. As such, other stability mitigation options should be considered including full sub-excavation and removal of the weak/soft soils. The advantages, disadvantages, relative costs and risks/consequences for the mitigation options at this area is discussed in Section 5.3 and summarized in Table 3.

As will be discussed in Section 5.2.2, the soft clay layers in this area will also cause time dependent (consolidation) settlements of the new embankment. The sub-excavation option would also mitigate the long-term (i.e. post-construction) settlements. These and other alternatives to

mitigate settlements are discussed in Section 5.3 and included in Table 3. The full sub-excavation option has been ranked as the preferred alternative for this area (as shown in Table 3).

5.2.1.2 Swamp A12W – West Side of MR 80 (Station 9+450 to 9+510)

On the Detour #1 / west side widening of MR 80 alignment between Station 9+450 to 9+510 (Swamp Area A12W), a soft to stiff varved silty clay layer was encountered in boreholes BH 03-182 and 03-183. The elevation of the top of this stratum is at about Elev. 268 m, which is about 3 m below the existing ground surface. The soft clayey stratum has an average total thickness of about 2.9 m and operative undrained shear strengths as low as about 22 kPa.

To achieve a Factor of Safety (FoS) = 1.3 for the up to 6 m high embankment fill in this area, it would be necessary to construct earth fill or rock fill berms at the west toe of this embankment. Stability analyses indicate that toe berms approximately 3 m wide by 2 m high would be necessary to achieve an adequate FoS at this location (see Figure 6).

Although the size of the berm required for embankment stability at this location is not impractical, other stability mitigation options should be considered including full sub-excavation and removal of the weak/soft soils. The advantages, disadvantages, relative costs and risks/consequences for the mitigation options at this area is discussed in Section 5.3 and summarized in Table 3.

As will be discussed in Section 5.2.2, the soft clay layers in this area will also cause time dependent (consolidation) settlements of the new embankment. The sub-excavation option would also mitigate the long-term (i.e. post-construction) settlements. These and other alternatives to mitigate settlements are discussed in Section 5.3 and included in Table 3. The full sub-excavation option has been ranked as the preferred alternative for this area (as shown in Table 3).

5.2.1.3 Swamp A13 – Detour #2 (Station 16+430 to 16+500)

On the Detour #2 alignment between Station 16+430 to 16+500, a very soft to firm varved silty clay layer was encountered in boreholes BH 03-177 and 03-178. The top of this stratum is located about 2.2 m below the existing ground surface underlying deposits of soft organic peat and compact sand. The very soft to firm varved silty clay stratum has a total thickness of about 1.5 m and operative undrained shear strengths are estimated to be as low as about 15 kPa.

To achieve a Factor of Safety (FoS) = 1.3 for the 8 m high embankment fill in this area, it would be necessary to construct earth fill or rock fill berms at the toe of this embankment. Stability

analyses indicate that a toe berm approximately 5.5 m wide by 4 m high would be necessary to achieve an adequate FoS at this location (see Figure 8).

Although the size of the berm required for embankment stability at this location is not impractical, other stability mitigation options were considered including full sub-excavation and removal of the weak/soft soils. A discussion of the advantages, disadvantages, relative costs and risks/consequences for the mitigation options at this area are presented in Table 4.

As will be discussed in Section 5.2.2, the soft clay layers in this area will also cause time dependent (consolidation) settlements of the new embankment. The sub-excavation option would also mitigate the long-term (i.e. post-construction) settlements. These and other alternatives to mitigate settlements are discussed in Section 5.3 and included in Table 4. The full sub-excavation option has been ranked as the preferred alternative for this area (as shown in Table 4).

5.2.2 Settlement

Settlement analyses were performed on the critical sections of the proposed fill embankments and widenings. For these analyses, the critical sections are assumed to correspond to the greatest new embankment height. At Swamp A13, where the subsoils are thick and where the required widening for Detour #2 is relatively narrow relative to the existing embankment, the settlement analysis was carried out using the commercially available program UNISETTLE (Version 3.0) produced by Unisoft Limited to more accurately assess the effects of the new widening relative to the adjacent existing embankment loading. In the other areas where the subsoils are thinner and where the required widenings are larger in relation to the existing roadway, the settlement analysis were performed using hand calculations.

The subsoils encountered at depth at the majority of the areas investigated are composed of both cohesionless soils (silt, silty sand and sand) and cohesive compressible strata (clayey silt and varved silty clay). Surficial deposits of soft organic soils (i.e. topsoil and/or fibrous peat) were encountered in most of the boreholes. Subsoils composed of cohesionless strata and/or bedrock only were encountered along the alignment of Detour #2 between Stations 16+180 to 16+430.

Provided that the surface (and near surface) deposits of topsoil and organic materials are removed prior to the new embankment fill placement (as discussed in Section 5.3 and 5.4), the settlement of the new fill embankments (due to the stress imposed by the new loading on the existing subsoils) in the areas where the subsoils consist of cohesionless soils only, is expected to be small. For embankment fills constructed over the swamp areas A2, A10, A12E, A12W and A13, some consolidation settlement with time is expected.

In the areas where new embankments will be constructed with rock fill, settlement of the new roadway embankment is also expected due to compression of the rock fill itself. In addition, depending on the type of fill material used and settlement mitigation measure adopted, some differential settlement may also be expected where the new embankment fills are placed adjacent to the existing roadway embankments (i.e. for the MR 80 widenings and detour tie-ins).

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

5.2.2.1 Settlement of Cohesionless Foundation Soils

Between Station 16+180 to 16+430 along the alignment of Detour #2, the subsoils encountered in the boreholes consisted of cohesionless soils only. In this area, the detour embankment fill sections range in height from about 1 m to 9 m. Considering that some of the rock fill quantities will be made available during the excavation of the deep cuts through the existing bedrock outcrops for other sections of this project, the detour embankments for this area were analysed assuming a rock fill composition, γ = 18 kN/m^3, 1.25H:1V side slopes and the maximum new height of embankment.

The settlement analyses performed assume that organic soils (encountered at or below the ground surface during drilling operations) have been removed prior to construction of the new embankments. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and measured in the standpipe installations. In general, in the swamp areas, the groundwater level is located at about the elevation of the ground surface.

The immediate compression of the loose to dense, sand to silty sand strata encountered in the boreholes in this areas were modelled by estimating an elastic modulus of deformation based on the SPT 'N' values and correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

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

Location	*Chainage*	*Maximum New Embankment Height* (m)*	*Estimated Settlement of Foundation Soils (mm)*
Detour #2	16+180 to 16+230	9 +0.1 = 9.1	60*
	16+230 to 16+430	7 +0.1 = 7.1	10**

Notes : *includes additional fill required after removal of maximum depth of organics.
**shallow foundation soils in this area after organics removed.

These settlements are expected to occur rapidly (i.e. during or shortly after construction) in response to the construction based on the estimated relatively high permeability of the soils as indicated by the results of the grain size distributions. In addition to the above foundation soil settlements, embankment settlement due to compression of the rock fill itself will occur as is discussed in Section 5.2.2.3 and summarized in Table 1.

5.2.2.2 Settlement of Cohesive Foundation Soils

Cohesive foundation soils were encountered in swamp areas A2, A10, A12E, A12W and A13. Settlement analysis were carried out for these areas using the results of the borehole information, in situ field test data (field vane and SPT), and consolidation data (from oedometer tests performed on specimens from other areas of the site) to estimate deformation parameters of the subsoils.

Considering that some of the rock fill quantities will be made available during the excavation of the deep cuts through the existing bedrock outcrops, most of the new embankments were analysed assuming a rock fill composition and 1.25H:1V side slopes. However, for the widening of MR 80 between Station 9+000 to 9+300 (at Swamp A10), the widened embankment sections were analysed using an earth fill composition and 2H:1V side slopes considering that the existing roadway embankment is mainly composed of sand and gravel fill in this area.

The immediate compression of the very loose to very dense silt, silty sand and sand layers was modelled by estimating an elastic modulus of deformation based on the SPT 'N' values and correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

At all areas, the settlement analyses assume that organic soils (encountered at or below the ground surface during drilling operations) have been removed prior to construction of the new embankments. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and measured in the nearby standpipe installations. In general, in the swamp areas, the groundwater level is located at about the elevation of the ground surface.

The following sections summarize the simplified stratigraphy and the associated unit weight and deformation parameters employed for the different soil types for each of the critical areas. In these sections, the maximum estimated settlement of the foundation soils in these areas is presented and a discussion on the time rate of settlement is provided.

5.2.2.2.1 Swamp Area - A10 (East Side of MR 80 – STA 9+000 to 9+340)

The following simplified stratigraphy has been developed for and employed in the settlement analysis of the proposed 2 m high earth fill and rock fill embankment in Swamp A10.

Soil	*Thickness (m)*	*Unit Weight (kN/m^3)*	*Deformation Properties*
Earth Fill (9+000 to 9+300) Rock Fill (9+300 to 9+340) (2 m embankment + 0.9 m after organics removed)	2.9 (high)	20 18	-
Silt	1.4	19	E' = 5 MPa
Varved Silty Clay	3.4	18	$m_v = 9.0 \times 10^{-4}$ kPa^{-1}

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in this area is estimated to be about 190 mm. The time dependent settlement of the cohesive soil layers comprises about 175 mm of this total. Assuming a coefficient of consolidation (c_v) of 1.1×10^{-2} (cm^2/s) and one-way drainage in the approximately 3.4 m thick clayey silt layer, it is estimated that about 95 percent of the settlement of the cohesive layer will be completed in about 4.5 months.

5.2.2.2.2 Swamp Area - A12E (East Side of MR 80 – STA 9+340 to 9+470)

The following simplified stratigraphy has been developed for and employed in the settlement analysis of the proposed up to 5.5 m high rock fill embankment in Swamp A12E.

Soil	*Thickness (m)*	*Unit Weight (kN/m^3)*	*Deformation Properties*
Rock fill (5.5 m embankment + 0.5 m after organics removed)	6.0 (high)	18	-
Silt	0.6	19	E' = 15 MPa
Varved Silty Clay	1.1	18	$m_v = 3.8 \times 10^{-4}$ kPa^{-1}

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in this area is estimated to be about 50 mm. The time dependent settlement of the cohesive soil layers comprises about 45 mm of this total. Assuming a coefficient of consolidation (c_v) of 5.0×10^{-3} (cm^2/s) and one-way drainage in the approximately 1.1 m thick clayey silt layer, it is estimated that about 95 percent of the settlement of the cohesive layer will be completed in about 1 month.

5.2.2.2.3 Swamp Area - A12W (West Side of MR 80 – STA 9+290 to 9+400)

The following simplified stratigraphy has been developed for and employed in the settlement analysis of the proposed up to 2.5 m high rock fill embankment in Swamp A12W.

Soil	*Thickness (m)*	*Unit Weight (kN/m^3)*	*Deformation Properties*
Rock fill (2.5 m embankment + 0.1 m after organics removed)	2.6 (high)	18	-
Silty Sand	0.6	19	E' = 15 MPa
Varved Silty Clay	3.4	18	$m_v = 6.8x10^{-4}$ kPa^{-1}
Silty Sand	1.0	19	E' = 15 MPa

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in this area is estimated to be up to about 110 mm. The time dependent settlement of the cohesive soil layers comprises about 105 mm of this total. Assuming a coefficient of consolidation (c_v) of $2.0x10^{-3}$ (cm^2/s) and two-way drainage in the approximately 1.1 m thick clayey silt layer, it is estimated that about 95 percent of the settlement of the cohesive layer will be completed in about 6 months

5.2.2.2.4 Swamp Area - A12W (West Side of MR 80 – STA 9+400 to 9+510)

The following simplified stratigraphy has been developed for and employed in the settlement analysis of the proposed up to 6 m high rock fill embankment in Swamp A12W.

Soil	*Thickness (m)*	*Unit Weight (kN/m^3)*	*Deformation Properties*
Rock fill (6 m embankment + 0.1 m after organics removed)	6.1 (high)	18	-
Silty Sand	0.6	19	E' = 15 MPa
Varved Silty Clay	3.4	18	$m_v = 6.8x10^{-4}$ kPa^{-1}
Silty Sand	1.0	19	E' = 15 MPa

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in this area is estimated to be up to about 270 mm. The time dependent settlement of the cohesive soil layers comprises about 255 mm of this total. Assuming a coefficient of consolidation (c_v) of $2.0x10^{-3}$ (cm^2/s) and two-way drainage in the approximately 1.1 m thick clayey silt layer, it is estimated that about 95 percent of the settlement of the cohesive layer will be completed in about 6 months.

5.2.2.2.5 Swamp Area – A2 (West Side of MR 80 – STA 9+850 to 9+950)

The following simplified stratigraphy has been developed for and employed in the settlement analysis of the proposed 2.5 m high rock fill embankment in Swamp A2.

Soil	*Thickness (m)*	*Unit Weight (kN/m^3)*	*Deformation Properties*
Rock fill (2.5 m embankment + 0.2 m after organics removed)	2.7 (high)	18	-
Clayey Silt	1.2	19	$m_v = 2.2x10^{-4}$ kPa^{-1}
Silt	2.1	20	E' = 25 MPa
Sand	1.0	20	E' = 20 MPa

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in this area is estimated to be about 25 mm. The time dependent settlement of the cohesive soil layers comprises about 15 mm of this total. Assuming a coefficient of consolidation (c_v) of $5x10^{-3}$ (cm^2/s) and two-way drainage in the approximately 1.2 m thick clayey silt layer, it is estimated that about 95 percent of the settlement will be completed in about 1 week.

5.2.2.2.6 Swamp Area – A13 (Detour #2 – STA 16+430 to 16+500)

The following simplified stratigraphy has been developed for and employed in the settlement analysis of the proposed 8 m high rock fill embankment in Swamp A13.

Soil	*Thickness (m)*	*Unit Weight (kN/m^3)*	*Deformation Properties*
Rock fill (8 m embankment + 0.3 m after organics removed)	8.3 (high)	18	-
Sand	1.8	19	E' = 25 MPa
Silty Clay	1.5	18	$m_v = 8.8x10^{-4}$ kPa^{-1}
Silty Sand	3.0	20	E' = 15 MPa

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in this area is estimated to range from about 170 mm (at the new widened embankment crest) to 35 mm (at the original embankment crest). The time dependent settlement of the cohesive soil layers comprises about 145 mm (at the new embankment crest) and 30 mm (at the original embankment crest) of these totals. Assuming a coefficient of consolidation (c_v) of

$2.0x10^{-3}$ (cm^2/s) and two-way drainage in the approximately 1.5 m thick clayey silt layer, it is estimated that about 95 percent of the settlement will be completed in about 4 weeks.

5.2.2.3 Settlement of Rock Fill

Where rock fill is used for the construction of the embankments, in addition to the embankment settlement due to compression of the cohesionless foundation soils and the consolidation of underlying cohesive layers, there will be settlement due to compression of the rock fill itself. Settlement of the rock fill depends on the method and sequence of placement and compaction of the rock fill. Assuming that that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in the Special Provision, Amendment to OPSS 206 dated September 1999, the settlement of the newly placed rock fill is expected to be relatively small.

For the hard granitic-type rock fill that is likely to be used at this site, it is estimated that the settlements of the new rock fill will be about 1% of the new effective embankment height for embankments up to about 10 m high, so long as OPSS 206 is followed. Therefore, the settlement of the newly placed rock fill (for embankments ranging from about 2 m to 9 m in height + additional rock fill to replace removed organic layers and compressible subsoils) is expected to range from about 25 mm to 120 mm at all fill areas. It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction. The estimated embankment rock fill settlement for each of the high fill and swamp crossing areas is included in Table 1.

5.2.2.4 Differential Settlements at Existing Roadway Embankments

In most of the areas described above, the fill for the new embankments will be required to be placed immediately adjacent to the existing MR 80 or Highway 17 embankments (i.e. for the MR 80 roadway widenings and where the new detour alignments tie in to the existing roadways).

In areas where the foundation soils are composed primarily of cohsionless soils and/or thin layers of cohesive soils (i.e. A2, A12E, Detour #2 between Station 16+180 to 16+430) the settlements described above are expected to occur rapidly during construction (or within about 1 month following construction) and therefore the impact of differential settlement on the long-term performance of the travelled road surface is anticipated to be minor.

However, in areas where the foundation soils are composed primarily of thicker layers of cohesive soils (i.e. MR 80 – A10 and A12W) and where the new rock fill embankment heights are relatively large (i.e. Detour #2 between Station 16+430 to 16+500), the settlements described

above will take longer (due to time dependent consolidation of the thick clay layers and post-construction settlement of the high rock fills). In these areas, there is a potential for post-construction differential settlement to occur between the existing and new portions of the highway. The magnitudes of these differential settlements are on the order of the settlements described in the previous sections.

To minimize the impact of the differential settlements on the performance of the new/widened roadways, we recommend that the new embankment fill in these areas be constructed as early in the contract as possible. Following this approach would allow as much time as possible for the settlements to occur prior to completing the pavement structure on the new roadways. Alternatively, the settlement mitigation measures described in the following sections could be adopted to reduce some of the post-construction differential settlement. However, even where the foundation soil settlement mitigation measures are adopted, some post-construction differential settlement due to compression of high embankment rock fill should be expected.

5.3 Mitigation of Stability Issues / Time Dependent Settlements

As discussed in Section 5.2.1 and 5.2.2, in the areas where the foundation strata consist of cohesionless soils only (i.e. Detour #2 from Station 16+180 to 16+430), it is not anticipated that there will be any embankment stability issues so long as all organic layers are removed prior to construction and the requirements for mid-height berms are incorporated into the design. Similarly, the settlements of the foundation soils in these areas are expected to occur during or shortly after construction, assuming that all of the surface and near surface organic layers have been removed within the footprint of the embankments prior to filling. As such, in these areas there is no need to implement any special construction procedures or schedule to maintain stability or mitigate the foundation soil settlements

In the areas where cohesive soils were encountered in the foundation strata however, time dependent settlements of the new embankment widenings are expected. In addition, in some of these areas, the presence of the weak/soft cohesive strata create stability problems for the new embankment heights. In these areas, consideration needs to be given to following a design and/or construction sequence to achieve the minimum target factor of safety of 1.3 for the proposed new embankment height and geometry and to limit the post-construction settlements and subsequent maintenance on the new roadway pavement structure.

In areas that contain relatively thin cohesive subsoil strata (i.e. Swamp A12E and Swamp A2), it is estimated that 95% of the consolidation settlement of the compressible cohesive layers will occur in less than about 1 month after placement of the embankment fill. As such, for the widenings in these areas it is recommended that the embankment fill be placed as early as

possible in the construction schedule. The embankments in these areas should be constructed to full height at the beginning of the construction contract and allowed to consolidate for at least 6 weeks prior to the construction of the final pavement structure. Following this sequence will reduce the post-construction settlements and need for maintenance of the roadway surface.

For swamp area A10, A12W and A13, the following sections outline the options and recommendations for achieving the target factor of safety for the required embankment geometry and for minimizing the time dependent, post-construction settlements that could affect the performance of the roadway. The advantages, disadvantages, relative costs and risks/consequences for the mitigation options at these areas are also summarized and ranked in Tables 2 to 4 following the text of this report.

In order to help mitigate post-construction settlements and maintain embankment stability in all areas, it is important that all surface and near surface layers of organic materials (i.e. topsoil and/or fibrous peat) be removed prior to the start of fill construction. In areas where new fill embankments are being constructed away from existing roadways, construction procedures should implement the guidelines of OPSD 203.010. In areas where new fill embankments will be constructed immediately adjacent to the existing Highway 17 and/or Municipal Road 80 embankments, construction procedures should implement the guidelines of OPSD 203.020. These guidelines require that the existing embankment side slopes are cut back to a 1H:1V slope prior to removal of organics and placement of new fill in order to excavate as much organic material as possible that may exist under the side slopes of the existing embankments.

5.3.1 Swamp A10 – East Side of MR 80 (Station 9+000 to 9+340)

As indicated in Section 5.2.2.2, a soft to very stiff varved silty clay strata was encountered in most of the boreholes and test pits put down along the east side MR 80 widening alignment that crosses through Swamp A10. The top of the varved clay is located about 1.4 m below the existing ground surface underlying layers of fibrous peat and/or silt. Where encountered, the varved silty clay stratum ranges in thickness from about 0.3 m to 3.4 m. The presence of the compressible varved silty clay stratum influences the magnitude of post-construction settlement of the proposed 2 m high embankment in this area.

In order to achieve the MTO's objective of producing a highway embankment design that will minimize post-construction settlements, the alternatives described in the following sections should be considered.

5.3.1.1 Full Sub-Excavation

At the locations investigated, the bottom of the varved silty clay layers are located less than about 5 m below the ground surface / top of the fibrous peat. Removal of the organics to depths ranging from about 0.1 m to 0.9 m will be required in this area prior to the construction of the new embankment. Since this work needs to be carried out regardless of the alternative selected, extending the depth of sub-excavation for an additional approximately 4.0 m to 4.5 m (that would be required to remove the soft, compressible layers) is considered practical and will provide the best technical solution in terms of the long-term performance of the roadway.

It should be noted that since the sub-excavation would be required at the toe of the existing MR 80 embankment, some measures must be adopted to protect the existing roadway and maintain the stability of the existing embankment during excavation. Protection of the existing roadway could be accomplished by either installing a temporary support system at the toe of the existing embankment or by performing the required excavation is a series of stages with limited widths. Recommendations for both of these options are given in Section 5.5.1.

In addition, since the groundwater table is located at about the level of the top of the peat layer (i.e. original ground surface), the sub-excavation would likely have to be carried out below the water table. At areas away from the toe of the existing MR 80 embankment, a side slope profile no steeper than 2H:1V would be required to maintain the stability of long, open excavations (i.e. of unlimited width) during dredging (assuming that that water table is maintained at the ground surface). If excavation is carried out in stages with limited widths, a steeper side slope profile could be employed as discussed in Section 5.5.1.2.

Adopting this alternative will also result in increasing the effective thickness of the new embankment by approximately 4.5 m because of the additional fill required below the existing ground surface. The additional below grade fill should be constructed with the same material and same side slope profile as that used for the above grade embankment. The increase in fill height will result in additional post-construction settlement of the embankment fill where rock fill is employed.

5.3.1.2 Pre-Loading

Since the varved silty clay layers encountered in this area are relatively thin (i.e. up to about 3.4 m thick), it is anticipated that the time-dependent settlements will occur relatively quickly. It is estimated that the 95 percent of the post-construction settlements will be completed in about 4.5 months time. If the construction schedule can accommodate this time period, pre-loading the foundation soils by building the embankment as early as possible can be considered. It should be

noted that some minor additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layer should be expected with this option. It is estimated that creep settlements on the order of about 15 mm over each log-cycle of time after the end of primary consolidation will occur. Therefore, following the completion of primary consolidation, about 30 mm of additional creep settlement is expected to occur within about 35 years.

5.3.1.3 Surcharging

Based on the estimated shear strengths for the soil strata in this area (as presented in Section 5.2.1) it is estimated that a 2 m high surcharge could be added on top of the proposed 2 m high embankment (without the need for toe berms) in order to provide more settlement in a shorter period of time. Assuming a coefficient of consolidation (c_v) of $1.1x10^{-2}$ (cm^2/s) and one-way drainage in the approximately 3.4 m thick clayey silt layer, it is estimated that 95 percent of the primary consolidation settlement of the final embankment geometry would be completed in about 2 months time under the influence of a 2 m high surcharge. This would potentially save about 2 months in the construction schedule. It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layers should be expected with this option.

5.3.1.4 Wick Drains

As noted previously, it is estimated that about 95 percent of the primary consolidation of the compressible foundation soils would be completed in about 4.5 months following completion of the embankment construction in this area. However, it is estimated that installing wick drains in a triangular grid at a 1.5 m spacing to a depth of about 5 m would accelerate the consolidation process such that 95 percent of the primary consolidation could be completed in about 3 weeks.

The embankment could be constructed in one stage, however, monitoring of the settlement and dissipation of the excess porewater pressures would be required to check that adequate consolidation had finished prior to proceeding with the final pavement construction stages.

It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layers (on the order of about 15 mm per log-cycle of time) should be expected with this option.

5.3.1.5 Light Weight Fill

In order to reduce the loads imposed by the 2 m high embankment on the soft and compressible foundation soils in this area, the use of light weight fill (i.e. expanded polystyrene fill) could be

considered. The use of this material for the embankment fill widening would result in very little time-dependent (consolidation) settlement of the soft strata. However, the cost of the EPS fill that would required to construct this approximately 2 m high x 5 m wide x 300 m long embankment in this area will be about an order of magnitude higher for this alternative than for some of the other options.

5.3.1.6 Summary

The settlement mitigation alternatives discussed above for Swamp A10 have been summarized, evaluated and ranked on the basis of advantages, disadvantages, relative costs and risks/consequences in Table 2 following the text of this report.

The full sub-excavation alternative is the preferred option for this area. There are a number of disadvantages associated with each of the other alternatives including: minimum time requirements for consolidation of the soft compressible soils that may not fit into the planned construction schedule; the need for wick drain installation; the need for instrumentation installation and on-going monitoring during and after construction; or the high costs of light-weight fill materials.

5.3.2 Swamp A12W – West Side of MR 80 (Station 9+290 to 9+510)

As indicated in Section 5.2.1 and 5.2.2.2, a very soft to stiff varved silty clay strata was encountered in all the boreholes put down along the west side MR 80 widening alignment that crosses through Swamp A12W. The top of the clay strata is located about 0.5 m to 2.1 m below the existing ground surface underlying layers of fibrous peat and silt and/or sand. Where encountered, the clay stratum ranges in thickness from about 1.0 m to 3.4 m.

Between Station 9+290 to 9+400, the presence of the compressible silty clay stratum influences the magnitude of post-construction settlement of the proposed 2.5 m high embankment in this area. Between Station 9+400 to 9+510, the presence of the very soft to stiff, compressible silty clay stratum influences both the magnitude of post-construction settlement and the stability of the up to 6 m high embankments in this area.

In order to achieve the MTO's objective of producing a highway embankment design that will minimize post-construction settlements, the alternatives described in the following sections should be considered.

5.3.2.1 Full Sub-Excavation

At the locations investigated, the bottom of the varved silty clay layers are located less than about 5 m below the ground surface / top of the fibrous peat. Removal of the organics to depths ranging from about 0.5 m to 0.9 m will be required in this area prior to the construction of the new embankment. Since this work needs to be carried out regardless of the alternative selected, extending the depth of sub-excavation for an additional approximately 4.0 m to 4.5 m (that would be required to remove the soft, compressible layers) is considered practical and will provide the best technical solution in terms of the long-term performance of the roadway.

It should be noted that since the sub-excavation would be required at the toe of the existing MR 80 embankment, some measures must be adopted to protect the existing roadway and maintain the stability of the existing embankment during excavation. Protection of the existing roadway could be accomplished by either installing a temporary support system at the toe of the existing embankment or by performing the required excavation is a series of stages with limited widths. Recommendations for both of these options are given in Section 5.5.1.

In addition, since the groundwater table is located at about the level of the top of the peat layer (i.e. original ground surface), the sub-excavation would likely have to be carried out below the water table. At areas away from the toe of the existing MR 80 embankment, a side slope profile no steeper than 2H:1V would be required to maintain the stability of long, open excavations (i.e. of unlimited width) during dredging (assuming that that water table is maintained at the ground surface). If excavation is carried out in stages with limited widths, a steeper side slope profile could be employed as discussed in Section 5.5.1.2.

Adopting this alternative will also result in increasing the effective thickness of the new embankment by approximately 5 m because of the additional fill required below the existing ground surface. The additional below grade fill should be constructed with the same material and side slope profile as that used for the above grade embankment. The increase in fill height will result in additional post-construction settlement of the embankment fill where rock fill is employed.

5.3.2.2 Pre-Loading and Toe Berms

Since the varved silty clay layers encountered in this area are relatively thin (i.e. up to about 3.4 m thick), it is anticipated that the time-dependent settlements will occur relatively quickly. It is estimated that the 95 percent of the post-construction settlements will be completed in about 6 months time. If the construction schedule can accommodate this, pre-loading the foundation

soils by building the embankment as early as possible can be considered. For this alternative, sub-excavation and the temporary support of the existing roadway would not be required.

However, to maintain stability of the up to 6 m high new embankments between Station 9+400 to 9+510, toe berms approximately 6 m wide by 2 m high (between Station 9+400 to 9+450) and approximately 3 m wide by 2 m high (between Station 9+450 to 9+510) will have to be constructed.

It should be noted that some minor additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layer should be expected with this option. It is estimated that creep settlements on the order of about 15 mm over each log-cycle of time after the end of primary consolidation will occur. Therefore, following the completion of primary consolidation, about 30 mm of additional creep settlement is expected to occur within about 40 years.

5.3.2.3 Wick Drains and Staged Construction

As noted previously, it is estimated that about 95 percent of the primary consolidation of the compressible foundation soils would be completed in about 6 months following completion of the embankment construction in this area. However, it is estimated that installing wick drains in a triangular grid at a 1.5 m spacing to a depth of about 5 m would accelerate the consolidation process such that 95 percent of the primary consolidation could be completed in about 3 weeks.

Between Station 9+290 to 9+400, the approximately 2.5 m high embankment could be constructed in one stage, however, monitoring of the settlement and dissipation of the excess porewater pressures would be required to check that adequate consolidation had finished prior to proceeding with the final pavement construction stages. Between Station 9+400 to 9+510, the embankment could be constructed in 2 stages, without the need for toe berms (or temporary embankment support) so long as each fill stage (approximately 2.5 m high) was allowed to consolidate for a minimum period of 3 weeks. However, monitoring of the settlement and dissipation of the excess porewater pressures would be required to check that adequate consolidation had occurred prior to proceeding with the subsequent construction stages.

It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layers (on the order of about 15 mm per log-cycle of time) should be expected with this option.

5.3.2.4 Surcharging

Based on the estimated shear strengths for the soil strata in this area (as presented in Section 5.2.1) it is estimated that a 2 m high surcharge could be added on top of the proposed embankment widenings in order to provide more settlement in a shorter period of time.

However, as noted above, between Station 9+400 to 9+510, toe berms are necessary in order to build the embankment up to the required final grade heights of up to about 6 m. Much larger toe berms would be required if a surcharge was to be placed on top of the required up to 6 m high embankment widening. For example, between Station 9+400 to 9+450 it is estimated that the length of the toe berms would have to be increased by about 5 m (i.e. total length of 11 m) in order to add a 2 m surcharge to the top of the embankment and maintain a FoS of 1.3. Between Station 9+290 to 9+400, it is estimated that a toe berm approximately 6 m wide by 2 m high would be required to be added in order support the additional 2 m high surcharge and maintain a FoS of 1.3. Between Station 9+450 to 9+510, it is estimated that the length of the toe berms would have to be increased by about 6 m (i.e. total length of 9 m) in order to add a 2 m surcharge to the top of the embankment and maintain a FoS of 1.3.

Assuming a coefficient of consolidation (c_v) of 2.0×10^{-3} (cm^2/s) and two-way drainage in the approximately 3.4 m thick varved silty clay layer, it is estimated that 95 percent of the primary consolidation settlement of the final embankment geometry would be completed in about 3 months time under the influence of a 2 m high surcharge. This would potentially save about 3 months in the construction schedule. It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layers should be expected with this option.

5.3.2.5 Light Weight Fill

In order to reduce the loads imposed by the 2.5 m to 6 m high embankments on the soft and compressible foundation soils in this area, the use of light weight fill (i.e. expanded polystyrene fill) could be considered. The use of this material for the embankment fill widening would result in very little time-dependent (consolidation) settlement of the soft strata. However, the cost of the EPS fill that would required to construct this approximately 4 m high (average) x 10 m wide x 200 m long embankment in this area will be about an order of magnitude higher for this alternative than for some of the other options.

5.3.2.6 Summary

The settlement and stability mitigation alternatives discussed above for Swamp A12W have been summarized, evaluated and ranked on the basis of advantages, disadvantages, relative costs and risks/consequences in Table 3 following the text of this report.

The full sub-excavation alternative is the preferred option for this area. There are a number of disadvantages associated with each of the other alternatives including: minimum time requirements for consolidation of the soft compressible soils that may not fit into the planned construction schedule; the need for additional fill materials for stability berm construction; the need for wick drain installation; the need for instrumentation installation and on-going monitoring during and after construction; or the high costs of light-weight fill materials.

5.3.3 Swamp A13 – Detour #2 (Station 16+430 to 16+500)

As noted in Section 5.2.1.3, a very soft to firm, varved silty clay layer exists at depth in this swamp area. The presence of this soft layer influences both the stability and magnitude of post-construction settlement of the proposed 8 m high embankment widening in this area.

In order to achieve the MTO's objective of producing a highway embankment design that will minimize post-construction settlements, the alternatives described in the following sections should be considered.

5.3.3.1 Full Sub-excavation

The bottom of the varved silty clay layer is located approximately 3.8 m below the top of the fibrous peat in this area. Removal of the surficial organics to a depth of about 0.5 m will be required prior to the construction of the new embankment. Since this work needs to be carried out regardless of the alternative selected, extending the depth of sub-excavation for an additional approximately 3.3 m (that would be required to remove all of the soft, compressible layers) is considered practical, will obviate the need for large toe berms and will provide the best technical solution in terms of the stability and long-term performance of the roadway.

It should be noted that a temporary support system will be required at the toe of the existing Highway 17 embankment to maintain its stability during the sub-excavation. Considering the height of the existing embankment and the depth of excavation required, carrying out the sub-excavation in stages with limited widths is not recommended for this area. Recommendation with respect to temporary support are given in Section 5.5.1. In addition, since the groundwater surface is located at about the level of the top of the peat layer (i.e. original ground surface), the

sub-excavation would likely be carried out by dragline below the water table. At areas away from the toe of the existing Highway 17 embankment, a side slope profile no steeper than 1.5H:1V will be required in order to maintain the stability of the excavation during dredging (assuming that that water table is maintained at the ground surface). If dewatering is required for the construction of the temporary support, it is estimated that a side slope profiles as flat as 4H:1V could be required in order to maintain the stability of the excavation (in areas away from the toe of the existing Highway 17 embankment).

Adopting this alternative will also result in increasing the effective thickness of the new embankment by approximately 4 m because of the additional fill required below the existing ground surface. The additional below grade fill should be constructed with the same material and side slope profile as that used for the above grade embankment. The increase in fill height will result in additional post-construction settlement of the embankment fill where rock fill is employed.

5.3.3.2 Staged Construction

Due to the relative thinness of the soft, compressible foundation soil layers in this area, it is estimated that about 95 percent of the primary consolidation of the subsoils would be completed in about 4 weeks following the completion of embankment construction. Considering this, if the construction schedule permits, the embankment could be constructed in 3 stages, without the need for large toe berms (or temporary embankment support) so long as the each fill stage (approximately 2.5 m high) was allowed to consolidate for a minimum period of 4 weeks. However, monitoring of the settlement and dissipation of the excess porewater pressures would be required to check that adequate consolidation had occurred prior to proceeding with the subsequent construction stages.

It should be noted that some small additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layer (less than about 10 mm per log-cycle of time) should be expected with this option.

5.3.3.3 Pre-Loading and Toe Berms

As noted above, since the silty clay layer encountered at depth is relatively thin (ie. only about 1.5 m thick) it is estimated that the 95 percent of the post-construction foundation soil settlements will be completed in about 4 weeks time. If the construction schedule can accommodate this period, pre-loading the foundation soils by building the embankment as early as possible can be considered. For this alternative, sub-excavation and the temporary support would not be required. However, to maintain stability, toe berms approximately 5.5 m wide by 4 m high on the south

side of the approximately 60 m long embankment will have to be constructed. It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layer should be expected with this option. It is estimated that creep settlements of less than about 10 mm over each log-cycle of time after the end of primary consolidation will occur. Therefore, following the completion of primary consolidation, about 25 mm of additional creep settlement is expected to occur within about 50 years.

5.3.3.4 Surcharging

As noted above, toe berms are necessary in order to build the embankment up to the required final grade height of about 8 m. Much larger toe berms would be required if a surcharge was to be placed on top of the required 8 m high embankment widening. It is estimated that the length of the toe berms would have to be increased by about 2.5 m (i.e. total length of 8 m) in order to add a 2 m surcharge to the top of the embankment and maintain a FoS of 1.3. However, it is estimated that 95 percent of the primary consolidation settlement of the final embankment geometry would be completed in about 3 weeks time under the influence of a 2 m high surcharge. This would potentially save about 1 week in the construction schedule. It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the cohesive layer should be expected with this option.

5.3.3.5 Light Weight Fill

In order to reduce the loads imposed by the 8 m high embankment on the soft and compressible foundation soils in this area, the use of light weight (i.e. expanded polystyrene fill) could be considered. The use of this material for the embankment fill would eliminate the need for stabilizing toe berms and would result in very little time-dependent (consolidation) settlement of the soft strata. However, considering the large volume of EPS fill that would required to construct this 8 m high x 5 m wide x 60 m long embankment in this area, the cost will be about an order of magnitude higher for this alternative than for some of the other options.

5.3.3.6 Summary

The settlement and stability mitigation alternatives discussed above for Swamp A13 have been summarized, evaluated and ranked on the basis of advantages, disadvantages, relative costs and risks/consequences in Table 4 following the text of this report.

The full sub-excavation alternative is the preferred option for this area. There are a number of disadvantages associated with each of the other alternatives including: minimum time requirements for consolidation of the soft compressible soils that may not fit into the planned

construction schedule; the need for additional fill materials for stability berm construction; the need for instrumentation installation and on-going monitoring during and after construction; or the high costs of light-weight fill materials

5.4 Subgrade Preparation and Embankment Construction

As discussed in Section 5.1, the construction of the detours for the Highway 17 / Municipal Road 80 Interchange and the MR 80 widening will require the construction of numerous high fill embankments and embankments over swamps. Prior to the placement of any fill for the new embankment construction, all surface and near surface layers of topsoil and organic deposits should be stripped from the plan limits of the proposed works. Table 1 summarizes the existing embankment fill types, the recommended fill type to be placed for the widenings, the location and depth of organics, the recommended side slope profiles, the requirements for side berms, the anticipated differential settlements and the recommended method of removal of organics. The following sections provide details on the recommendations for subgrade preparation and embankment construction.

5.4.1 Removal of Organics

Based on the information from the borings and test pits obtained during the field investigation, organic deposits (i.e. fibrous peat) of up to about 0.9 m deep can be expected in some areas.

In areas where new fill embankments will be constructed immediately adjacent to, or on top of, the existing Municipal Road 80 or Highway 17 embankments (i.e. at widenings or where the detour alignment tie in to the existing roadways), construction procedures should implement the guidelines of OPSD 203.020. These guidelines require that the slopes of the existing embankment be temporarily excavated to a 1H:1V profile to allow for removal of a larger extent of organic material. However, in all cases, during excavation, measures must be adopted to ensure that the existing roadways are protected and that the stability of the existing embankments are maintained. A provision for traffic control measures should also be included in the Contract to maintain the safe operation of MR 80 during any adjacent excavation works.

In areas where new fill embankments are being constructed away from existing roadways, construction procedures should implement the guidelines of OPSD 203.010.

5.4.2 Embankment Fill Placement

Where existing embankments are composed of sand and gravel fill (i.e. on MR 80 between about Station 9+000 to 9+300), benching into the existing side slopes should be carried out as per

OPSD 208.010 during the construction of the tie-ins of the new embankments. Where existing embankments are composed of rock fill (such as those along Highway 17 and along MR 80 between about Station 9+300 to 9+500 and 9+850 to 9+950), any loose or deleterious material should be removed from the toe and slopes prior to any new fill placement.

Where earth fill (granular) is used for the construction of the new embankments on MR 80 between about Station 9+000 to 9+300, placement of all granular fill material should be carried out in accordance with OPSS 206.07.07, in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the Standard Proctor maximum dry density. The final lift prior to placement of the granular sub-base or base course should be placed and compacted to current MTO requirements for pavements. Inspection and field density testing should be carried out by qualified geotechnical personnel during all earth fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. If local supplies of earth fill such as Select Subgrade Material are difficult to obtain, consideration could be given to using a Granular 'B' Type II material for embankment construction in this area.

For MR 80 widening beyond about Station 9+300, rock fill is recommended for embankment construction in order to maintain a similar fill type with the existing roadway. At the transition between the earth fill (or Granular 'B' Type II) and rock fill at about Station 9+300, a provision must be made in the Contract to have a graded granular separator between the dissimilar fills to avoid loss of fines from the earth fill into the rock fill

Where rock fill is used for the construction of the new embankments, placement of all rock fill material should be carried out in accordance with the requirements as outlined in the Special Provision, Amendment to OPSS 206 dated September 1999. The rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging shall be minimized by blading, dozing and 'chinking' the rock to form a dense, compact mass.

Assuming the use of rock fill for the embankment widening, final side slopes should be no steeper than 1.25H:1V. Where earth fill is employed, final side slopes should be no steeper than 2H:1V.

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

5.5 Excavations and Groundwater Control

As noted in Section 5.4, excavation within the plan limits of the proposed works will be required in order to remove topsoil and organic deposits and, in some area, soft compressible strata prior to embankment fill placement. Groundwater flow into the excavations can be expected to occur due to the high groundwater levels observed at the sites and the presence of some relatively permeable strata at most areas. Where the purpose of the excavations is only to remove shallow organic deposits, the presence of water in the excavations should not impact the stability. Where the excavations are required to be deeper in order to remove soft compressible strata, the recommendations on side slope profiles noted in Section 5.3 and Section 5.5.1.2 should be followed. Where deep excavations are required at the toe of existing roadway embankments, measures that will protect the stability and performance of the existing roadway must be implemented. In addition, a provision for traffic control measures should be included in the Contract to maintain the safe operation of MR 80 during excavation operations.

Conventional excavation equipment should be suitable for the majority of excavation through the on-site soils. However, at locations where the new embankments tie in to the existing roadway rock fill embankments, it should be noted that boulders could be encountered at or just below the ground surface. The presence of the boulders may interfere with or slow the progress of stripping and excavation at these locations.

Where cutting is required through the rock outcrops (as may be necessary for Detour #1 between about 9+510 to 9+600), rock excavation and blasting will be required to accommodate the road widening and new alignment. All rock excavation and grading should be carried out in accordance with the requirements as outlined in OPSS 206. Rock faces can be graded to vertical or near vertical as shown on OPSD – 201.010. Guidelines for the required geometry and blasting of rock outcrops are provided in Section 5.6.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects.

5.5.1 Protection of Existing Roadways

In areas where the new alignments for the detours and/or MR 80 widening will be immediately adjacent to the existing roadway embankments, and where the full sub-excavation is chosen to mitigate the settlement and stability problems, some measures must be adopted to protect and maintain the stability of the existing roadway embankments.

Protection of the existing roadway could be accomplished by either installing a temporary support system at the toe of the existing embankment or by performing the required excavation is a series of stages with limited widths. Recommendations for both of these options are given below.

5.5.1.1 Temporary Shoring

The support system would be located at the toe of a 1H:1V slope projected downwards from the crest of the existing highway embankment slope. It is recommend that the temporary excavation support be in accordance with MTO Special Provision 539S01. The temporary support system should be designed to Performance Level 2 as defined in SP 539S01. Roadway protection should be as per current MTO Special Provision 539S01. All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects.

5.5.1.2 Staged Excavation

Recommendations for staged excavation are as follows :

- work can be carried out simultaneously from both ends of the swamp working towards the centre;

- removal of the peat/organics and varved silty clay deposits for the entire section of the widening should be carried out in short sections perpendicular to the highway alignment with the base of the excavation/trench not wider than 3m at any time;

- in areas adjacent to the toe of the existing MR 80 roadway embankment, the excavation should be carried out such that the base of the excavation is maintained outside a zone defined by a line drawn downward at 1 horizontal : 1 vertical (1H:1V) from the crest of the existing roadway embankment to the base of the excavation;

- in areas away from the toe of the existing MR 80 roadway embankment, the following profiles are recommended for temporary excavation slopes (i.e. back slopes) where z = depth of excavation:
  - $0\ m < z < 4\ m$ – 0.5H:1V
  - $4\ m < z < 5\ m$ – 1H:1V

- excavation and backfilling operations should be carried out simultaneously in a manner that the excavation is not left open for more than 3 m in length at any given time;

- since some distress to the existing roadway may occur during the staged excavation, provision for traffic control measures must be included in the Contract to maintain the safe operation of MR 80 during the excavation and backfilling operations.

It should be noted that following the above procedures will still result in some of the soft clayey deposits remaining in place below the transition area between the existing and widened embankments. Consideration should be given to preloading and surcharging this transition area in order to consolidate as much of the remaining clayey layers as possible.

Depending on the length of time available for preloading and/or surcharging, some remaining differential settlement between the original and new embankment widening could occur within this transition area. This could require on-going maintenance to raise the road grade in some areas.

5.6 Blasting Recommendations for Rock Cuts

5.6.1 Stability Considerations

Based on the exposed conditions observed along the existing rock cuts and rock outcrops around the site, the following general comments can be made on the anticipated stability conditions of any new rock cuts required along MR 80.

- careful rock excavation will be required (i.e. controlled blasting) to minimize damage to the final rock faces and minimize the potential for raveling. Any unnecessary fracturing of the final rock faces will exacerbate ice jacking related to freeze-thaw cycles causing blocks to loosen and eventually detach.
- based on the joint data collected at the various rock cuts and outcrops there is a potential for some sliding wedges, overhanging blocks and steep (toppling type) slabs to develop which could result in some rock mass instabilities in the new cut faces. The extent of these types of failure mechanisms which might develop on the final faces will only be known during and after excavation. As such it will be important that the final faces be inspected for unstable rock and appropriate measures taken (i.e. mechanical scaling or rock bolting) as required.
- adequate catchment ditches should be provided at the toe of each of the rock cuts. These catchment ditches should also be regularly inspected and cleaned when significant debris has accumulated.

5.6.2 Excavation Considerations

For permanent cut slopes through the bedrock, the overall slope to the cut face may be formed vertical to near vertical (i.e. 0.25 horizontal to 1 vertical). The use of carefully controlled

excavation techniques will be required in order to ensure a neat excavation line and minimize face instabilities and long-term maintenance problems resulting from damage to the rock mass.

Excavation will generally require controlled blasting, although for some of the smaller cuts, mechanical excavation could be considered. Where rock cuts are of sufficient height that the proposed clear zone may not be adequate to contain rock fall debris, additional remedial measures (i.e. rock scaling or rock bolting) in order to eliminate any new rock hazards which might be exposed in the newly constructed rock face, may be required. The actual condition of the final faces will only be known after excavation and any scaling of loose rock or other remedial work will need to be finalized at that time.

5.6.3 Special Provisions

5.6.3.1 Blasting

The use of controlled blasting techniques is recommended for all of the bedrock excavation for the final rock faces. It is recommended that a separate Special Provision for the control of all blasting operations be prepared (refer to SP 299F06). The Special Provision should include, but not be limited to, the following:

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

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

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

5.6.3.2 Rock Scaling

Inspection of all new rock cut faces by qualified geotechnical personnel immediately after blasting should be carried out in order to assess where scaling / loosened rock removal should be carried out. All loose, unstable rock should be removed from the cut faces before access to the toe area of the slope is permitted. The appropriate Special Provisions (refer to SP229F03) for rock scaling and for use of a Quality Verification Engineer should be included in the Contract Documents.

5.6.3.3 Rock Bolting

Where potentially unstable rock blocks or wedges cannot be removed safely or where the removal of such blocks/wedges could undermine the rock mass above, rock bolting may be required. As such, a Special Provision for rock bolting should be included in the contract documents (refer to SP229S07). The extent of any rock bolting, if required, will only be known following an inspection of the final rock faces.

GOLDER ASSOCIATES LTD.

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LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. General

π	3.1416
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
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

2 Shear strength = (Compressive strength)/2

Golder Associates

RECORD OF BOREHOLE No BH03-151 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5144547.5 ;E 304390.5 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | DATE Mar. 21, 2003

ORIGINATED BY EHS | COMPILED BY DKB | CHECKED BY JPD

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
268.8	ICE SURFACE										
0.0	ICE										
268.3	Fibrous peat										
0.5	Black										
	Silt, some clay, trace sand, occ. fine to medium sand seams. (Alluvium)										
	Very loose										
267.6	Light brown and grey		1	SS	4		268				
1.2	Moist										
	Varved Silty Clay to Clayey Silt, trace fine sand, occ. fine sand seams										
	Soft to firm		2	SS	4		267				
	Grey and brown to dark grey below 3.4m depth										
	Wet										
			3	SS	3		266				
							265	+ 2.6, + 6.9			
	Occ. cobbles at 4.4m depth										
264.2											
4.6	END OF BOREHOLE Auger Refusal										

Notes:
1. Upper 0.4m of soil frozen during drilling operation.
2. Water level in open borehole measured at 3.0m depth (El. 265.8m) upon completion of drilling.

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 17/10/03

Golder Associates

RECORD OF BOREHOLE No BH03-152 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5144468.6 ;E 304382.9 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | DATE Mar. 21, 2003

ORIGINATED BY EHS | COMPILED BY DKB | CHECKED BY JPD

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
268.7	ICE SURFACE									
0.0	ICE									
268.2	Fibrous peat Dark brown									
0.5	Silt, some clay, trace fine sand, occ. fine sand seams (Alluvium) Loose Brown & grey Wet		1	SS	4		268			
267.3										
1.4	Varved Silty Clay to Clayey Silt, trace fine sand, occ. silt seams Firm to very stiff Grey and reddish brown Wet		2	SS	5		267			
						▽	266	+ 100		Field vane >100 kPa
								+ 100		Field vane >100 kPa
265.4			3	SS	55/0.23					
3.3	END OF BOREHOLE Spoon and Auger Refusal									

Notes:
1. Upper 0.4m of soil frozen during drilling operation.
2. Water level in open borehole measured at 2.6m depth (El. 266.1m) upon completion of drilling.

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 17/10/03

Golder Associates

RECORD OF BOREHOLE No BH03-153 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5144388.3 ;E 304366.7 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | DATE Mar. 21, 2003

ORIGINATED BY EHS | COMPILED BY DKB | CHECKED BY JPD

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
269.2	GROUND SURFACE										
0.0	Fibrous peat Dark brown						269				
268.3											
0.9	Silt, some clay, trace fine sand, trace gravel (Alluvium) Loose Light brown & grey mottled, oxidized Moist to wet		1	SS	4		268				
267.4			2	SS	7						
267.1	Varved Silty Clay, trace fine silt Grey & reddish brown										
2.1	Wet Silt, some clay, trace fine sand, occ. silt layers Compact Grey and dark grey, layered Wet		3	SS	10	▽	267				
265.9			4	SS	18		266				
	Sandy Silt, some gravel (Till)										
3.5	END OF BOREHOLE Spoon and Auger Refusal										

Notes:
1. Upper 0.9m of soil frozen during drilling operation.
2. Water level in open borehole measured at 2.7m depth (El. 266.6m) upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No BH03-154 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | LOCATION N 5144331.5 ;E 304356.9 | ORIGINATED BY EHS

DIST 54 | HWY 17 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY DKB

DATUM GEODETIC | DATE Mar. 21, 2003 | CHECKED BY JPD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
269.3	GROUND SURFACE										
0.0 / 0.1	Topsoil Dark brown Silt, some clay, trace fine sand, trace gravel, trace organics, occ. cobbles in upper 0.7m of deposit Very loose Light brown & grey, oxidized Moist		1	SS	3		269				
267.9							268				
1.4	Silt, some clay, trace fine sand, occ. fine sand seams (Alluvium) Loose to compact Light brown & grey, oxidized, layered Moist to wet		2	SS	8						
267.0											
2.3	END OF BOREHOLE Auger Refusal Note: 1. Open borehole dry upon completion of drilling.										

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

Golder Associates

RECORD OF BOREHOLE No BH03-173 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5143750.3 ;E 304149.1 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | DATE Jul. 28, 2003

ORIGINATED BY EHS | COMPILED BY JDR | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
279.1	GROUND SURFACE						279				
0.0	Fibrous Peat										
0.2	Clayey Silt, trace sand, trace organics Firm to stiff Brown Wet		1	SS	5				○		
277.7			2	SS	15	▽	278		○		
1.4	Silt, trace to some sand, trace gravel Very dense to dense Light brown to grey Moist to wet		3	SS	55				○		
276.1			4	SS	42		277				
3.1	Silty fine Sand, trace gravel , occasional cobbles Dense Light brown Wet		5	SS	47		276		○		
275.4											
275.1	Sand, some gravel, trace silt Compact Light brown Wet		6	SS	62/0.1				○		
4.0	END OF BOREHOLE Spoon and Auger Refusal										

Note:
1. Water level in open borehole measured at 1.2m depth (El. 277.9) upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No BH03-174 1 OF 1 METRIC

PROJECT 021-1147 | LOCATION N 5143703.8 ;E 304130.3 | ORIGINATED BY EHS

W.P. GWP 99-98-00 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY JDR

DIST 54 HWY 17 | DATUM GEODETIC | DATE Jul. 28, 2003 | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: ○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
278.9	GROUND SURFACE										
0.0	Fibrous Peat										
0.2	Clayey Silt, some clay, some fine sand, trace gravel, trace organics, occasional sand seams Soft to firm Light brown, oxidized Wet		1	SS	3						
			2	SS	8	▽	278		○		2 15 68 15
277.4											
1.5	Silt, some fine sand, trace organics, occasional fine sand seams Compact Light brown, oxidized Wet		3	SS	21		277				
			4	SS	13		276				
275.6											
3.4	Sand, some silt, trace gravel Compact to loose Light brown, oxidized Wet		5	SS	26				○		
			6	SS	7		275				
274.3											
4.6	END OF BOREHOLE Spoon and Auger Refusal Note: 1. Water level in open borehole measured at 1.3m depth (277.6 m) upon completion of drilling.		7	SS	50/0						

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

Golder Associates

RECORD OF BOREHOLE No BH03-175 1 OF 1 METRIC

PROJECT 021-1147 | LOCATION N 5143385.9 ;E 304477.2 | ORIGINATED BY EHS

W.P. GWP 99-98-00 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY JDR

DIST 54 HWY 17 | DATUM GEODETIC | DATE Jul. 28, 2003 | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
278.0	GROUND SURFACE										
0.0 / 0.1	Fibrous Peat Dark brown Silt, some fine sand, trace organics Very loose		1	SS	2				o		
277.2 / 0.8	Light brown, oxidized Wet Sand, some silt, trace clay, occasional silt interlayers Dense to compact Light brown to grey Wet		2	SS	21		277				
			3	SS	45	▽	276		o		0 84 14 2
			4	SS	18		275				
			5	SS	20						
274.0 / 4.0	Sand, trace to some silt, trace to some gravel, occasional silt interlayers Loose to compact Wet		6	SS	9		274		o		
			7	SS	12		273				
272.4			8	SS	53/0.1				o		
5.6	END OF BOREHOLE Spoon and Auger Refusal Note: 1. Water level in open borehole measured at 2.1m depth (275.9 m) upon completion of drilling.										

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

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Golder Associates

RECORD OF BOREHOLE No BH03-176 1 OF 1 METRIC

PROJECT 021-1147 | LOCATION N 5143316.3 ;E 304692.5 | ORIGINATED BY EHS

W.P. GWP 99-98-00 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY JDR

DIST 54 HWY 17 | DATUM GEODETIC | DATE Jul. 29, 2003 | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (20 40 60 80 100): O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
276.9	GROUND SURFACE					▽					
0.0	Fibrous Peat										
0.2	Dark brown Silty fine Sand, trace gravel and cobbles Compact Light brown Wet		1	SS	16				o		
275.8			2	SS	25		276		o		
1.2	Fine to medium grained Sand, some gravel, trace silt Compact Light brown Wet										
	END OF BOREHOLE Auger Refusal										

Note:
1. Water level in open borehole measured at surface upon completion of drilling.

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 17/10/03

Golder Associates

RECORD OF BOREHOLE No BH03-177 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5143302.3 ;E 304714.1 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | DATE Jul. 30, 2003

ORIGINATED BY EHS | COMPILED BY JDR | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS
276.9	GROUND SURFACE					
0.0 / 276.6	Water					
276.3	Fibrous Peat, Dark brown					
0.6	Sand, trace to some silt, trace clay, trace gravel, occasional silty clay interlayers, Compact, Grey, oxidized to 1.0m depth, Wet		1	SS	25	
			2	SS	22	
274.8						
2.1	Varved Silty Clay, trace sand, occasional sand seams, Very soft to firm, Grey and reddish brown, Wet		3	SS	1	
273.4						
3.5	Silt, some sand, trace to some clay, occasional clayey silt interlayers, Very loose, Grey, Wet		4	SS	2	
272.2						
4.7	Silty Sand, trace clay, Compact, Grey, Wet		5	SS	12	
			6	SS	14	
270.2						
6.7	END OF BOREHOLE, Dynamic Cone Refusal (41/0.3, 50/0.0)					

Note:
1. Water level in open borehole measured at surface upon completion of drilling.

ELEVATION SCALE: 276, 275, 274, 273, 272, 271

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

+6.7, +4.8, +4.0

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

48, 52.6

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 20/10/03

Golder Associates

RECORD OF BOREHOLE No BH03-178 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | LOCATION N 5143289.3 ;E 304739.4 | ORIGINATED BY EHS

DIST 54 HWY 17 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY JDR

DATUM GEODETIC | DATE Jul. 30, 2003 | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
276.9	GROUND SURFACE										
0.0	Fibrous Peat										
0.2	Dark brown Sand, trace to some silt, trace gravel, trace clay, contains cobbles, occasional silty clay interlayers Compact to very dense Light brown and oxidized Wet		1	SS	14						
			2	SS	113		276		o		
			3	SS	25		275		46		
274.6									48.7 o		
2.3	Varved Silty Clay, trace fine sand, occasional sand seams Firm to very soft Grey and reddish brown Wet		4	SS	4		274	+ 2.4 + 2.0			
273.1											
3.8	Sand and Silt, trace clay, trace gravel, contains cobbles Very dense Grey Wet		5	SS	99		273		o		
272.0			6	SS	102		272				
5.0	END OF BOREHOLE Auger Refusal										

Note:
1. Water level in open borehole measured at surface upon completion of drilling
2. Sample No. 2 encountered a cobble during sampling at 1.6m depth
3. Sample No. 6 contained angular rock fragments in tip of split spoon sampler.

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

Golder Associates

RECORD OF BOREHOLE No BH03-179 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5144275.8 ;E 304310.4 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | DATE Jul. 31, 2003

ORIGINATED BY EHS | COMPILED BY JDR | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (20 40 60 80 100): O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
269.2	GROUND SURFACE										
0.0	Water										
	Fibrous Peat Dark brown		1	SS	WH		269				
0.5	Silt, some clay, trace fine sand Loose Light brown and grey, oxidized Wet		2	SS	9						
	Varved Silty Clay, trace fine sand, trace organics Firm to stiff Grey and reddish brown to 1.8m, grey and light grey to 2.4m						268	+ 18.0			
								+ 8.5			
266.8							267				
2.5	Silty Sand, trace organics Grey END OF BOREHOLE Spoon and Auger Refusal		3	SS	61/0.1						

Note:
1. Water level in open borehole measured at surface upon completion of drilling.

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 17/10/03

Golder Associates

RECORD OF BOREHOLE No BH03-180 1 OF 1 METRIC

PROJECT 021-1147 | LOCATION N 5144228.1 ;E 304296.7 | ORIGINATED BY EHS

W.P. GWP 99-98-00 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY JDR

DIST 54 HWY 17 | DATUM GEODETIC | DATE Aug. 01, 2003 | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
269.6	GROUND SURFACE										
0.0	Water										
269.1	Fibrous Peat Dark brown		1	SS	3				o		
0.5	Silty Clay with silt and sand seams, trace sand, trace organics Stiff Grey and light brown Wet		2	SS	24		269			o	
268.2											
1.5	END OF BOREHOLE Auger Refusal Note: 1. Water level in open borehole measured at surface upon completion of drilling.										

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

Golder Associates

RECORD OF BOREHOLE No BH03-181 1 OF 1 METRIC

PROJECT 021-1147 | LOCATION N 5144180.2 ;E 304282.3 | ORIGINATED BY EHS

W.P. GWP 99-98-00 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY JDR

DIST 54 HWY 17 | DATUM GEODETIC | DATE Aug. 01, 2003 | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS
269.7	GROUND SURFACE					
0.0 / 269.4	Water					
0.3 / 268.8	Fibrous Peat Dark brown		1	SS	WH	
268.5	Silt, some clay, trace to some sand, trace gravel Very loose to loose Grey and light brown Wet		2	SS	4	
1.2	Varved Silty Clay to Clay, trace sand, contains cobbles at 2.9m depth Firm to soft Reddish brown and grey Wet		3	SS	7	
267.0						
2.7	Varved Silty Clay to Clay, some silt Very soft to soft Grey Wet		4	SS	1	
265.1						
4.6	Silt with sand, some clay, occasional silty clay interlayers Very loose Grey Wet		5	SS	WH	
264.4						
5.3	END OF BOREHOLE Auger Refusal Note: 1. Water level in open borehole measured at surface upon completion of drilling.					

ELEVATION SCALE: 269, 268, 267, 266, 265

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: 20 40 60 80 100
- O UNCONFINED
- + FIELD VANE
- ● QUICK TRIAXIAL
- × REMOULDED

+8.0, +6.7, +1.8, +3.0

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

59.2, 57, 45.6, 57, 56.2

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

+3, ×3: Numbers refer to Sensitivity

O3%: STRAIN AT FAILURE

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 17/10/03

Golder Associates

RECORD OF BOREHOLE No BH03-182 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5144155.1 ;E 304272.1 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | DATE Aug. 01, 2003

ORIGINATED BY EHS | COMPILED BY JDR | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
269.7	GROUND SURFACE										
0.0 / 269.4	Water		1	SS	1		269				
0.5	Fibrous Peat Dark brown Silty Sand, trace to some clay, trace gravel, occasional clay seams Compact		2	SS	25						4 56 28 12
268.3 / 1.4	Light brown Very wet Silt, trace to some sand, trace clay, occasional sand interlayers Loose		3	SS	9		268				
267.6 / 2.1	Light brown Wet Clayey Silt, trace to some sand, frequent sand seams Firm		4	SS	7		267			41.2	
266.8 / 2.9	Grey Wet Varved Silty Clay, trace sand Very soft to stiff Reddish brown and grey Wet		5	SS	WH		266	6.5 6.3		56.4	
264.7							265	8.0			
5.0	Silt, some sand, trace clay Compact Grey		6	SS	11		264				
264.0	Wet										
5.9	Sandy Silt, trace gravel, trace clay Wet END OF BOREHOLE Auger Refusal										

Note:
1. Water level in open borehole measured at surface upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No BH03-183 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5144133.6 ;E 304263.7 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | DATE Aug. 01, 2003

ORIGINATED BY EHS | COMPILED BY JDR | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
269.8	GROUND SURFACE										
0.0 / 269.3	Fibrous Peat, Dark brown		1	SS	4	▽			58.6		
0.5 / 268.7	Sand, some silt, trace clay, trace gravel, Loose to compact, Light brown, Wet		2	SS	22		269		o		
1.1 / 267.7	Varved Silty Clay, trace sand, frequent sand seams, Stiff to very stiff, Reddish brown and grey, Wet		3	SS	9		268				
2.1 / 267.3	Sandy Silt and Sand, Compact, Grey, oxidized, Wet		4	SS	64/0.1				o		
2.5	END OF BOREHOLE, Spoon and Auger Refusal										

Note:
1. Water level in open borehole measured at 0.2m depth (268.6 m) upon completion of drilling.

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 17/10/03

Golder Associates

RECORD OF BOREHOLE No BH03-184 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | LOCATION N 5144173.0 ;E 304306.9 | ORIGINATED BY EHS

DIST 54 HWY 17 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | COMPILED BY JDR

DATUM GEODETIC | DATE Aug. 01, 2003 | CHECKED BY CMG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
269.9	GROUND SURFACE										
0.0 / 269.4	Fibrous Peat Dark brown		1	SS	1						
0.5 / 268.8	Silt, some clay, trace sand, trace organics Compact Light brown and grey Wet		2	SS	19		269				
1.1 / 267.7	Varved Clayey Silt, occasional silt interlayers, trace sand Compact Grey and light brown Wet		3	SS	18		268				
2.2	END OF BOREHOLE Auger Refusal Note: 1. Water level in open borehole measured at surface upon completion of drilling.										

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 17/10/03

Golder Associates

RECORD OF BOREHOLE No BH03-185 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5144221.8 ;E 304318.3 | BOREHOLE TYPE POWER AUGERING WITH 108mm I.D. HOLLOW STEM AUGERS | DATE Aug. 01, 2003

ORIGINATED BY EHS | COMPILED BY JDR | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
270.0	GROUND SURFACE										
0.0 / 269.7	Fibrous Peat Dark brown		1	SS	4				o		
269.3	Clayey Silt, trace to some organics, trace sand, cobbles, wood fragments, boulders and rock fragments Very loose to loose Grey Wet										
0.7	END OF BOREHOLE Auger Refusal										

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 17/10/03

Golder Associates

RECORD OF PENETRATION TEST No DC03-54 1 OF 1 METRIC

PROJECT 021-1147
W.P. GWP 99-98-00
DIST 54 HWY 17
DATUM GEODETIC

LOCATION N 5143773.8 ;E 304158.0
BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST
DATE Jul. 28, 2003

ORIGINATED BY EHS
COMPILED BY JDR
CHECKED BY CMG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
279.7	GROUND SURFACE										
0.0							279 278 277				
276.7											
3.1	End of Penetration Test Refusal at 3.1m							58 Blows for last 300mm Refusal - 50 Blows / 0 mm			

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

Golder Associates

RECORD OF PENETRATION TEST No DC03-55 1 OF 1 METRIC

PROJECT 021-1147
W.P. GWP 99-98-00 LOCATION N 5143727.1 ;E 304139.6 ORIGINATED BY EHS
DIST 54 HWY 17 BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST COMPILED BY JDR
DATUM GEODETIC DATE Jul. 28, 2003 CHECKED BY CMG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
278.9	GROUND SURFACE										
0.0							278				
							277				
276.5											
2.4	End of Penetration Test Refusal at 2.4m							50 Blows for last 280mm Refusal - 50 Blows / 0 mm			

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 7/10/03

Golder Associates

RECORD OF PENETRATION TEST No DC03-56 1 OF 1 METRIC

PROJECT 021-1147
W.P. GWP 99-98-00
DIST 54 HWY 17
DATUM GEODETIC
LOCATION N 5144299.8 ;E 304317.1
BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST
DATE Jul. 31, 2003
ORIGINATED BY EHS
COMPILED BY JDR
CHECKED BY CMG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
269.3	GROUND SURFACE										
0.0							269				
							268				
							267				
							266				
							265				
							264				
							263				
							262				
261.3											
8.0	End of Penetration Test Refusal at 8.0m							30 Blows for last 100mm Refusal - 50 Blows / 0 mm			

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

Golder Associates

RECORD OF PENETRATION TEST No DC03-57 1 OF 1 METRIC

PROJECT 021-1147
W.P. GWP 99-98-00 LOCATION N 5144252.1 ;E 304303.1 ORIGINATED BY EHS
DIST 54 HWY 17 BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST COMPILED BY JDR
DATUM GEODETIC DATE Aug. 01, 2003 CHECKED BY CMG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
269.5	GROUND SURFACE										
0.0							269				
268.6											
0.9	End of Penetration Test Refusal at 0.9m							20 Blows for last 300mm Refusal - 50 Blows / 0 mm			

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

Golder Associates

RECORD OF PENETRATION TEST No DC03-58 1 OF 1 METRIC

PROJECT 021-1147
W.P. GWP 99-98-00
DIST 54 HWY 17
DATUM GEODETIC
LOCATION N 5144204.5 ;E 304288.5
BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST
DATE Aug. 01, 2003
ORIGINATED BY EHS
COMPILED BY JDR
CHECKED BY CMG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
269.7	GROUND SURFACE										
0.0							269				
							268				
							267				
266.0							266				
3.7	End of Penetration Test Refusal at 3.7m							13 Blows for last 75mm Refusal - 50 Blows / 0 mm			

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

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

MISS_MTO 021-1147 BH.GPJ ON_MOT.GDT 7/10/03

Golder Associates

RECORD OF PENETRATION TEST No DC03-59 1 OF 1 METRIC

PROJECT 021-1147 | W.P. GWP 99-98-00 | DIST 54 HWY 17 | DATUM GEODETIC

LOCATION N 5144157.0 ;E 304272.8 | BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST | DATE Aug. 01, 2003

ORIGINATED BY EHS | COMPILED BY JDR | CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
269.7	GROUND SURFACE										
0.0	Water						269				
268.7							268				
1.0							267				
							266				
							265				
							264				
262.8							263				
6.9	End of Penetration Test Refusal at 6.9m							70 Blows for last 225mm Refusal - 50 Blows / 0 mm			

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

Golder Associates

RECORD OF PENETRATION TEST No DC03-60 1 OF 1 METRIC

PROJECT 021-1147
W.P. GWP 99-98-00
DIST 54 HWY 17
DATUM GEODETIC

LOCATION N 5144148.7 ;E 304300.9
BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST
DATE Aug. 01, 2003

ORIGINATED BY EHS
COMPILED BY JDR
CHECKED BY CMG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
270.5	GROUND SURFACE							20 40 60 80 100	10 20 30		
0.0							270				
269.2											
1.3	End of Penetration Test Refusal at 1.3m							12 Blows for last 150mm Refusal - 50 Blows / 0 mm			

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

Golder Associates

RECORD OF PENETRATION TEST No DC03-61 1 OF 1 METRIC

PROJECT 021-1147
W.P. GWP 99-98-00
DIST 54 HWY 17
DATUM GEODETIC
LOCATION N 5144196.9 ;E 304314.0
BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST
DATE Aug. 01, 2003
ORIGINATED BY EHS
COMPILED BY JDR
CHECKED BY CMG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
269.8	GROUND SURFACE										
0.0							269				
268.1											
1.7	End of Penetration Test Refusal at 1.7m							34 Blows for last 150mm Refusal - 50 Blows / 0 mm			

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

FIELD TEST PIT LOG

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	March 17, 2003
TEST PIT NUMBER:	TP03-01	**TEST PIT SIZE:**	1.2 m x 2.4 m x 4.6 m	**ELEVATION:**	269.4 m
MACHINE TYPE:	Cat E120 B	**CONTRACTOR:**	Interpaving	**DATUM:**	Geodetic
TEMPERATURE:	+5 °C	**WEATHER:**	Overcast	**LOCATION:**	N 5144583.1 E 304388.8

Depth		Soil Description	Samples		Pocket Penetrometer Results		Remarks
From (m)	To (m)		No.	Depth (m)	Depth (m)	q_u (kg/cm^2)	
0.0	0.2	Topsoil, dark brown					
0.2	1.3	Silt, some clay, trace sand, mottled brown and grey, moist	1	0.2-1.3			w=32.7%
1.3	2.9	Varved Silty Clay, trace sand, occasional silt seams, firm to stiff, light brown and reddish brown	2	1.3-2.9	1.3-2.9	1.00 1.00 1.25	w=41.8%

Comments:

End of Test Pit at 2.9 m below ground surface

Water Conditions in Test Pit:

Water seeping in from ground surface

☐ Test Pit Dry

JOB No.	021-1147
TEST PIT No.:	01
ENGINEER:	KJB

FIELD TEST PIT LOG

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	March 17, 2003
TEST PIT NUMBER:	TP03-02	**TEST PIT SIZE:**	1.2 m x 1.8 m	**ELEVATION:**	268.7
MACHINE TYPE:	Cat E120 B	**CONTRACTOR:**	Interpaving	**DATUM:**	Geodetic
TEMPERATURE:	+5 °C	**WEATHER:**	Overcast	**LOCATION:**	N 544538.4 E 304383.2

Depth		Soil Description	Samples		Pocket Penetrometer Results		Remarks
From (m)	To (m)		No.	Depth (m)	Depth (m)	q_u (kg/cm^2)	
0.0	0.6	Fibrous Peat, dark brown					Frozen to depth of 0.5 m
0.6	1.5	Silt, some clay, trace sand, brown and grey	1	0.6-1.5			w=39.1%
1.5	2.4	Varved Silty Clay to Clay, trace sand, frequent sand seams, grey and reddish brown	2	1.5-2.4			w=36.5%, w_l=52.4%, w_p=18.2%
2.4	3.4	Varved Silty Clay, trace sand, occasional sand seams, firm to stiff, light brown and reddish brown	3	2.4-3.4	2.4-3.4	1.00 1.00 1.25	w=40.7%

Comments:

End of Test Pit at 3.4 m below ground surface

Water Conditions in Test Pit:

Water seeping in from ground surface

☐ Test Pit Dry

JOB No.	021-1147
TEST PIT No.:	02
ENGINEER:	KJB

FIELD TEST PIT LOG

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	March 17, 2003
TEST PIT NUMBER:	TP03-03	**TEST PIT SIZE:**	1.2 m x 2.4 m x 4.6 m	**ELEVATION:**	268.7
MACHINE TYPE:	Cat E120 B	**CONTRACTOR:**	Interpaving	**DATUM:**	Geodetic
TEMPERATURE:	-1 °C	**WEATHER:**	Overcast	**LOCATION:**	N 5144493.8 E 304377.2

Depth		Soil Description	Samples		Pocket Penetrometer Results		Remarks
From (m)	To (m)		No.	Depth (m)	Depth (m)	q_u (kg/cm^2)	
0.0	0.4	Fibrous Peat, dark brown					Frozen to 0.4 m depth
0.4	1.7	Silt, some clay, trace sand and gravel, mottled brown and grey, moist	1	0.4-1.7			w=35.8%
1.7	2.5	Varved Silty Clay to Clay, trace sand, occasional sand seams, stiff, grey and reddish brown	2	1.7-2.5	1.7-2.5	1.25 1.50 1.25	w=41.0%, w_l=63.8%, w_p=24.7%
2.5	3.8	Varved Silty Clay, trace sand, occasional sand seams, very stiff, grey and reddish brown	3	2.5-3.8	2.5-3.8	2.25 2.25 2.25	w=42.6%

Comments:

End of Test Pit at 3.8 m below ground surface
Refer to Appendix A for Grain Size Distribution of Sample No. 2.

Water Conditions in Test Pit:

Water seeping in from ground surface

☐ Test Pit Dry

JOB No.	021-1147
TEST PIT No.:	03
ENGINEER:	KJB

FIELD TEST PIT LOG

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	March 17, 2003
TEST PIT NUMBER:	TP03-04	**TEST PIT SIZE:**	1.2 m x 1.8 m x 4.6 m	**ELEVATION:**	269.0
MACHINE TYPE:	Cat E120 B	**CONTRACTOR:**	Interpaving	**DATUM:**	Geodetic
TEMPERATURE:	+4 °C	**WEATHER:**	Overcast	**LOCATION:**	N 5144443.9 E 304370.9

Depth		**Soil Description**	**Samples**		**Pocket Penetrometer Results**		**Remarks**
From (m)	**To (m)**		**No.**	**Depth (m)**	**Depth (m)**	**q_u (kg/cm^2)**	
0.0	0.3	Topsoil, some gravel, dark brown					Frozen to 0.75 m depth
0.3	2.1	Silt, some clay, trace sand and gravel, mottled grey and brown, moist	1	0.3-2.1			w=38.1%
2.1	3.2	Varved Silty Clay, trace sand, transition to Varved Clayey Silt at 2.7 m depth, very stiff, reddish brown	2	2.1-3.2	2.1-3.2	2.50 2.50 2.50	w=38.1%
-	3.2	Refusal (Inferred bedrock)					

Comments:

End of Test Pit at 3.2 m depth

Water Conditions in Test Pit:

Water seeping in at depth of 3.2 m (on top of exposed bedrock)

☐ Test Pit Dry

JOB No. 021-1147
TEST PIT No.: 04
ENGINEER: KJB

FIELD TEST PIT LOG

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	March 17, 2003
TEST PIT NUMBER:	TP03-05	**TEST PIT SIZE:**	1.2 m x 1.8 m x 1.6 m	**ELEVATION:**	269.4
MACHINE TYPE:	Cat E120 B	**CONTRACTOR:**	Interpaving	**DATUM:**	Geodetic
TEMPERATURE:	+5 °C	**WEATHER:**	Overcast	**LOCATION:**	N 5144406.4
					E 304363.2

Depth		Soil Description	Samples		Pocket Penetrometer Results		Remarks
From (m)	To (m)		No.	Depth (m)	Depth (m)	q_u (kg/cm^2)	
0.0	0.15	Crushed Gravel, brown (FILL)					Soil frozen to a depth of 1m.
0.15	0.3	Sand, some gravel, trace silt and cobbles, light brown (FILL)					
0.3	0.6	Fibrous Peat, trace cobbles, dark brown					
0.6	1.6	Silt, some clay, trace sand and gravel, mottled brown and grey					
1.6	-	Inferred Bedrock surface					

Comments:

Bedrock outcrop visually identified 3 m Southeast of Test Pit location.
End of Test Pit at 1.6 m below ground surface.

Water Conditions in Test Pit:

Water seeping in at ground surface.

☒ Test Pit Dry

JOB No.	021-1147
TEST PIT No.:	05
ENGINEER:	KJB

FIELD TEST PIT LOG

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Embankments / Sudbury	**DATE:**	March 17, 2003
TEST PIT NUMBER:	TP03-06	**TEST PIT SIZE:**	1.2 m x 2.4 m x 5.2 m	**ELEVATION:**	269.1
MACHINE TYPE:	Cat E120 B	**CONTRACTOR:**	Interpaving	**DATUM:**	Geodetic
TEMPERATURE:	+5 °C	**WEATHER:**	Overcast	**LOCATION:**	N 5144359.1 E 304353.5

Depth		Soil Description	Samples		Pocket Penetrometer Results		Remarks
From (m)	To (m)		No.	Depth (m)	Depth (m)	q_u (kg/cm^2)	
0.0	0.3	Ice					
0.3	0.9	Fibrous Peat, dark brown					
0.9	1.9	Clayey Silt, trace sand and cobbles, grey, moist (Occasional cobbles and boulders from 1.5 m to 1.9 m)	1, 2	0.9 – 1.5			w=22.6%
1.9	3.5	Varved Clayey Silt, trace sand, contains cobbles/boulders, stiff, grey.	3	1.9-3.5	1.9-3.5	1.75 1.75 1.75	w=30.1%, w_l=32.3%, w_p=21.2%

Comments:

End of Test Pit at 3.5 m below ice surface.
Refer to Appendix A for Grain Size Distribution of Samples No. 1 and No. 3

Water Conditions in Test Pit:

0.3 m of ice at ground surface.

☐ Test Pit Dry

JOB No.	021-1147
TEST PIT No.:	06
ENGINEER:	KJB

FIELD TEST PIT LOG

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	March 17, 2003
TEST PIT NUMBER:	TP03-07	**TEST PIT SIZE:**	1.2 m x 2.4 m x 3.0 m	**ELEVATION:**	269.2
MACHINE TYPE:	Cat E120 B	**CONTRACTOR:**	Interpaving	**DATUM:**	Geodetic
TEMPERATURE:	+5 °C	**WEATHER:**	Overcast	**LOCATION:**	N 5144309.9 E 304343.0

Depth		Soil Description	Samples		Pocket Penetrometer Results		Remarks
From (m)	To (m)		No.	Depth (m)	Depth (m)	q_u (kg/cm^2)	
0.0	0.3	Ice					
0.3	0.35	Topsoil, dark brown					
0.35	0.5	Rock fill with gravel, sand and silt (FILL)					
0.5	1.7	Silt, some clay, trace sand and gravel, mottled brown and gray, moist	1	0.5-1.7			w=39.5%
1.7	2.0	Varved Clayey Silt, trace sand, grey	2	1.7-2.0			w=47.0%
2.0	3.0	Varved Silty Clay, trace sand, firm, reddish brown and brown.	3	2.0-3.0	2.0-3.0	0.75 0.75 0.75	w=48.8%
3.0	3.5	Varved Silty Clay to Clay, trace sand, very soft to soft, reddish brown and grey	4	3.0-3.5	3.0-3.50	<0.25 <0.25 <0.25	w=53.9%, w_l=55.6%, w_p=22.8%

Comments:

End of Test Pit at 3.5 m below ground surface.

Water Conditions in Test Pit:

0.3 m of ice above ground surface.

☐ Test Pit Dry

JOB No.	021-1147
TEST PIT No.:	07
ENGINEER:	KJB

FIELD TEST PIT LOG

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours/ Sudbury	**DATE:**	March 17, 2003
TEST PIT NUMBER:	TP03-08	**TEST PIT SIZE:**	1.2 m x 1.8 m	**ELEVATION:**	269.6
MACHINE TYPE:	Cat E120 B	**CONTRACTOR:**	Interpaving	**DATUM:**	Geodetic
TEMPERATURE:	+5 °C	**WEATHER:**	Overcast	**LOCATION:**	N 5144280.7 E 304335.3

Depth		Soil Description	Samples		Pocket Penetrometer Results		Remarks
From (m)	To (m)		No.	Depth (m)	Depth (m)	q_u (kg/cm^2)	
0.0	0.15	Topsoil, dark brown					
0.15	1.0	Silt, some clay, trace sand, mottled brown and grey					
1.0	1.50	Varved Silty Clay, trace sand, reddish brown to grey	1	1.0-1.5			w=37.0%, w_l=46.5%, w_p=22.6%
1.5	2.9	Silt, some clay, trace gravel and cobbles, occasional boulders, brown	2	1.5-2.9			w=27.2%

Comments:

Bedrock surface or boulder encountered at 0.5 m below ground surface at west side of Test Pit.

Water Conditions in Test Pit:

Water seeping into Test Pit from ground surface

☐ Test Pit Dry

JOB No.	021-1147
TEST PIT No.:	08
ENGINEER:	KJB

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FIELD PROBE HOLE LOG No. PH03-01

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	August 29, 2003
PROBEHOLE NUMBER:	PH03-01	**PROBEHOLE SIZE:**	50 mm diameter	**ELEVATION:**	282.4 m
MACHINE TYPE:	Hand driven	**CONTRACTOR:**	Golder	**DATUM:**	Geodetic
TEMPERATURE:	n/a	**WEATHER:**	Overcast	**LOCATION:**	N 5143383.4 E 304539.0

Depth From (m)	Depth To (m)	Soil Description	Samples No.	Samples Depth (m)	Remarks
0.0 -	- 0.85	Surficial soil consists of Silty Sand with cobbles and boulders Refusal at 0.85 m depth	-	-	Bedrock outcrop was visually confirmed at Station 16+256

Comments:

Location: Station No. 16+260, along staked RT EP Line

Refusal of Probehole at 0.85 m depth below ground surface.

JOB No.	021-1147
PROBEHOLE No.:	PH03-01
ENGINEER:	KJB

FIELD PROBE HOLE LOG No. PH03-02

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	August 29, 2003
PROBEHOLE NUMBER:	PH03-02	**PROBEHOLE SIZE:**	50 mm diameter	**ELEVATION:**	284.2 m
MACHINE TYPE:	Hand driven	**CONTRACTOR:**	Golder	**DATUM:**	Geodetic
TEMPERATURE:	n/a	**WEATHER:**	Overcast	**LOCATION:**	N 5143370.4 E 304587.2

Depth From (m)	Depth To (m)	Soil Description	Samples No.	Samples Depth (m)	Remarks
0.0 -	- 0.75	Surficial soil consists of Silty Sand with cobbles and boulders Refusal at 0.75 m depth	-	-	Bedrock outcrops were visually confirmed at Stations 16+306 and 16+315

Comments:

Location: Station No. 16+310, along staked RT EP Line

Refusal of Probehole at 0.75 m depth below ground surface.

JOB No.	021-1147
PROBEHOLE No.:	PH03-02
ENGINEER:	KJB

FIELD PROBE HOLE LOG No. PH03-03

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	August 29, 2003
PROBEHOLE NUMBER:	PH03-03	**PROBEHOLE SIZE:**	50 mm diameter	**ELEVATION:**	282.0 m
MACHINE TYPE:	Hand driven	**CONTRACTOR:**	Golder	**DATUM:**	Geodetic
TEMPERATURE:	n/a	**WEATHER:**	Overcast	**LOCATION:**	N 5143351.0 E 304633.3

Depth From (m)	Depth To (m)	Soil Description	Samples No.	Samples Depth (m)	Remarks
0.0	-	Surficial soil consists of Silty Sand with cobbles and boulders	-	-	Bedrock outcrops were visually confirmed at Stations 16+355 and 16+375
-	0.50	Refusal at 0.50 m depth			

Comments:

Refusal of Probehole at 0.50 m depth below ground surface.

Station No. 16+360, along staked RT EP Line

JOB No.	021-1147
PROBEHOLE No.:	PH03-03
ENGINEER:	KJB

FIELD PROBE HOLE LOG No. PH03-04

JOB NUMBER:	021-1147	**JOB NAME:**	EarthTech / Detours / Sudbury	**DATE:**	August 29, 2003
PROBEHOLE NUMBER:	PH03-04	**PROBEHOLE SIZE:**	50 mm diameter	**ELEVATION:**	280.1 m
MACHINE TYPE:	Hand driven	**CONTRACTOR:**	Golder	**DATUM:**	Geodetic
TEMPERATURE:	n/a	**WEATHER:**	Overcast	**LOCATION:**	N 5143336.8 E 304665.2

Depth From (m)	Depth To (m)	Soil Description	Samples No.	Samples Depth (m)	Remarks
0.0	-	Refusal on bedrock outcrop	-	-	Surficial boulders were visually confirmed in the vicinity of the probehole

Comments:

Location: Station No. 16+395, along staked RT EP Line

Note: An additional probehole achieved refusal at 1.3 m depth at Station 16+390

JOB No.	021-1147
PROBEHOLE No.:	PH03-04
ENGINEER:	KJB

TABLE 1
Summary of Recommendations at High Fill and Swamp Crossings
Detour #1, Detour #2, and Widening of Municipal Road 80
G.W.P. 99-98-00

Highway / Roadway	*Approx. Station*	*Proposed Works*	*Surface Conditions*	*Recommended Embankment Fill Type*	*Organics Encountered along alignment*	*Recommended Side Slope*	*Mitigation Measure and Side Berm Recommended*	*Estimated Post-Construction Settlement* (mm)*	*Swamp Excavation / Organic Removal OPSD*
East Side of Municipal Road 80	9+000 to 9+340	Swamp Crossing (fill up to 2 m high)	Swamp Area A10	Earth fill (from 9+000 to 9+300) Rock fill (from 9+300 to 9+340) (separator req'd at ≈ 9+300)**	Yes. Up to 0.9 m below ground surface.	2H:1V (for earth fill) 1.25H:1V (for rock fill)	Subexcavation (in strips) up to 5 m deep. Protection of existing MR 80 required. No berms required.	<5 (earth fill) 30 (rock fill)	203.020 (excavate existing slopes to 1H:1V)
	9+340 to 9+470	Swamp Crossing (fill up to 5.5 m high)	Swamp Area A12E	Rock fill	Yes. Up to 0.5 m below ground surface.	1.25H: 1V	No.	105	203.020 (excavate existing slopes to 1H:1V)
West Side of Municipal Road 80 and Detour #1	9+290 to 9+400	Swamp Crossing (fill up to 2.5 m high)	Swamp Area A12W	Rock fill	Yes. Up to 0.5 m below ground surface.	1.25H:1V	Subexcavation (in strips) up to 2.5 m deep. Protection of existing MR 80 required. No berms required.	50	203.020 (excavate existing slopes to 1H:1V)
	9+400 to 9+450	Swamp Crossing (fill up to 5 m high)	Swamp Area A12W	Rock fill	Yes. Up to 0.9 m below ground surface.	1.25H: 1V	Subexcavation (in strips) up to 5 m deep. Protection of existing MR 80 required. No berms required.	100	203.020 (excavate existing slopes to 1H:1V)

Note : ***Settlements include compression of new effective thickness of rock fill plus compression of cohesive layers below embankment (where not removed)**
****Graded granular separator required at transition from earth fill to rock fill**

TABLE 1
Summary of Recommendations at High Fill and Swamp Crossings
Detour #1, Detour #2, and Widening of Municipal Road 80
G.W.P. 99-98-00

Highway / Roadway	*Approx. Station*	*Proposed Works*	*Surface Conditions*	*Recommended Embankment Fill Type*	*Organics Encountered along alignment*	*Recommended Side Slope*	*Mitigation Measure and Side Berm Recommended*	*Estimated Post-Construction Settlement* (mm)*	*Swamp Excavation / Organic Removal OPSD*
	9+450 to 9+510	Swamp Crossing (fill up to 6 m high)	Swamp Area A12W	Rock fill	Yes. Up to 0.5 m below ground surface.	1.25H:1V	Subexcavation (in strips) up to 5 m deep. Protection of existing MR 80 required. No berms required.	110	203.020 (excavate existing slopes to 1H:1V)
	9+850 to 9+950	Swamp Crossing (fill up to 2.5m high)	Swamp Area A2	Rock fill	Yes. Up to 0.2 m below ground surface.	1.25H:1V	No.	40	203.020 (excavate existing slopes to 1H:1V)
Detour # 2	16+180 to 16+230	High Fill (fill up to 9 m)	East limit of Swamp Area A8	Rock fill	Yes. Up to 0.1 m below ground surface.	1.25H:1V	Yes, berms 6m high x 2 m wide (where embankment height exceeds 6 m).	90	203.010 (new embankment over swamp) and remove all organics within footprint of embankment.
	16+230 to 16+430	High Fill (fill up to 7 m)	Undulating surface with bedrock outcrops	Rock fill	Less than 0.1 m thick.	1.25H:1V	Yes, berms 6m high x 2 m wide (where embankment height exceeds 6 m).	70	Remove all organics within footprint of embankment.

***Note : Settlements include compression of new effective thickness of rock fill plus compression of cohesive layers below embankment (where not removed)**

TABLE 1
Summary of Recommendations at High Fill and Swamp Crossings
Detour #1, Detour #2, and Widening of Municipal Road 80
G.W.P. 99-98-00

Highway / Roadway	*Approx. Station*	*Proposed Works*	*Surface Conditions*	*Recommended Embankment Fill Type*	*Organics Encountered along alignment*	*Recommended Side Slope*	*Mitigation Measure and Side Berm Recommended*	*Estimated Post-Construction Settlement* (mm)*	*Swamp Excavation / Organic Removal OPSD*
	16+430 to 16+500	Swamp Crossing (fill up to 8 m high)	Swamp Area A13	Rock fill	Yes. Up to 0.6 m below ground surface.	1.25H : 1V	Subexcavation up to 3.8 m. Protection of existing Hwy 17 required. Yes, berms 6m high x 2 m wide (where embankment height exceeds 6 m).	120	203.020 (excavate existing slopes to 1H:1V)

***Note : Settlements include compression of new effective thickness of rock fill plus compression of cohesive layers below embankment (where not removed)**

TABLE 2
EVALUATION OF SETTLEMENT MITIGATION ALTERNATIVES
SWAMP A10
MUNICIPAL ROAD 80 WIDENING (EAST SIDE) – STATION 9+000 TO 9+340
G.W.P. 99-98-00

Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Full Sub-excavation (up to 5 m deep)	1	Long-term settlement issues minimized since all weak, soft and compressible material are removed.	Specialized excavation equipment may be required (drag-line) due to high groundwater level and deep excavation. Increased time for construction. Protection of adjacent existing MR 80 roadway embankments required (by staged excavation or temporary shoring)	Effective thickness of embankment increases and adds to fill quantities. Additional costs for subexcavation plus costs of increased time for excavation in stages or temporary shoring	Settlement of foundation soils minimized. Additional embankment rock fill settlement due to increased effective embankment height.
Pre-Loading (about 4.5 months)	2	No deep subexcavation.	Construction schedule may not allow preloading period required. Increased construction time required. Settlement of foundation soils takes about 4.5 months to reach 95% consolidation.	Reduced costs for smaller quantity of embankment fill material as compared with full sub-excavation and surcharging.	Settlement of embankment material and foundation soils will occur. Secondary consolidation (creep) settlements will occur.
Surcharging (2 m high and 2 months surcharge time)	3	Time for 95% of final embankment geometry consolidation settlement will be reduced to about 2 months.	Approximately twice as much embankment fill material required to be placed and removed.	Increased cost of construction and material for surcharge, however surcharge fill can be reused elsewhere at site.	Construction schedule accelerated by about 2 months. Settlement of embankment material will occur. Secondary consolidation (creep) settlements will occur.
Wick Drains (1.5 m triangular drain spacing and 3 weeks preload time)	4	95% of time-dependant consolidation settlement complete within about 3 weeks.	Increased time for installation. Monitoring of settlements and pore-pressures required.	Cost savings in surcharge fill or subexcavation offset by wick drain installation and monitoring costs.	Settlement of embankment material will occur. Secondary consolidation (creep) settlements will occur.
Light Weight Fill (up to 300 m long x 5 m wide x 2 m high)	5	Reduces load on compressible soils thereby reducing settlement of foundation soils. Settlement of embankment materials minimized.	Very large costs of materials.	Cost savings in berm fill or subexcavation, but relative cost of fill is up to an order of magnitude higher than for the other options.	Settlements of foundation soils and embankment materials minimized.

TABLE 3
EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES
SWAMP A12W
MUNICIPAL ROAD 80 WIDENING (WEST SIDE) – STATION 9+290 TO 9+510
G.W.P. 99-98-00

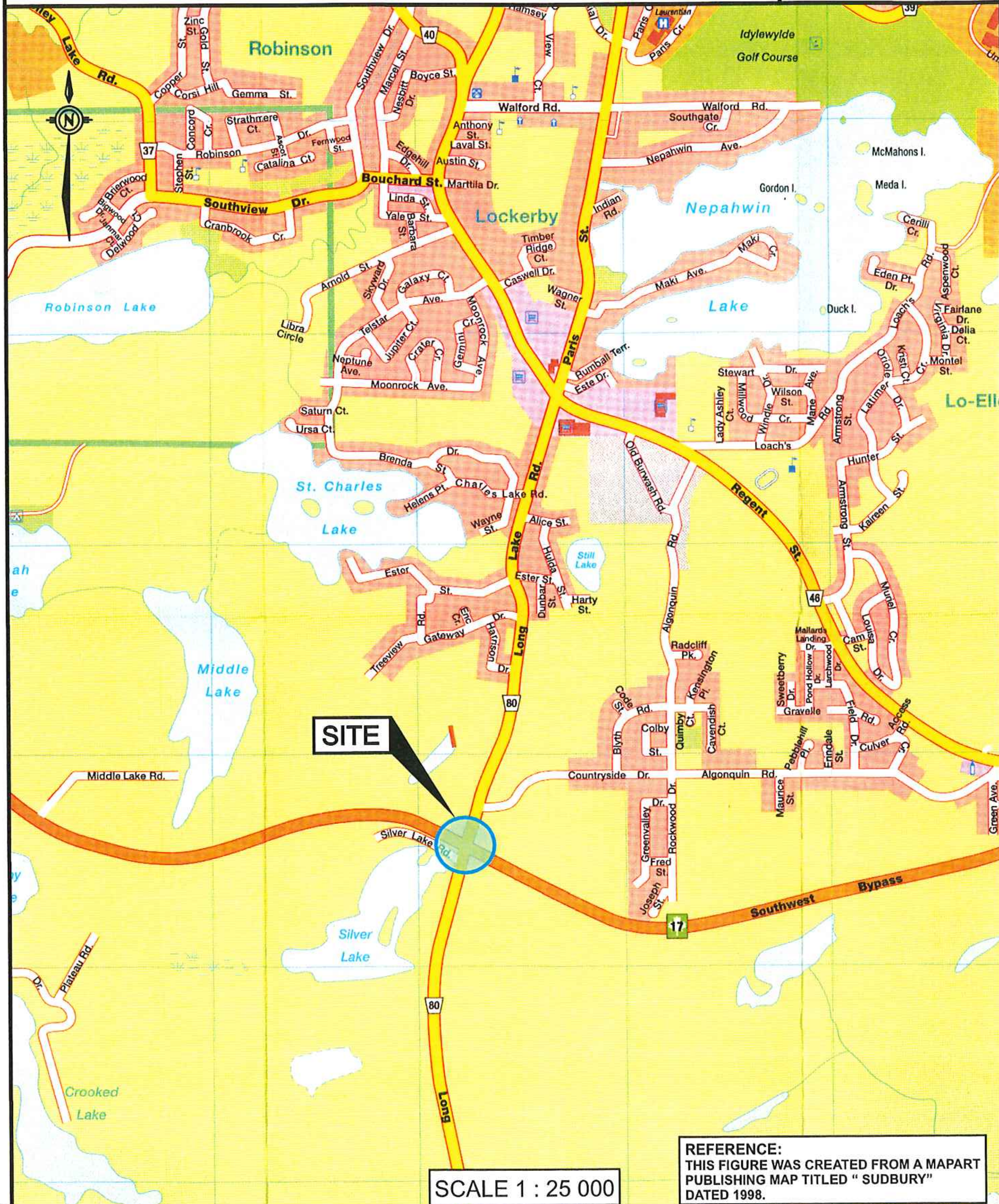
Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Full Sub-excavation (up to 5 m deep)	1	Stability and long-term settlement issues minimized since all weak, soft and compressible material are removed. Toe berms not required for stabilty between Station 9+400 to 9+500.	Specialized excavation equipment may be required (drag-line) due to high groundwater level and deep excavation. Increased time for construction. Protection of adjacent existing MR 80 roadway embankments required (by staged excavation or temporary shoring).	Effective thickness of embankment increases and adds to fill quantities. Additional costs for subexcavation plus costs of increased time for excavation in stages or temporary shoring.	Settlement of foundation soils minimized. Factor of safety for stability increased. Additional embankment rock fill settlement due to increased effective embankment height.
Pre-Loading (about 6 months) and Toe Berms (6 m wide x 2 m high x 50 m long and 3 m wide x 2 m high x 50 m long)	2	No additional subexcavation.	Toe berms required for stability between Station 9+400 to 9+500. Increased quantity of materials and increased construction time required. Settlement of foundation soils takes about 6 months to reach 95% consolidation.	Increased cost for berm material and construction.	Factor of safety for stability increased. Settlement of embankment material and foundation soils will occur. Secondary consolidation (creep) settlements will occur.
Wick Drains / Staged Construction (1.5 m triangular drain spacing and 3 weeks preload time)	3	95% of time-dependant consolidation settlement complete within about 3 weeks. Staged construction will reduce size of berms required for stability.	Increased time for installation. Monitoring of settlements and pore-pressures required for staged construction.	Cost savings in berm fill or subexcavation offset by wick drain installation and monitoring costs.	Settlement of embankment material will occur. Secondary consolidation (creep) settlements will occur.
Surcharging (2 m high and 3 months surcharge time)	4	Time for 95% of final embankment geometry consolidation settlement will be reduced to about 3 months.	Toe berms will be required along entire length and increased in size (up to a maximum of about 11 m wide) to maintain stability of higher embankment.	Increased cost of construction and material for surcharge and wider toe berms, however, surcharge fill can be reused elsewhere at site.	Construction schedule accelerated by about 3 months. Settlement of embankment material will occur. Secondary consolidation (creep) settlements will occur.
Light Weight Fill (up to 200 m long x 10 m wide x 4 m high on average)	5	Reduces load on compressible soils thereby increasing stability and reducing settlement of foundation soils. Settlement of embankment materials minimized.	Very large costs of materials.	Cost savings in berm fill or subexcavation, but relative cost of fill is up to an order of magnitude higher than for the other options.	Factor of safety for stability increased. Settlements of foundation soils and embankment materials minimized.

TABLE 4
EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES
SWAMP A13
DETOUR #2 – STATION 16+430 TO 16+500
G.W.P. 99-98-00

Stability/ Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Full Sub-excavation (up to 3.8 m deep)	1	Stability and long-term settlement issues minimized since all weak, soft and compressible material are removed. Toe berms not required for stabilty.	Specialized excavation equipment may be required (drag-line) due to site access and high groundwater level. Temporary shoring would be required to support the adjacent existing Highway 17 embankment during subexcavation. Increased time for construction.	Effective thickness of embankment increases and adds to fill quantities. Additional costs for subexcavation and for temporary shoring.	Settlement of foundation soils minimized. Factor of safety for stability increased. Additional embankment rock fill settlement due to increased effective embankment height.
Staged Construction (2.5 m high lifts each constructed over a 4 week period)	2	95% of time-dependant consolidation settlement for each lift will be complete within about 4 weeks. Staged construction will reduce size of berm required for stability and amount of subexcavation. No temporary shoring required.	Monitoring of settlements and pore-pressures required for staged construction.	Cost savings in berm fill, subexcavation and temporary shoring offset by monitoring costs.	Settlement of embankment material and foundation soils will occur. Secondary consolidation (creep) settlements will occur.
Pre-Loading (about 4 weeks) and Toe Berms (5.5 m wide x 4 m high)	3	Stability improved with use of toe berms. No deep subexcavation. No temporary shoring.	Increased quantity of materials and increased construction time required.	Increased cost for berm materials and construction.	Factor of safety for stability increased. Settlement of embankment material and foundation soils will occur.
Surcharging (2 m high and 3 weeks surcharge time)	4	Time for 95% of final embankment geometry consolidation settlement will be reduced to about 3 weeks.	Toe berms must be increased in size (to about 8 m wide) to maintain stability of higher embankment.	Increased cost of construction and material for surcharge and wider toe berms, however, surcharge fill can be reused elsewhere at site.	Construction schedule accelerated by about 1 week. Settlement of embankment material will occur. Secondary consolidation (creep) settlements will occur.
Light Weight Fill (60 m long x 5 m wide x 8 m high)	5	Reduces load on compressible soils thereby increasing stability and reducing settlement of foundation soils. Settlement of embankment materials minimized.	Very large costs of materials.	Relative cost of fill materials may be an order of magnitude higher than the other options.	Factor of safety for stability increased.

SITE LOCATION MAP

FIGURE 1



REFERENCE:
THIS FIGURE WAS CREATED FROM A MAPART PUBLISHING MAP TITLED " SUDBURY" DATED 1998.

Date: OCTOBER 2003

Project: 021-1147

Golder Associates

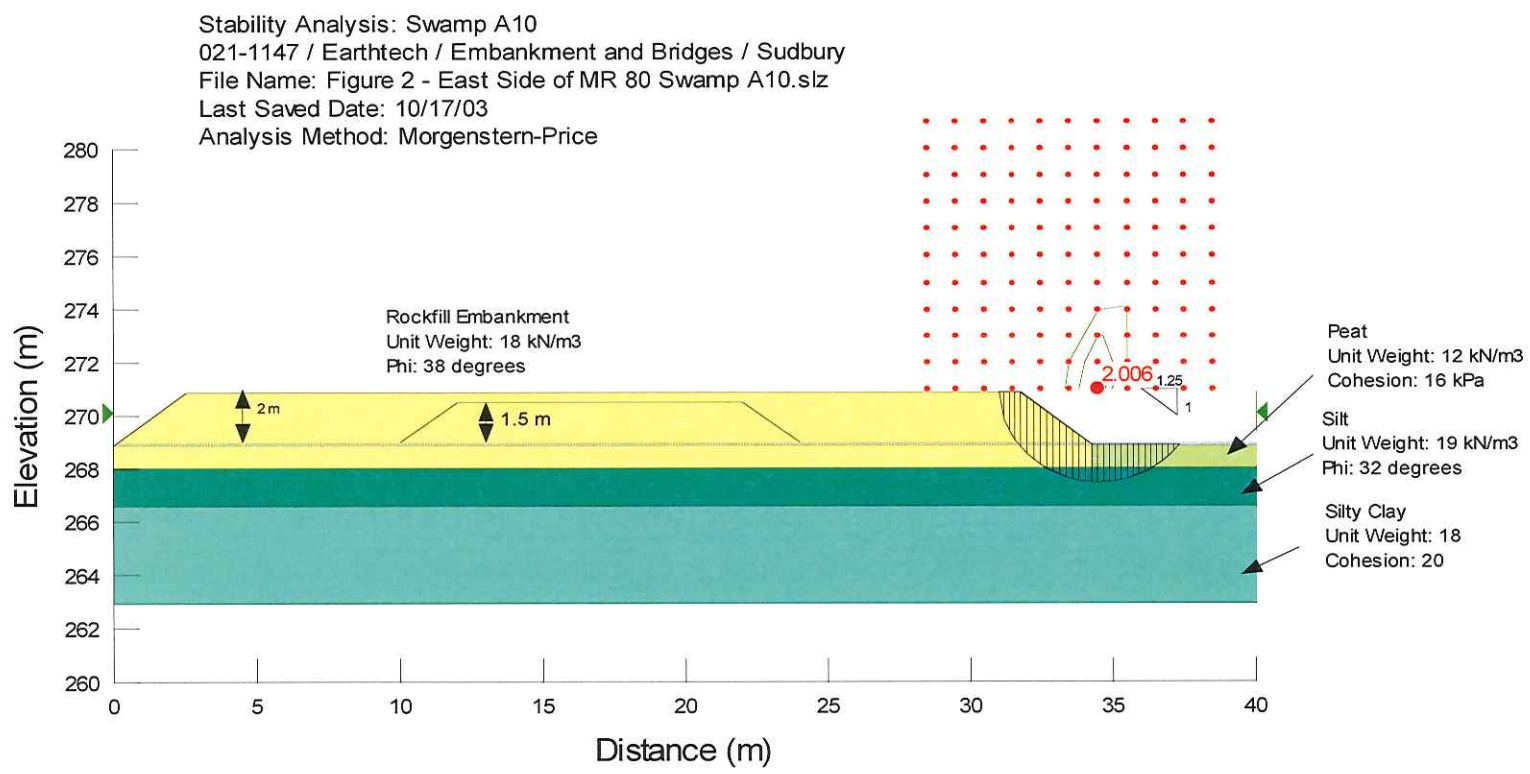
Drawn: RJ

Chkd:

0211147FIG01.cdr

EMBANKMENT OVER SWAMP STABILITY
SWAMP A10 - EAST SIDE OF MR 80

FIGURE 2



NOTE: ORGANICS REMOVED BELOW EMBANKMENT

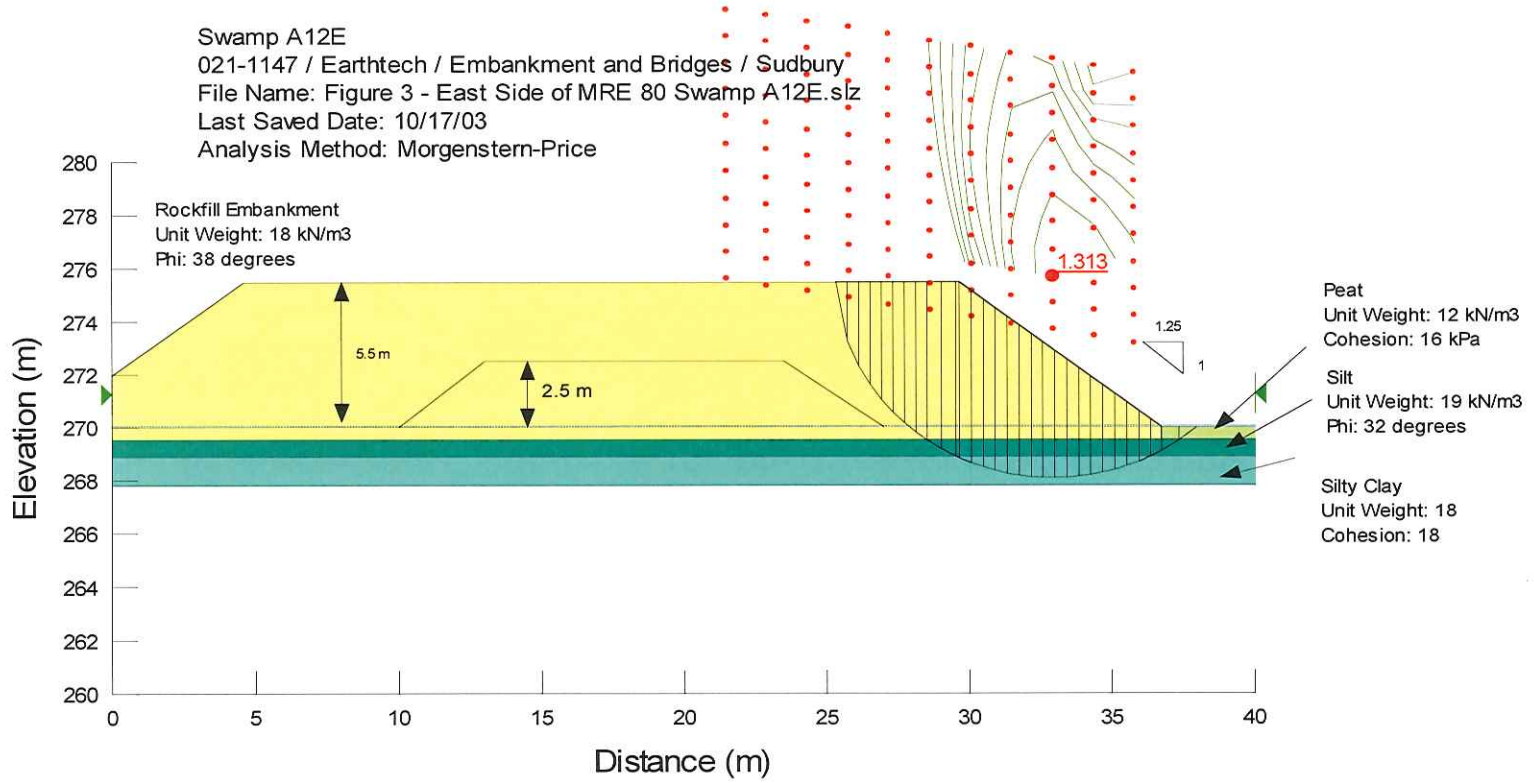
Date: February, 2004
Project: 021-1147

Golder Associates

Drawn: KG
Checked: JPD

EMBANKMENT OVER SWAMP STABILITY
SWAMP A12E - EAST SIDE OF MR 80

FIGURE 3



NOTE: ORGANICS REMOVED BELOW EMBANKMENT

Date: February, 2004
Project: 021-1147

Golder Associates

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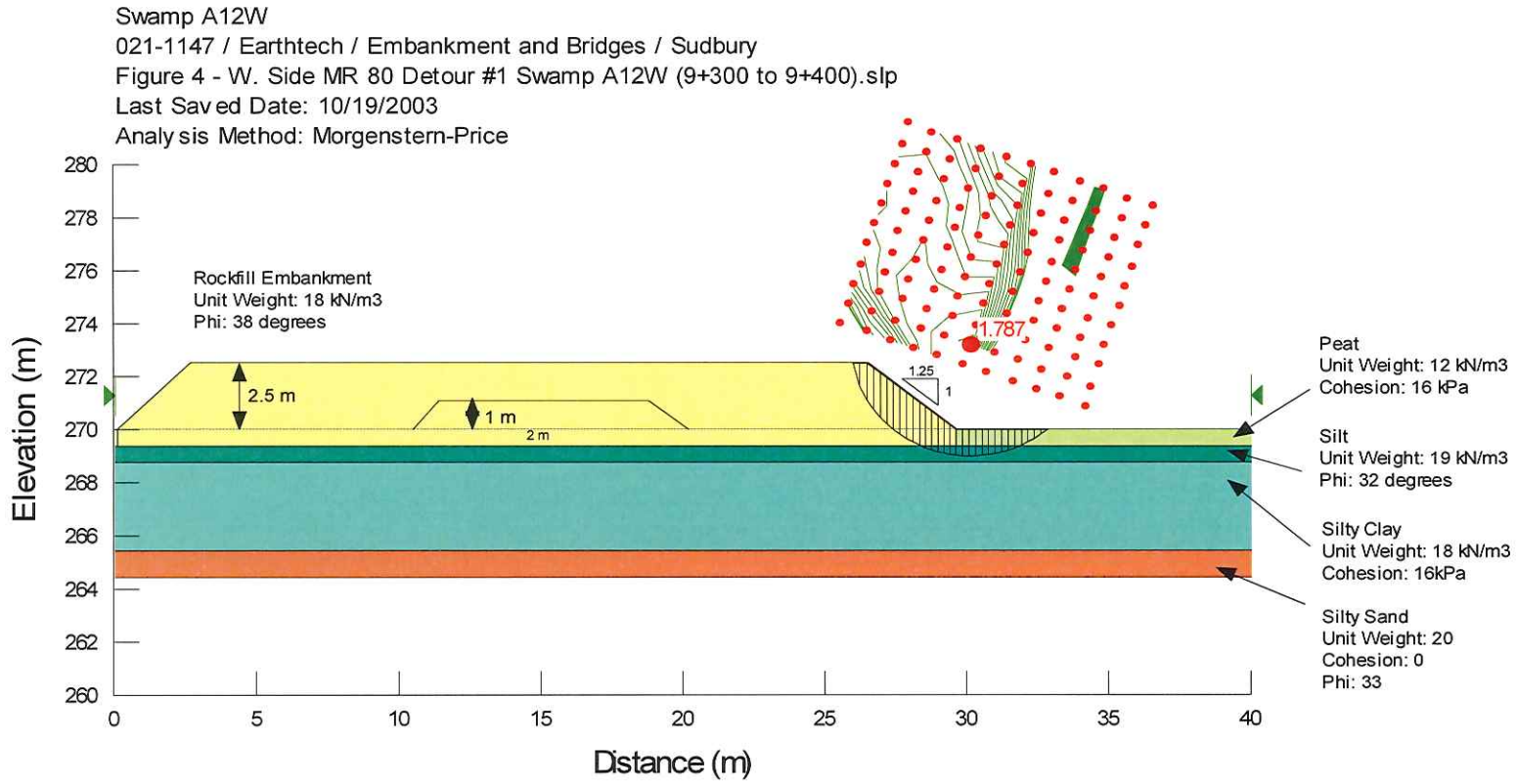
Checked: JPD

EMBANKMENT OVER SWAMP STABILITY
SWAMP A12W - WEST SIDE OF MR 80 AND DETOUR #1 (9+290 TO 9+400)

FIGURE 4



NOTE: ORGANICS REMOVED BELOW EMBANKMENT

Date: February, 2004
Project: 021-1147

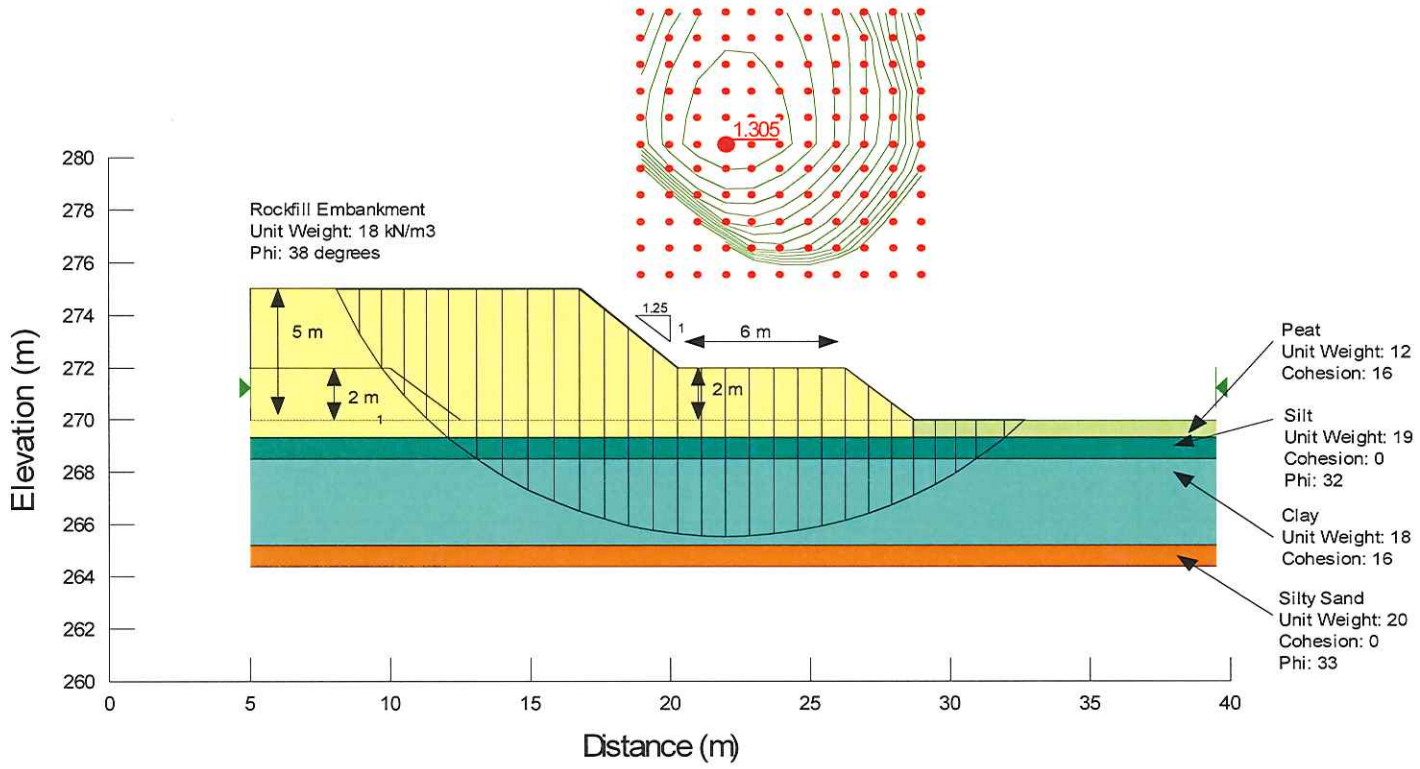
Golder Associates

Drawn: KG
Checked: JPD

EMBANKMENT OVER SWAMP STABILITY
SWAMP A12W - WEST SIDE OF MR 80 AND DETOUR #1 (9+400 TO 9+450)

FIGURE 5

Swamp A12W
021-1147 / Earthtech / Embankment and Bridges / Sudbury
File Name: Figure 5 - W.Side of MR 80 A12W (9+400 to 9+450) 6m berm.slz
Last Saved Date: 10/19/2003
Analysis Method: Morgenstern-Price



NOTE: ORGANICS REMOVED BELOW EMBANKMENT

Date: February, 2004
Project: 021-1147

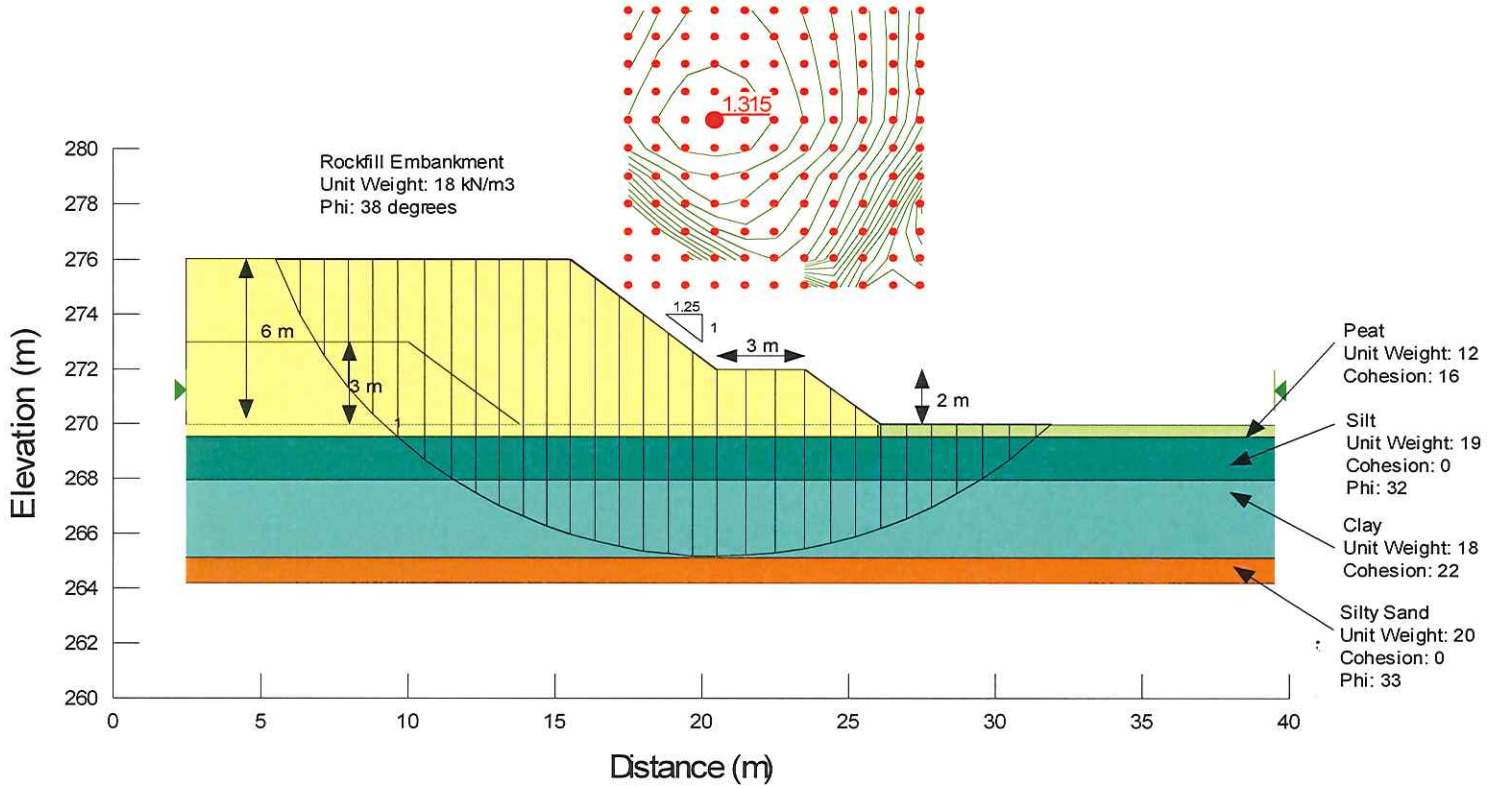
Golder Associates

Drawn: KG
Checked: JPD

EMBANKMENT OVER SWAMP STABILITY
SWAMP A12W - WEST SIDE OF MR 80 AND DETOUR #1 (9+450 TO 9+510)

FIGURE 6

Swamp A12W
021-1147 / Earthtech / Embankment and Bridges / Sudbury
File Name: Figure 6 - W.Side of MR 80 A12W (9+450 to 9+500) 3m Berm.slz
Last Saved Date: 10/19/2003
Analysis Method: Morgenstern-Price



NOTE: ORGANICS REMOVED BELOW EMBANKMENT

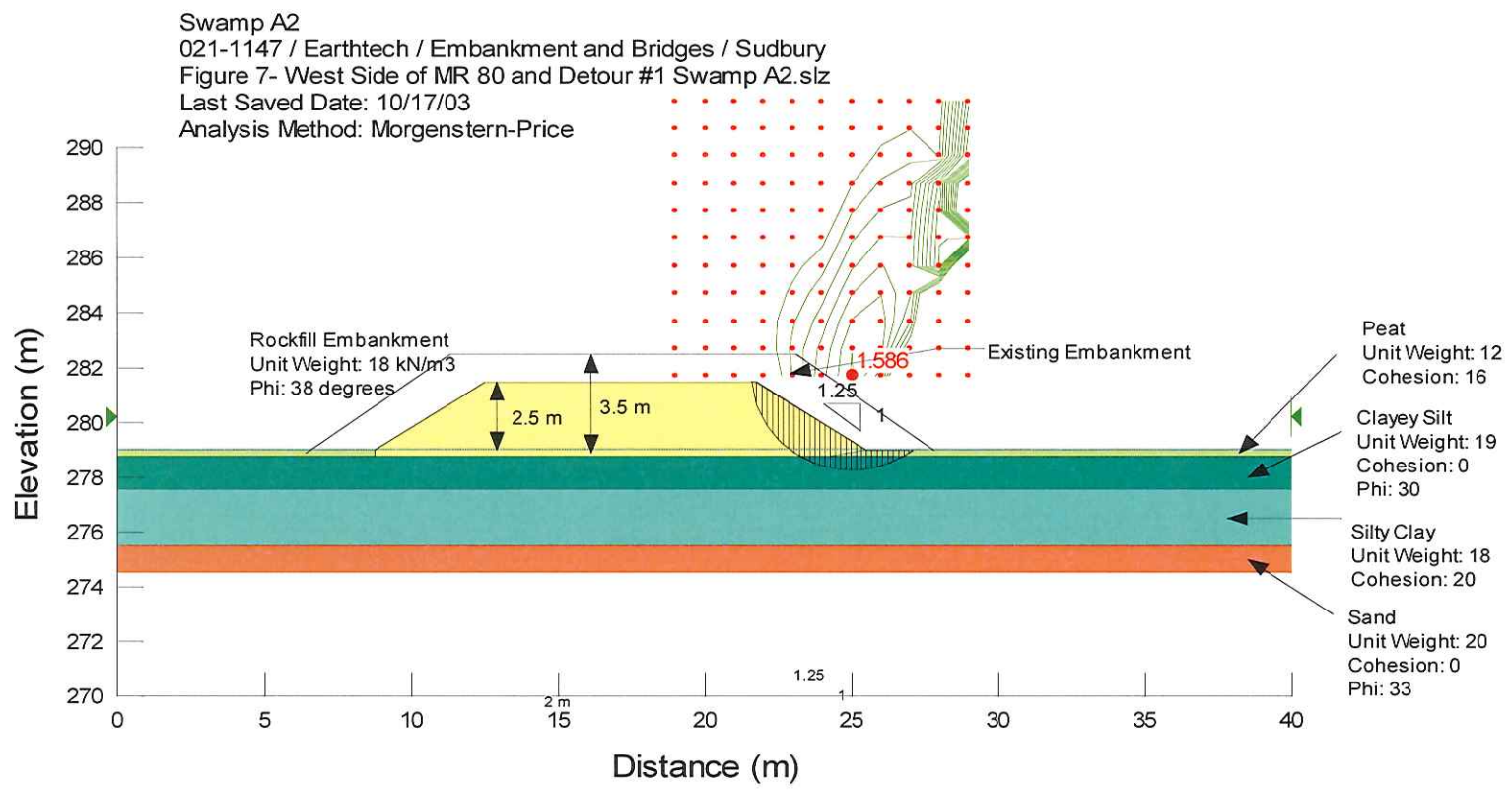
Date: February, 2004
Project: 021-1147

Golder Associates

Drawn: KG
Checked: JPD

EMBANKMENT OVER SWAMP STABILITY
SWAMP A2 - WEST OF MR 80 AND DETOUR #1

FIGURE 7



NOTE: ORGANICS REMOVED BELOW EMBANKMENT

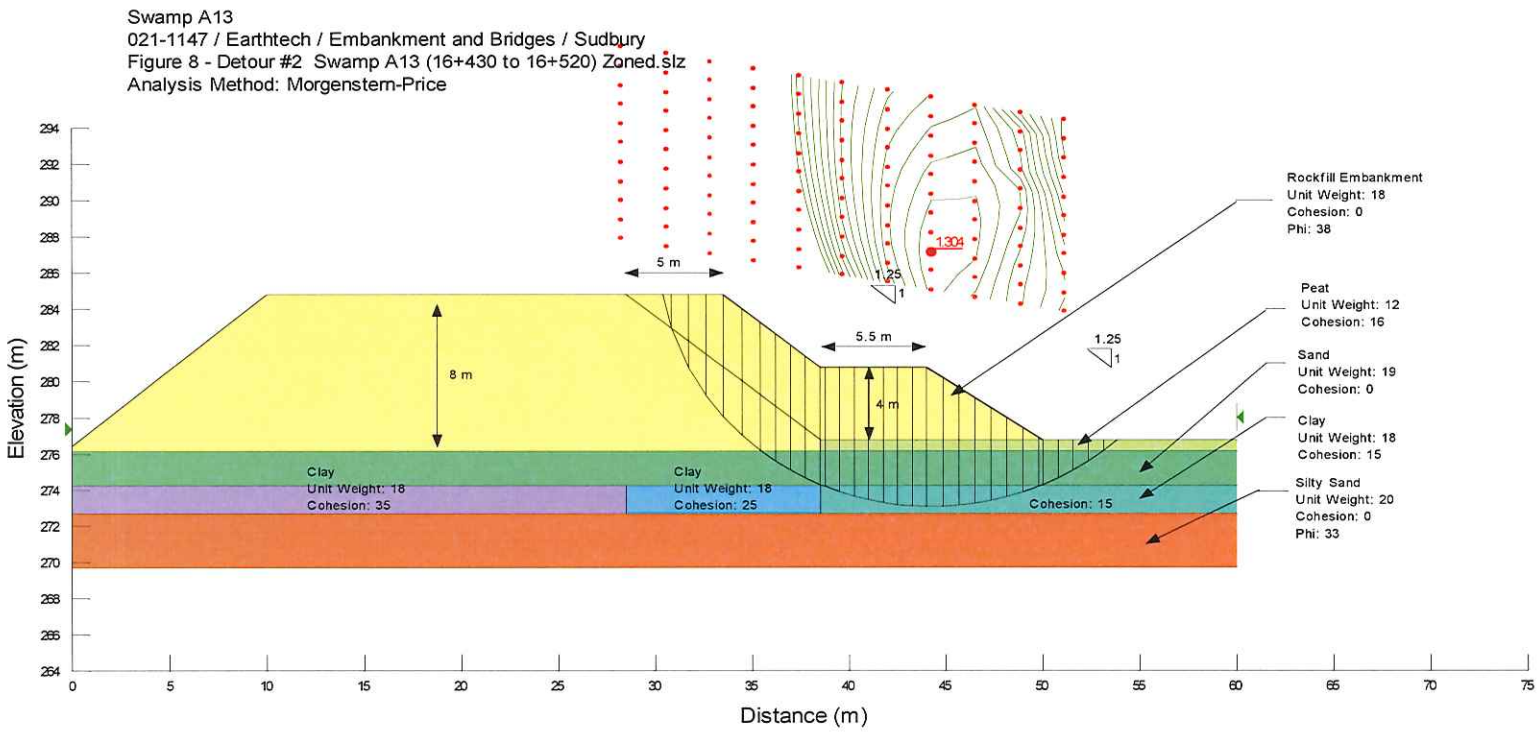
Date: February, 2004
Project: 021-1147

Golder Associates

Drawn: KG
Checked: JPD

EMBANKMENT OVER SWAMP STABILITY
SWAMP A13 - DETOUR #2

FIGURE 8



NOTE: ORGANICS REMOVED BELOW EMBANKMENT

Date: February, 2004
Project: 021-1147

Golder Associates

Drawn: KG
Checked: JPD

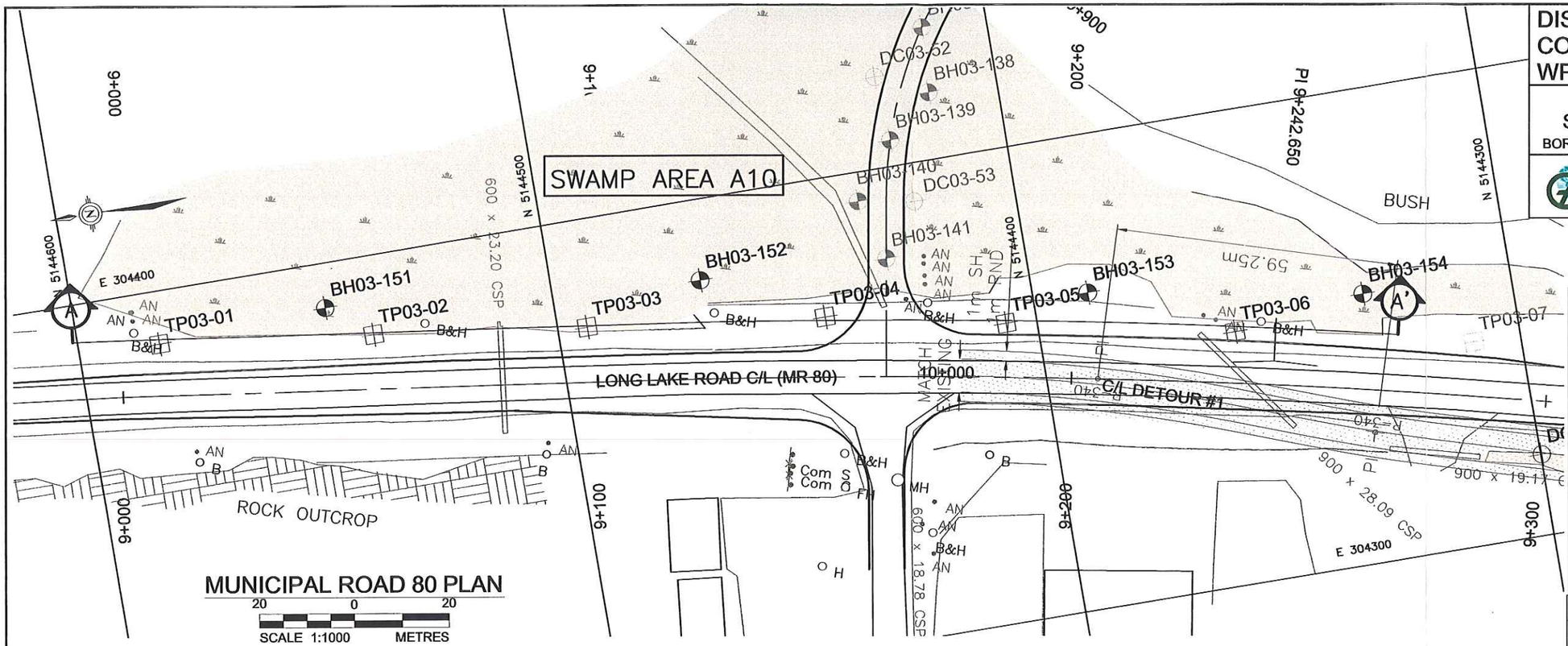
DIST. 54 HWY. 17
CONT No.
WP No. 99-98-00

SHEET

MUNICIPAL ROAD 80
STATION 9+000 to 9+260
BOREHOLE LOCATION AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



MUNICIPAL ROAD 80 PLAN
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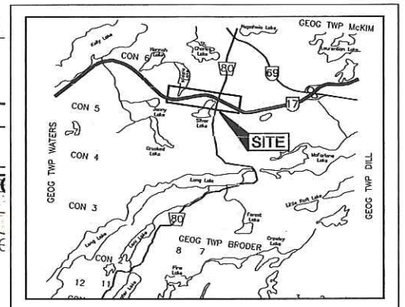
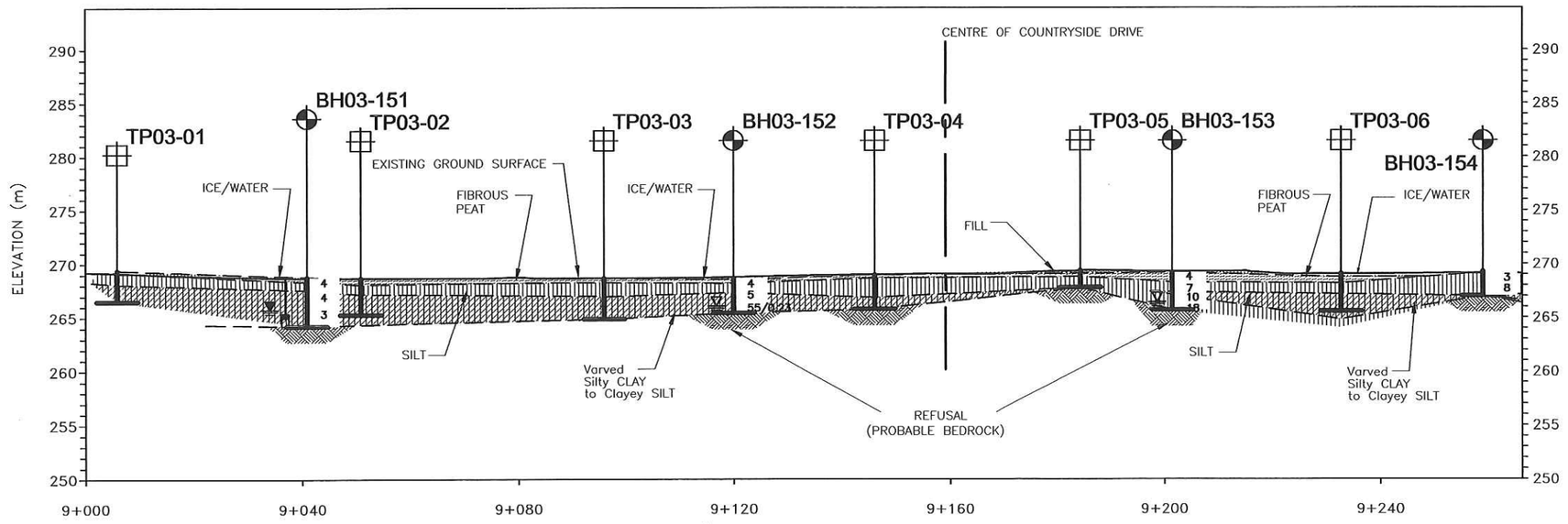
SCALE 1:1000 METRES



KEY PLAN



CHAINAGE (m) ALONG MUNICIPAL ROAD 80 ℄

A-A' PROFILE – 12m LT (EAST) OF ℄ MR 80

HORZ. SCALE
20 0 20 METERS
10 0 10
VERT. SCALE

LEGEND

- Borehole
- Test Pit
- Dynamic Cone Penetration Test
- 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 borehole upon completion of drilling
- WL in piezometer, March 21, 2003

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
BH03-151	268.8	5144547.5	304390.5
BH03-152	268.7	5144468.6	304382.9
BH03-153	269.2	5144388.3	304366.7
BH03-154	269.3	5144331.5	304356.9
TP03-01	269.4	5144583.1	304388.8
TP03-02	268.7	5144538.4	304383.2
TP03-03	268.7	5144493.8	304377.2
TP03-04	269.0	5144443.9	304370.9
TP03-05	269.4	5144406.4	304363.2
TP03-06	269.1	5144359.1	304353.5
TP03-07	269.2	5144309.9	304343.0

NOTES
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

For subsurface information only.

NOTES
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REFERENCE
Digital files provided by EARTH TECH (CANADA INC) LONDON, ONTARIO. "golder_b0648001700 4.dwg, golder_b0648001700 6.dwg and PROFILES.dwg received March 2003 with GOLDER-PROFILES-SEPT22-2003.dwg Provided September 22, 2003.

NO.	DATE	BY	REVISION

Geocres No. 411-157

HWY. 17	PROJECT NO. 021-1147-4		DIST.
SUBM'D.	CHKD. JPD	DATE: FEB., 2004	SITE:
DRAWN: JDR	CHKD. FJH	APPD.	DWG. 2

PLOT DATE: March 01, 2004
FILENAME: T:\Projects\2002\021-1147\REPORT-4\R114702d.dwg

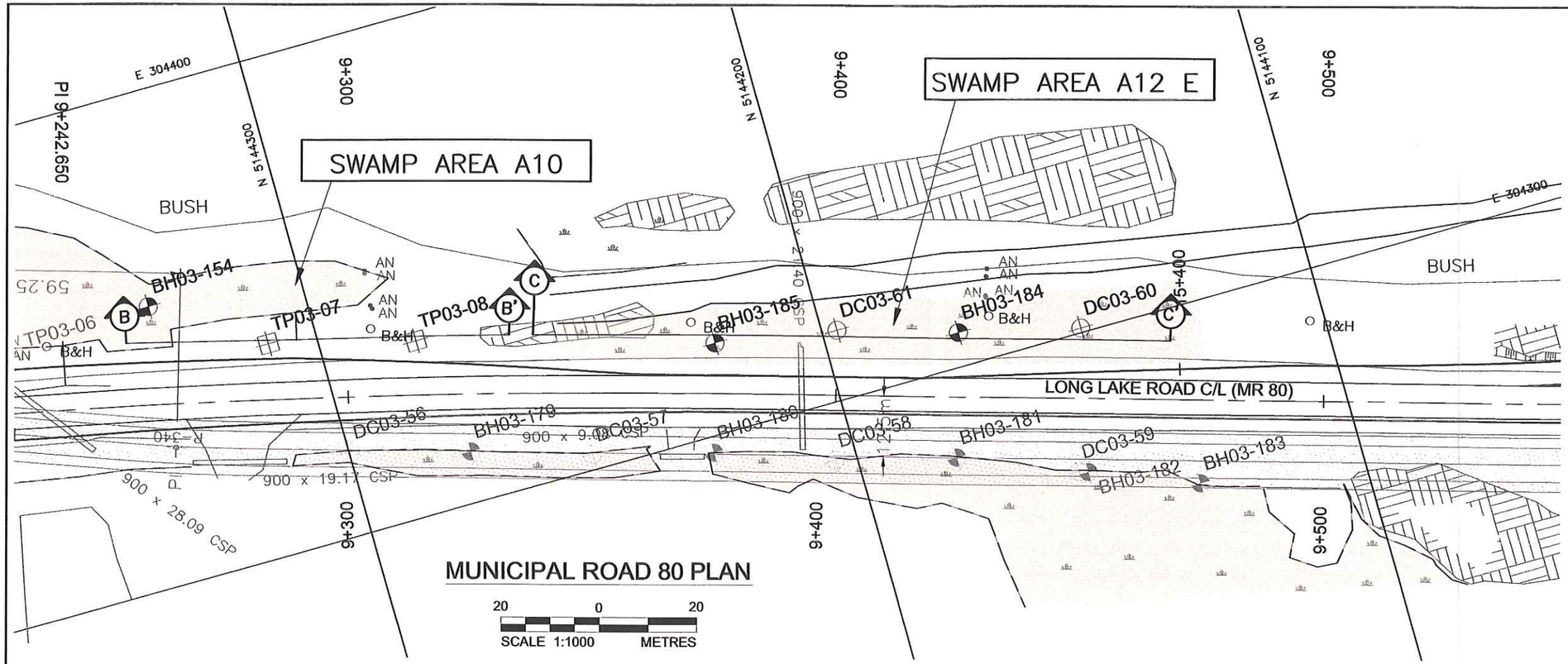
DIST. 54 HWY. 17
CONT No.
WP No. 99-98-00

MUNICIPAL ROAD 80
STATION 9+260 to 9+470
BOREHOLE LOCATION AND SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



MUNICIPAL ROAD 80 PLAN

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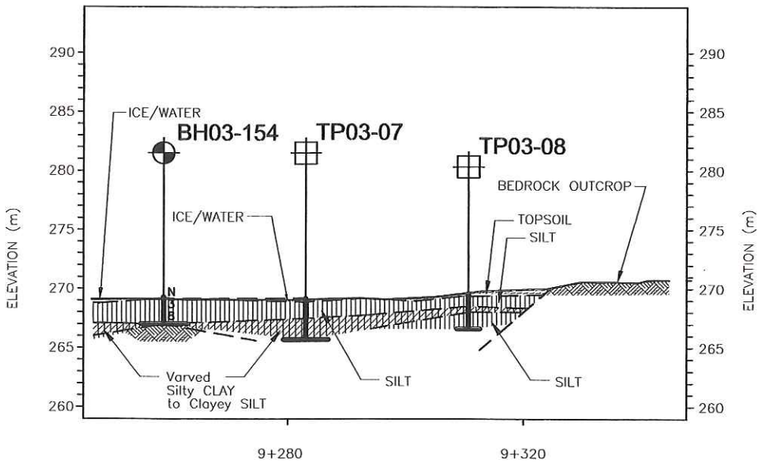
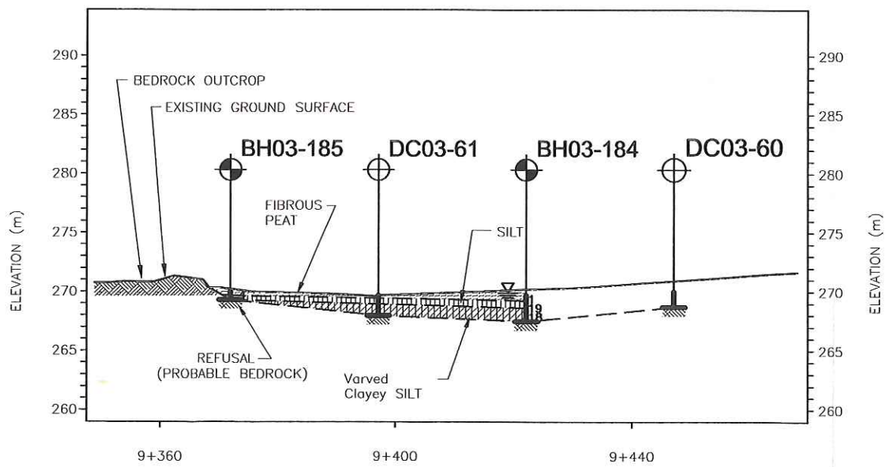
SCALE 1:1000 METRES



CHAINAGE (m) ALONG MUNICIPAL ROAD 80 ℄

B-B' PROFILE – 12m LT (EAST) OF ℄ MR 80

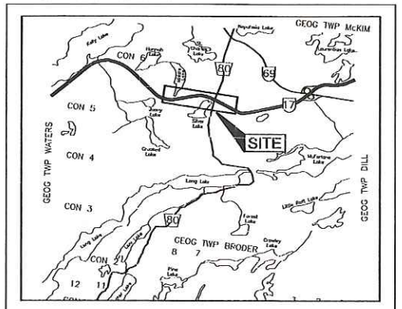
HORZ. SCALE
20 0 20 METERS
10 0 10
VERT. SCALE



CHAINAGE (m) ALONG MUNICIPAL ROAD 80 ℄

C-C' PROFILE – 12m LT (EAST) OF ℄ MR 80

HORZ. SCALE
20 0 20 METERS
10 0 10
VERT. SCALE



KEY PLAN

LEGEND	
⊕	Borehole
⊞	Test Pit
⊕	Dynamic Cone Penetration Test
	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 borehole upon completion of drilling
▼	WL in piezometer

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
BH03-184	269.9	5144173.0	304306.9
BH03-185	270.0	5144221.8	304318.3
DC03-60	270.5	5144148.7	304300.9
DC03-61	269.8	5144196.9	304314.0
TP03-08	269.6	5144280.7	304335.3

NOTES
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For subsurface information only.

NOTES
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REFERENCE
Digital files provided by EARTH TECH (CANADA INC) LONDON, ONTARIO. "golder_b06480017004.dwg, golder_b06480017006.dwg and PROFILES.dwg received March 2003 with GOLDER-PROFILES-SEPT22-2003.dwg Provided September 22, 2003.

NO.	DATE	BY	REVISION

Geocres No. 411-157

HWY. 17	PROJECT NO. 021-1147-4		DIST.
SUBM'D.	CHKD. JPD	DATE: FEB., 2004	SITE:
DRAWN: JDR	CHKD. FJH	APPD.	DWG. 3

PLOT DATE: March 01, 2004
FILENAME: T:\Projects\2002\021-1147\REPORT-411\411-47003.dwg

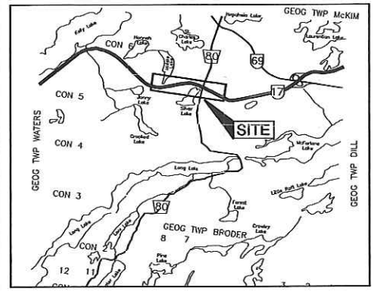
DIST. 54 HWY. 17
CONT No.
WP No. 99-98-00

MUNICIPAL ROAD 80
STATION 9+290 to 9+510
BOREHOLE LOCATION AND SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



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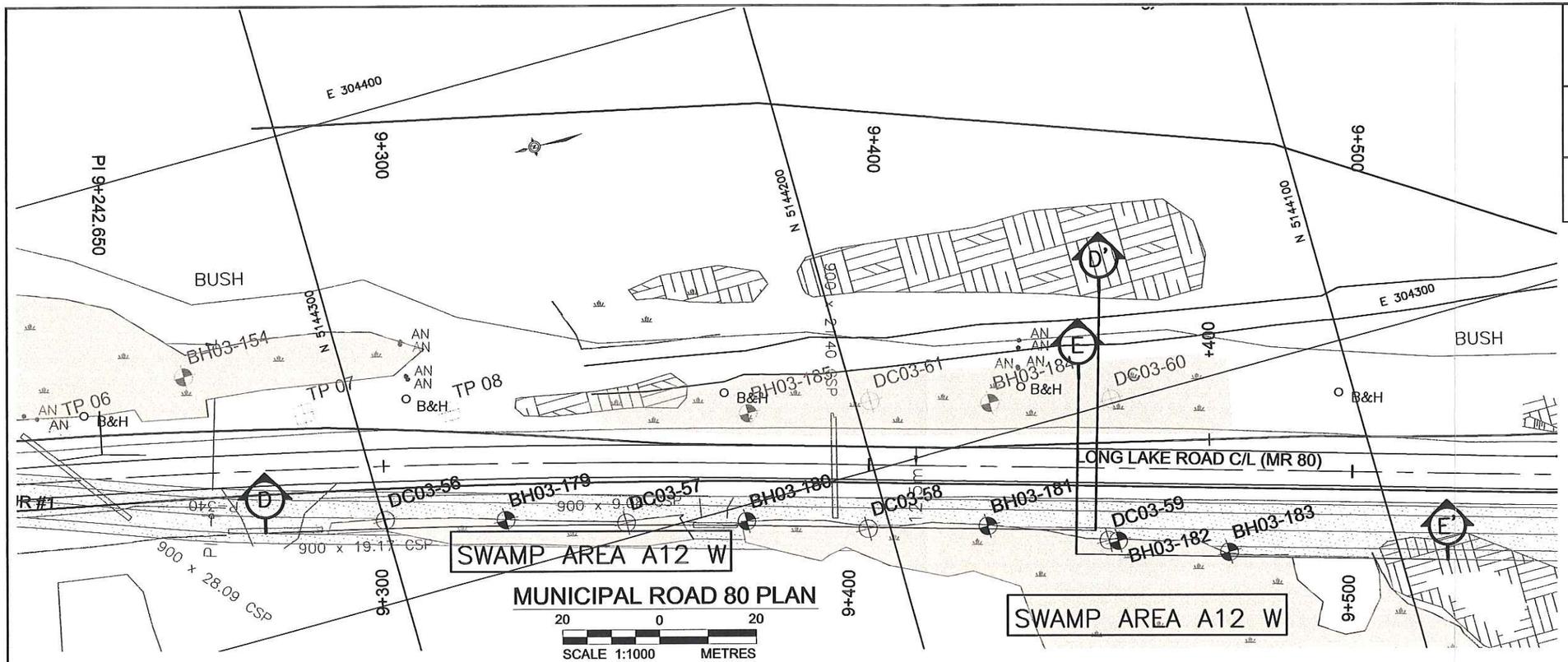

MUNICIPAL ROAD 80 PLAN
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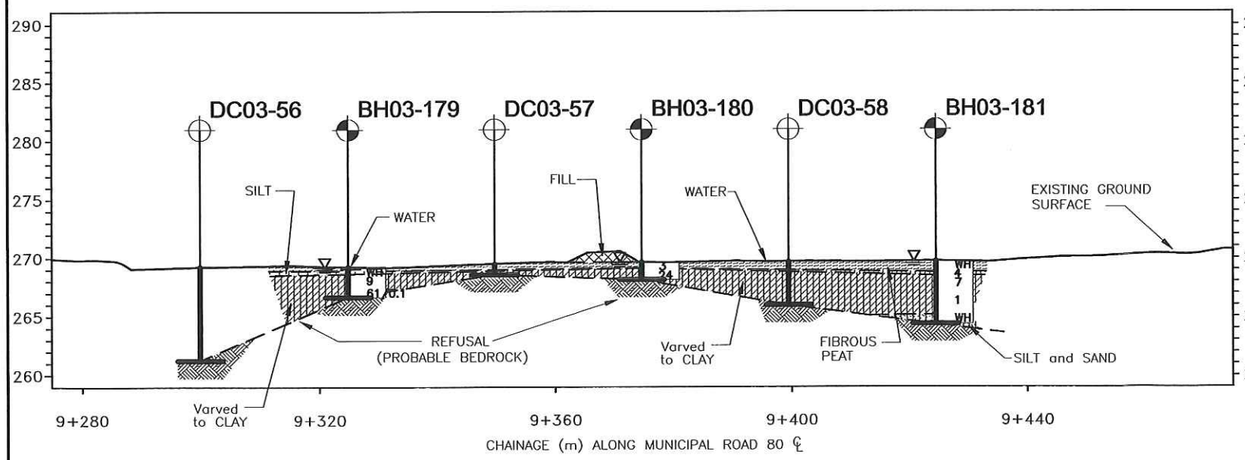
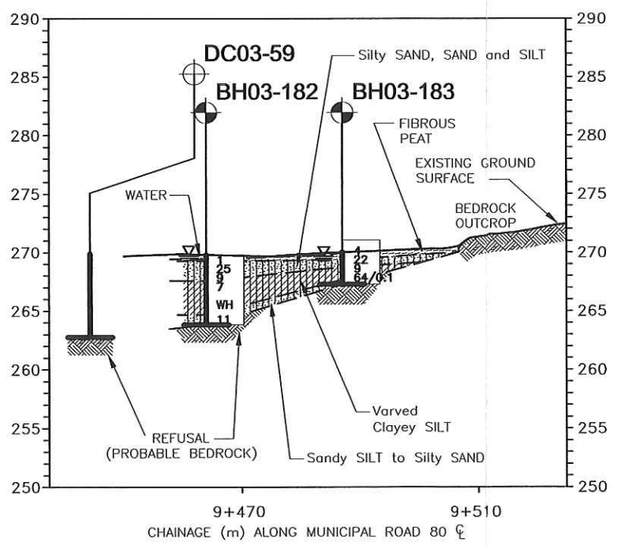
SCALE 1:1000 METRES



CHAINAGE (m) ALONG MUNICIPAL ROAD 80 ℄

D-D' PROFILE – 13m RT (WEST) OF MR 80



CHAINAGE (m) ALONG MUNICIPAL ROAD 80 ℄

E-E' PROFILE – 18m RT (WEST) OF MR 80

LEGEND

- Borehole
- Test Pit
- Dynamic Cone Penetration Test
- 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 borehole upon completion of drilling
- WL in piezometer

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
BH03-179	269.2	5144275.8	304310.4
BH03-180	269.6	5144228.1	304296.7
BH03-181	269.7	5144180.2	304282.3
BH03-182	269.7	5144155.1	304272.1
BH03-183	269.8	5144133.6	304263.7
DC03-56	269.3	5144299.8	304317.1
DC03-57	269.5	5144252.1	304303.1
DC03-58	269.7	5144204.5	304288.5
DC03-59	269.7	5144157.0	304272.8

NOTES
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For subsurface information only.

NOTES
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REFERENCE
Digital files provided by EARTH TECH (CANADA INC) LONDON, ONTARIO. "golder_b06480017004.dwg, golder_b06480017006.dwg and PROFILES.dwg received March 2003 with GOLDER-PROFILES-SEPT22-2003.dwg Provided September 22, 2003.

HORZ. SCALE
20 0 20
METERS
10 0 10
VERT. SCALE

HORZ. SCALE
20 0 20
METERS
10 0 10
VERT. SCALE

NO.	DATE	BY	REVISION

Geocres No. 41I-157

HWY. 17	PROJECT NO. 021-1147-4		DIST.
SUBM'D.	CHKD. JPD	DATE: FEB., 2004	SITE:
DRAWN: JDR	CHKD. FJH	APPD.	DWG. 4

PLOT DATE: March 01, 2004
PLOTTED BY: T:\Projects\2002\021-1147\REPORT-41I11+7004.dwg

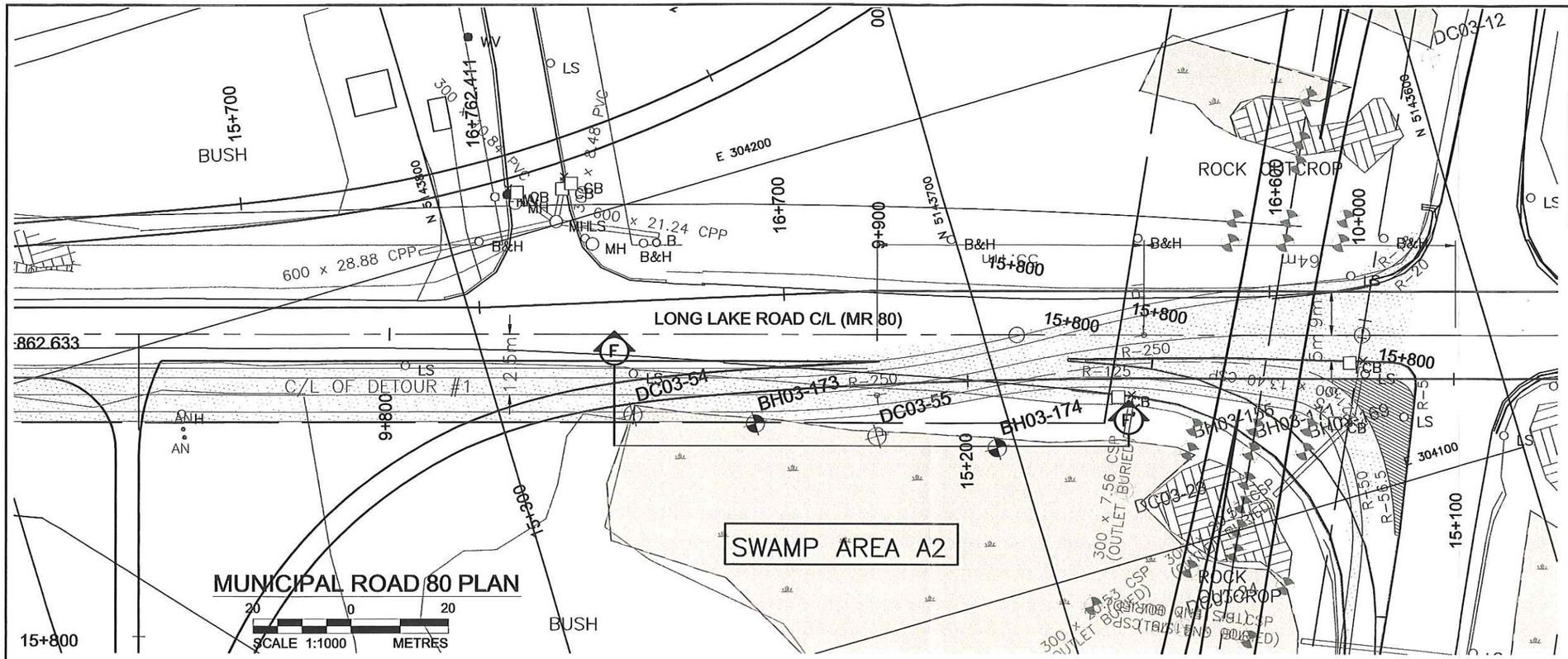
DIST. 54 HWY. 17
CONT No.
WP No. 99-98-00

MUNICIPAL ROAD 80
STATION 9+850 to 9+950
BOREHOLE LOCATION AND SOIL STRATA

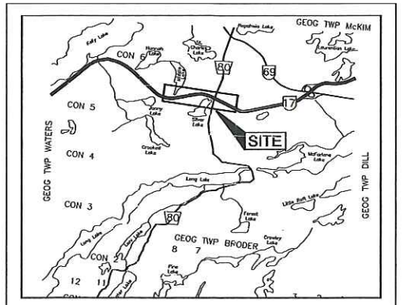
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

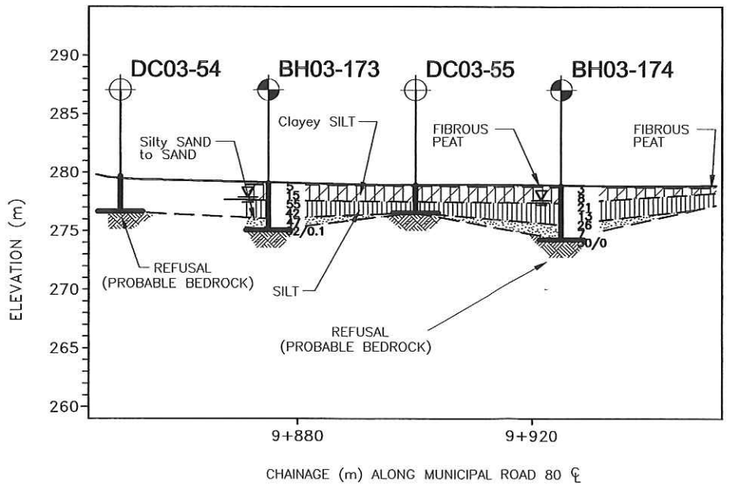
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



MUNICIPAL ROAD 80 PLAN



KEY PLAN



F-F' PROFILE – 23m RT (WEST) OF ℄ MR 80

HORZ. SCALE
20 0 20 METERS
10 0 10
VERT. SCALE

LEGEND

- Borehole
- Test Pit
- Dynamic Cone Penetration Test
- 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 borehole upon completion of drilling
- WL in piezometer

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
BH03-155	279.1	5143666.0	304118.4
BH03-169	282.4	5143643.4	304111.8
BH03-171	282.1	5143653.9	304114.9
BH03-173	279.1	5143750.3	304149.1
BH03-174	278.9	5143703.8	304130.3
DC03-54	279.7	5143773.8	304158.0
DC03-55	278.9	5143727.1	304139.6

NOTES
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For subsurface information only.

NOTES
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REFERENCE
Digital files provided by EARTH TECH (CANADA INC) LONDON, ONTARIO. "golder_b06480017004.dwg, golder_b06480017006.dwg and PROFILES.dwg received March 2003.

NO.	DATE	BY	REVISION

Geocres No. 41I-157

HWY. 17	PROJECT NO. 021-1147-4		DIST.
SUBM'D.	CHKD. JPD	DATE: FEB., 2004	SITE:
DRAWN: JDR	CHKD. FJH	APPD.	DWG. 5

PLOT DATE: February 27, 2004
FILENAME: T:\Projects\2002\021-1147\REPORT-A\R1147003.dwg

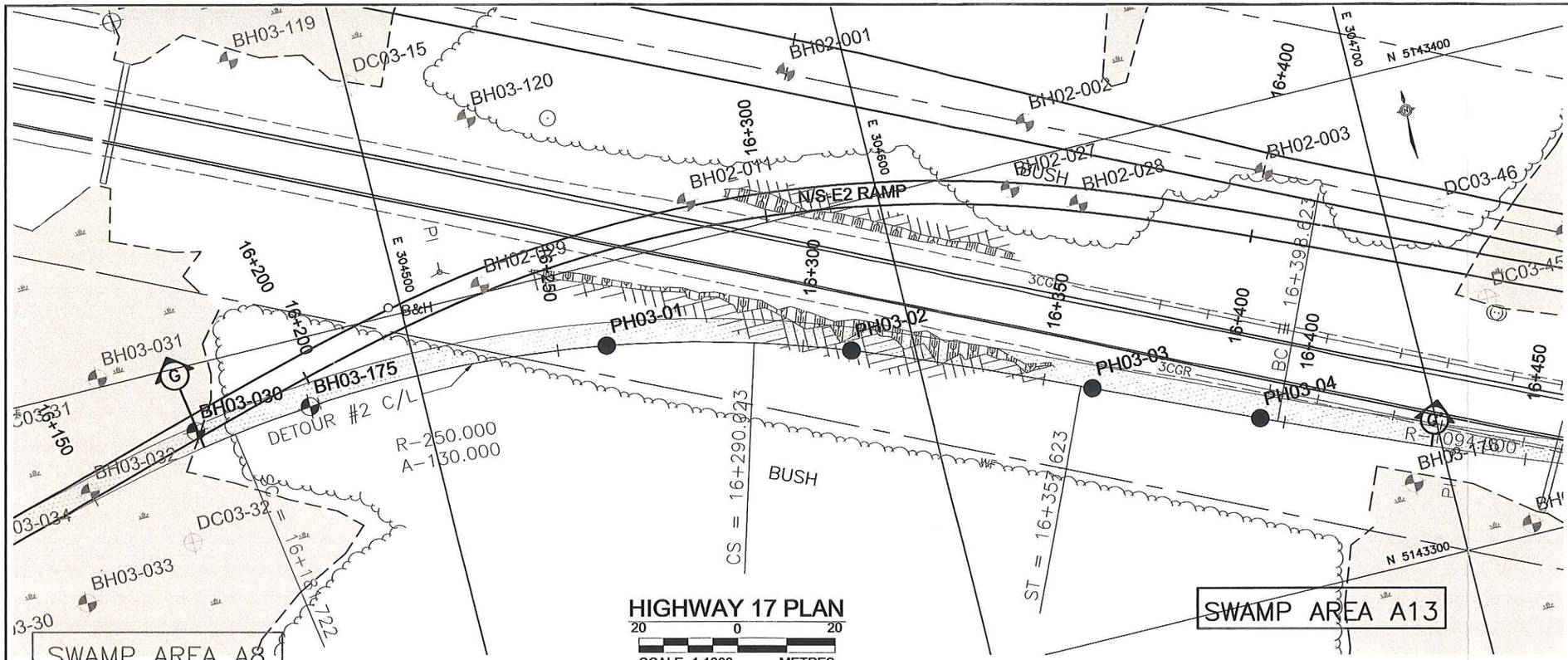
DIST. 54 HWY. 17
CONT No.
WP No. 99-98-00

DETOUR No. 2
STATION 16+180 to 16+430
BOREHOLE LOCATION AND SOIL STRATA

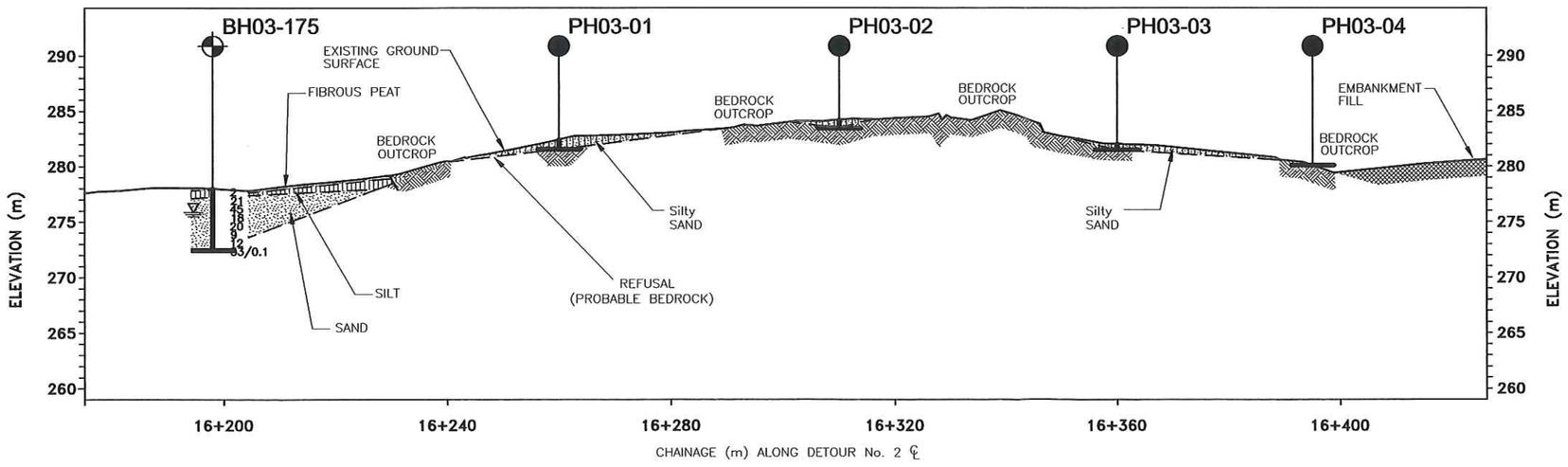
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

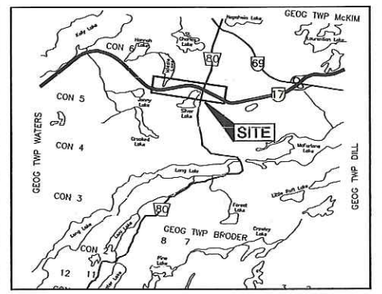


HIGHWAY 17 PLAN
20 0 20
SCALE 1:1000 METRES



G-G' PROFILE – ℄ OF DETOUR #2

HORZ. SCALE
20 0 20
METERS
10 0 10
VERT. SCALE



LEGEND

- Borehole
- Test Pit
- Dynamic Cone Penetration Test
- 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 borehole upon completion of drilling
- WL in piezometer

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
BH03-175	278.0	5143385.9	304477.2
PH03-01	282.4	5143383.4	304539.0
PH03-02	284.2	5143370.4	304587.2
PH03-03	282.0	5143351.0	304633.3
PH03-04	280.1	5143336.8	304665.2

NOTES
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

For subsurface information only.

NOTES
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

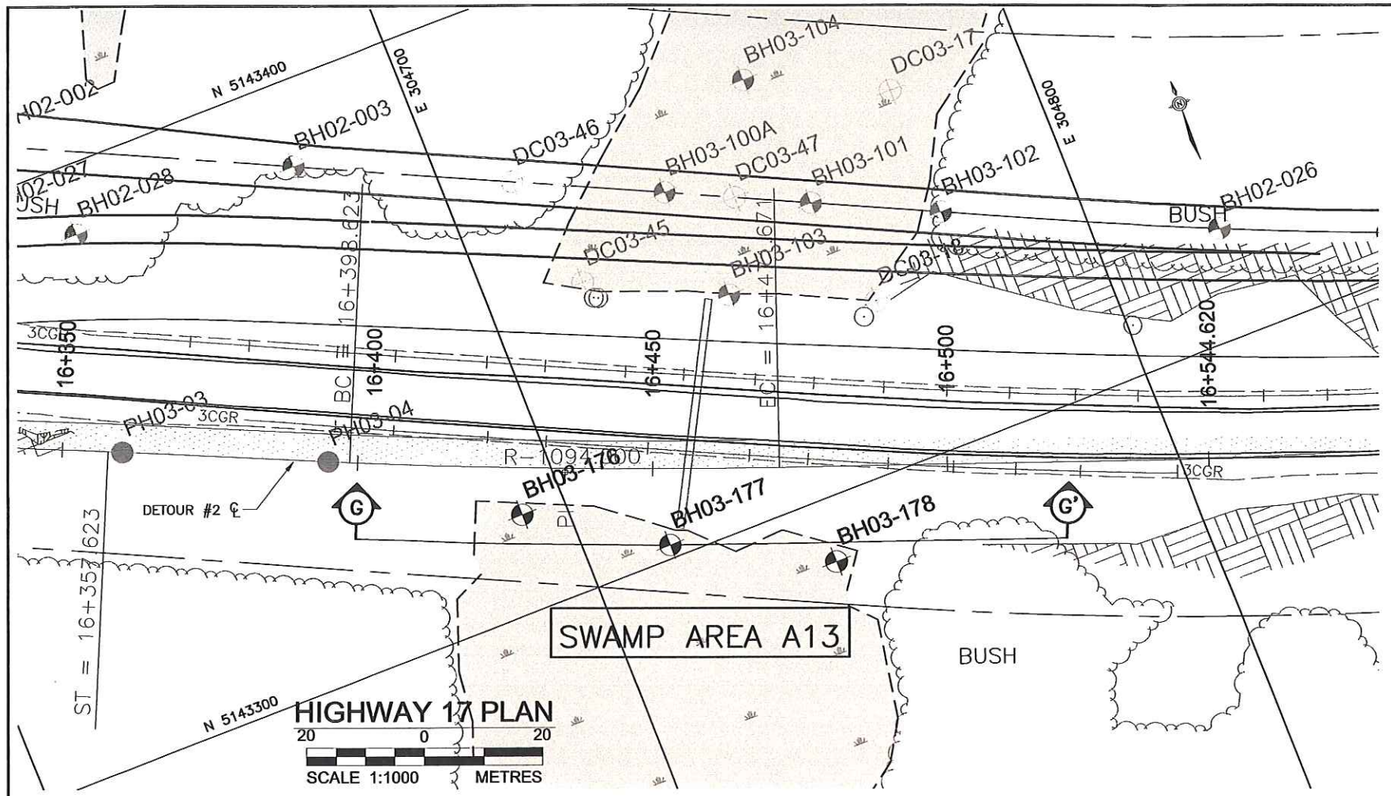
REFERENCE
Digital files provided by EARTH TECH (CANADA INC) LONDON, ONTARIO. "golder_b06480017004.dwg, golder_b06480017006.dwg and PROFILES.dwg received March 2003.

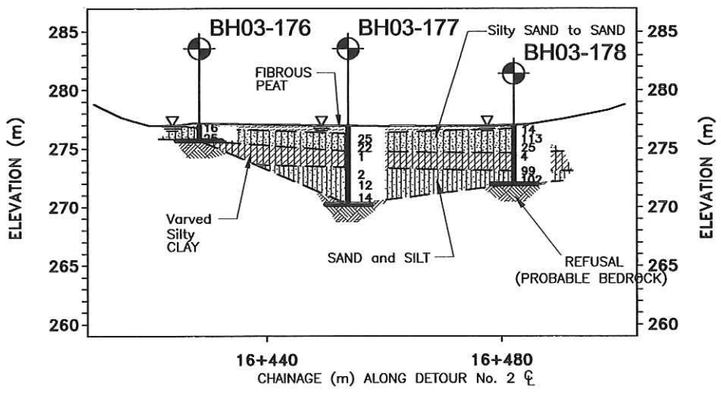
NO.	DATE	BY	REVISION

Geocres No. 41I-157

HWY. 17	PROJECT NO. 021-1147-4		DIST.
SUBM'D.	CHKD. JPD	DATE: FEB., 2004	SITE:
DRAWN: JDR	CHKD. FJH	APPD.	DWG. 6

PLOT DATE: March 01, 2004
FILENAME: T:\Projects\2002\021-1147\REPORT-A\R1147008.dwg





H-H' PROFILE – 13m RT (SOUTH) OF DETOUR #2

NOTES

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REFERENCE

Digital files provided by EARTH TECH (CANADA INC) LONDON, ONTARIO. "golder_b06480017004.dwg, golder_b06480017006.dwg and PROFILES.dwg received March 2003.

HORZ. SCALE
20 0 20
METERS
10 0 10
VERT. SCALE

PLOT DATE: March 01, 2004
FILENAME: T:\Projects\2002\021-1147\REPORT-A\R1147027.dwg

DIST. 54 HWY. 17
CONT No.
WP No. 99-98-00

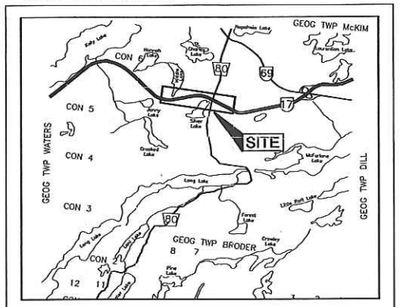
SHEET

DETOUR No. 2
STATION 16+430 TO 16+500
BOREHOLE LOCATION AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



KEY PLAN

LEGEND

- Borehole
- Test Pit
- Dynamic Cone Penetration Test
- 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 borehole upon completion of drilling
- WL in piezometer

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
BH03-176	276.9	5143316.3	304692.5
BH03-177	276.9	5143302.3	304714.1
BH03-178	276.9	5143289.3	304739.4

NOTES

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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For subsurface information only.

NO.	DATE	BY	REVISION

Geocres No. 41I-157

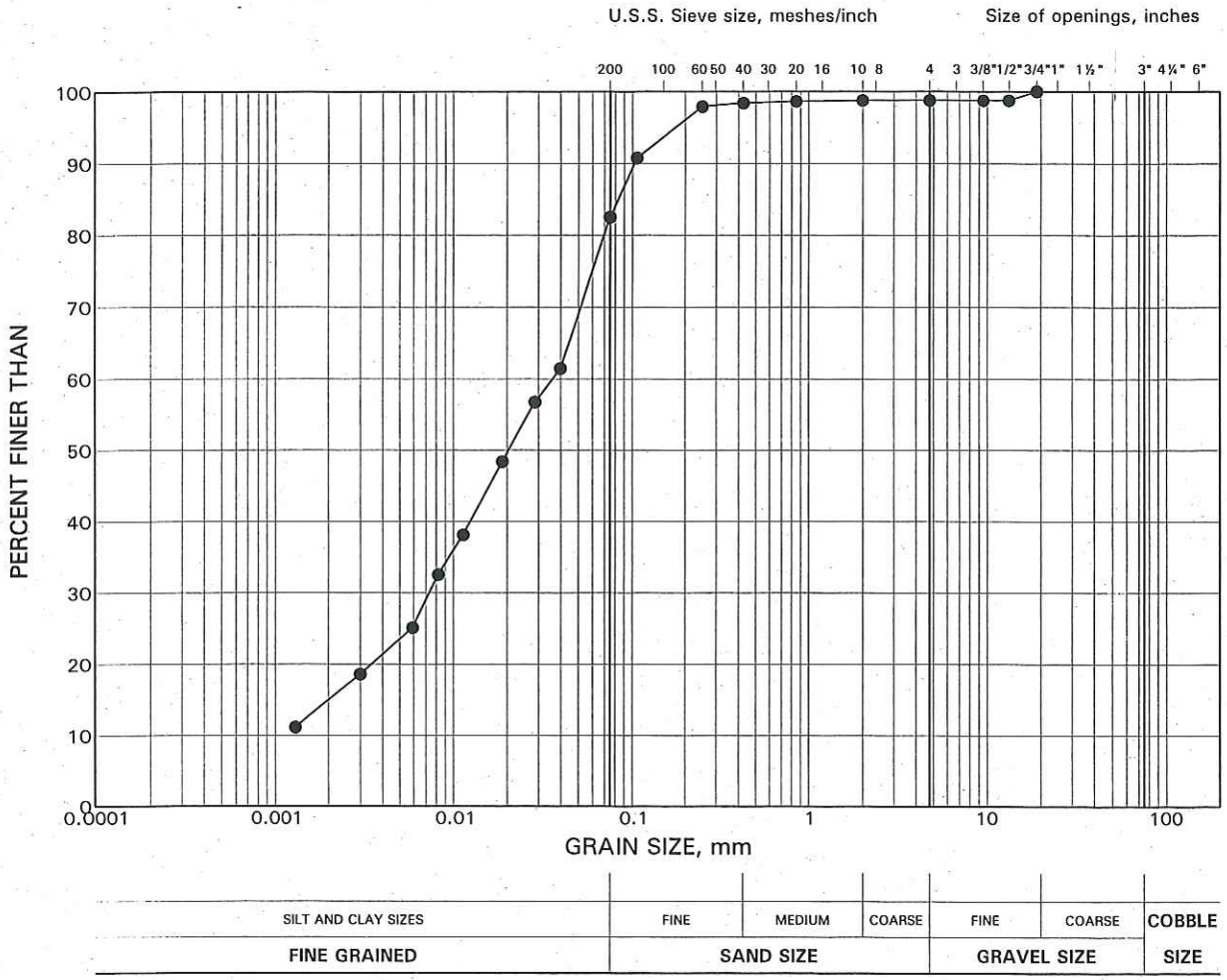
HWY. 17		PROJECT NO. 021-1147-4	DIST.
SUBM'D.	CHKD. JPD	DATE: FEB., 2004	SITE:
DRAWN: JDR	CHKD. FJH	APPD.	DWG. 7

APPENDIX A

LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION
Clayey Silt

FIGURE A1



LEGEND

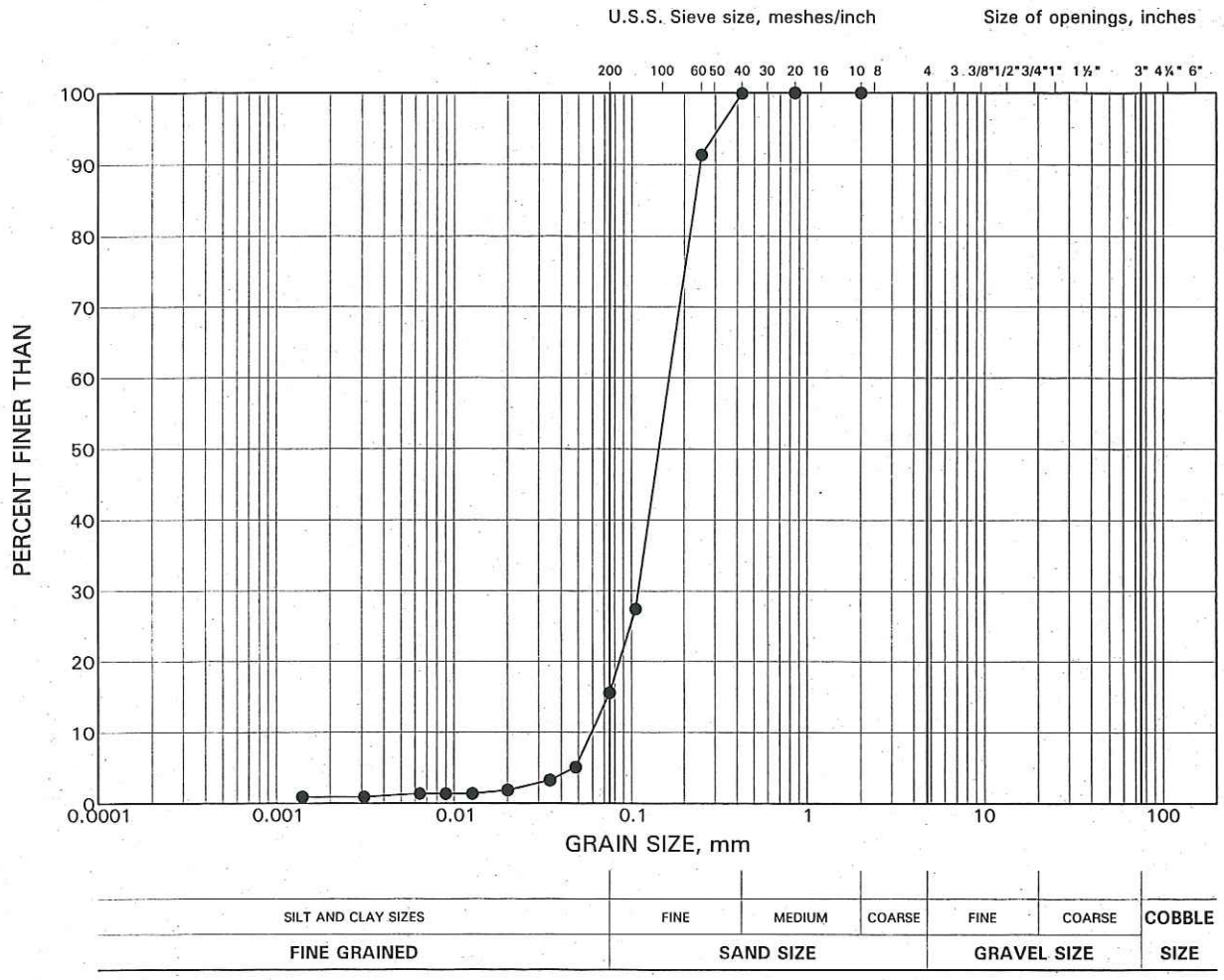
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	03-174	2	277.5

Project 021-1147

Golder Associates

GRAIN SIZE DISTRIBUTION
Sand

FIGURE A2



LEGEND

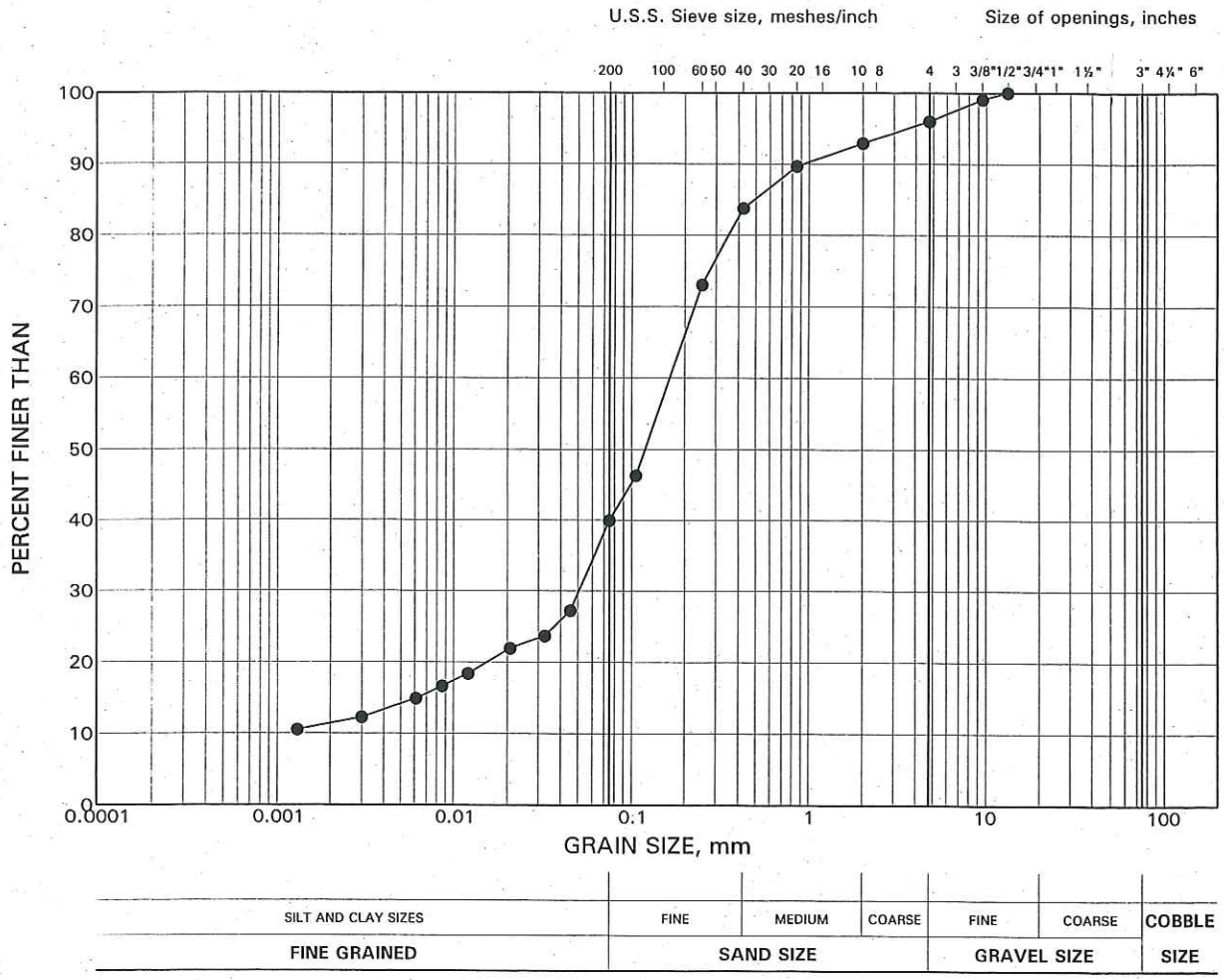
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	03-175	3	275.9

Project 021-1147

Golder Associates

GRAIN SIZE DISTRIBUTION
Silty Sand

FIGURE A3



LEGEND

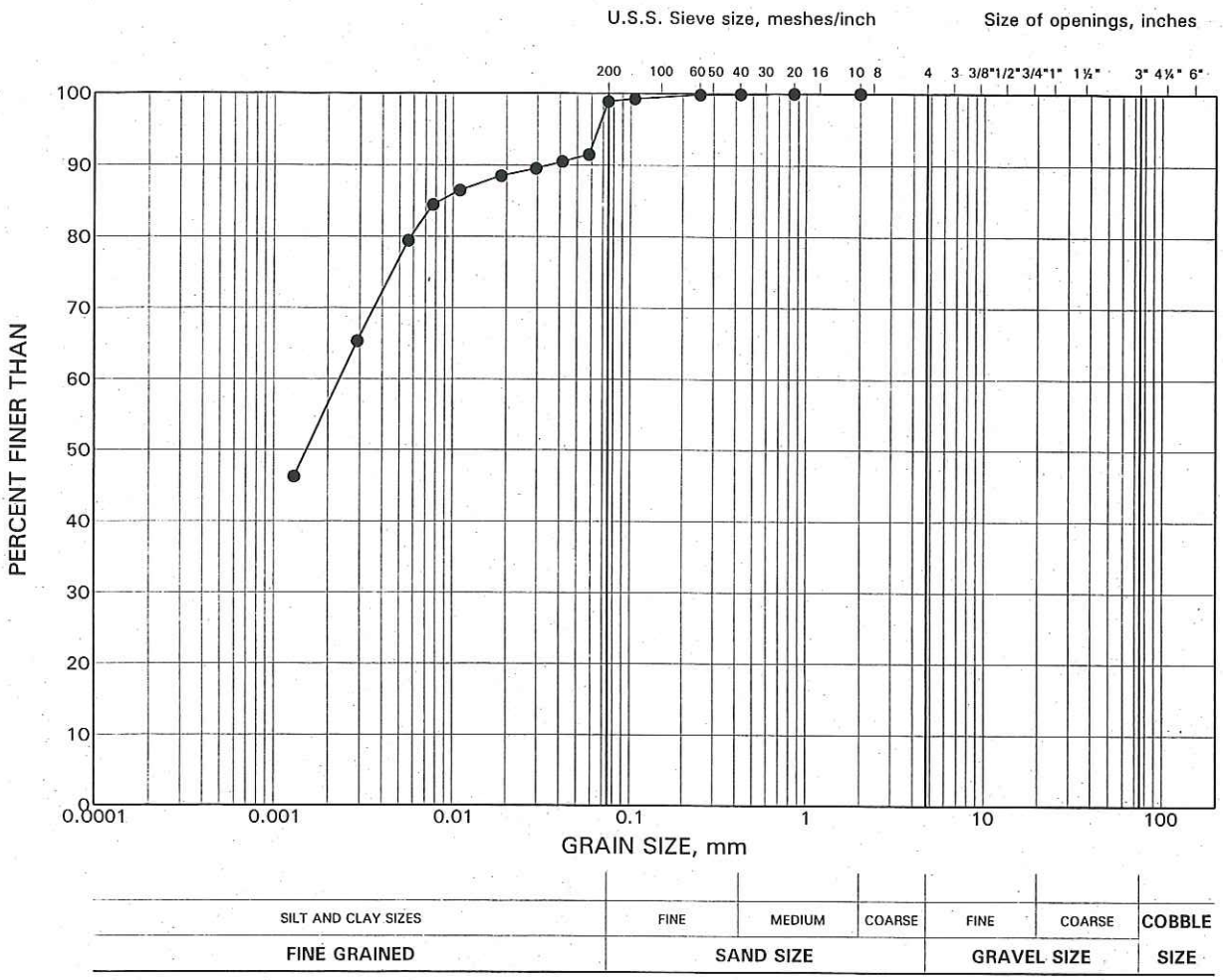
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	03-182	2	268.3

Project 021-1147

Golder Associates

GRAIN SIZE DISTRIBUTION
Clay

FIGURE A4



LEGEND

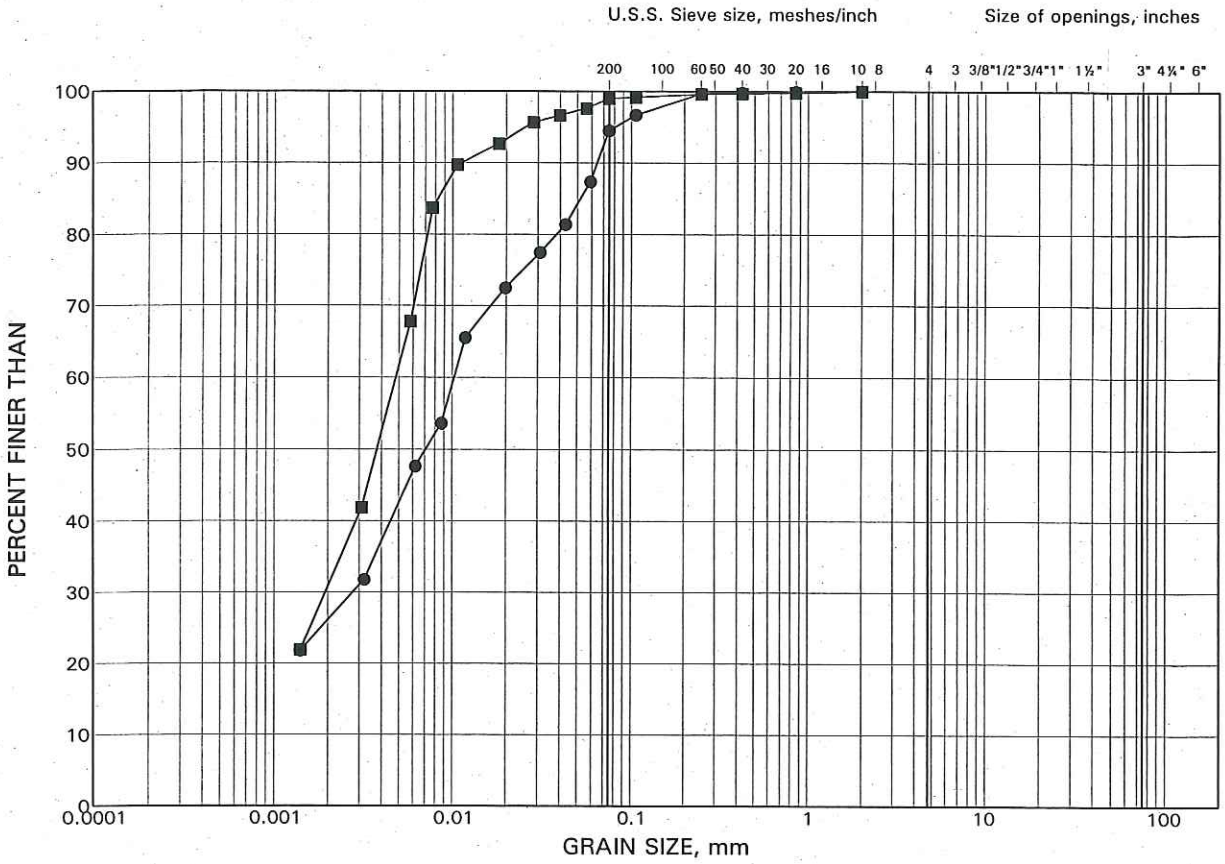
SYMBOL	TEST PIT	SAMPLE	ELEVATION (m)
●	03-3	2	266.2

Project 021-1147

Golder Associates

GRAIN SIZE DISTRIBUTION
Clayey Silt

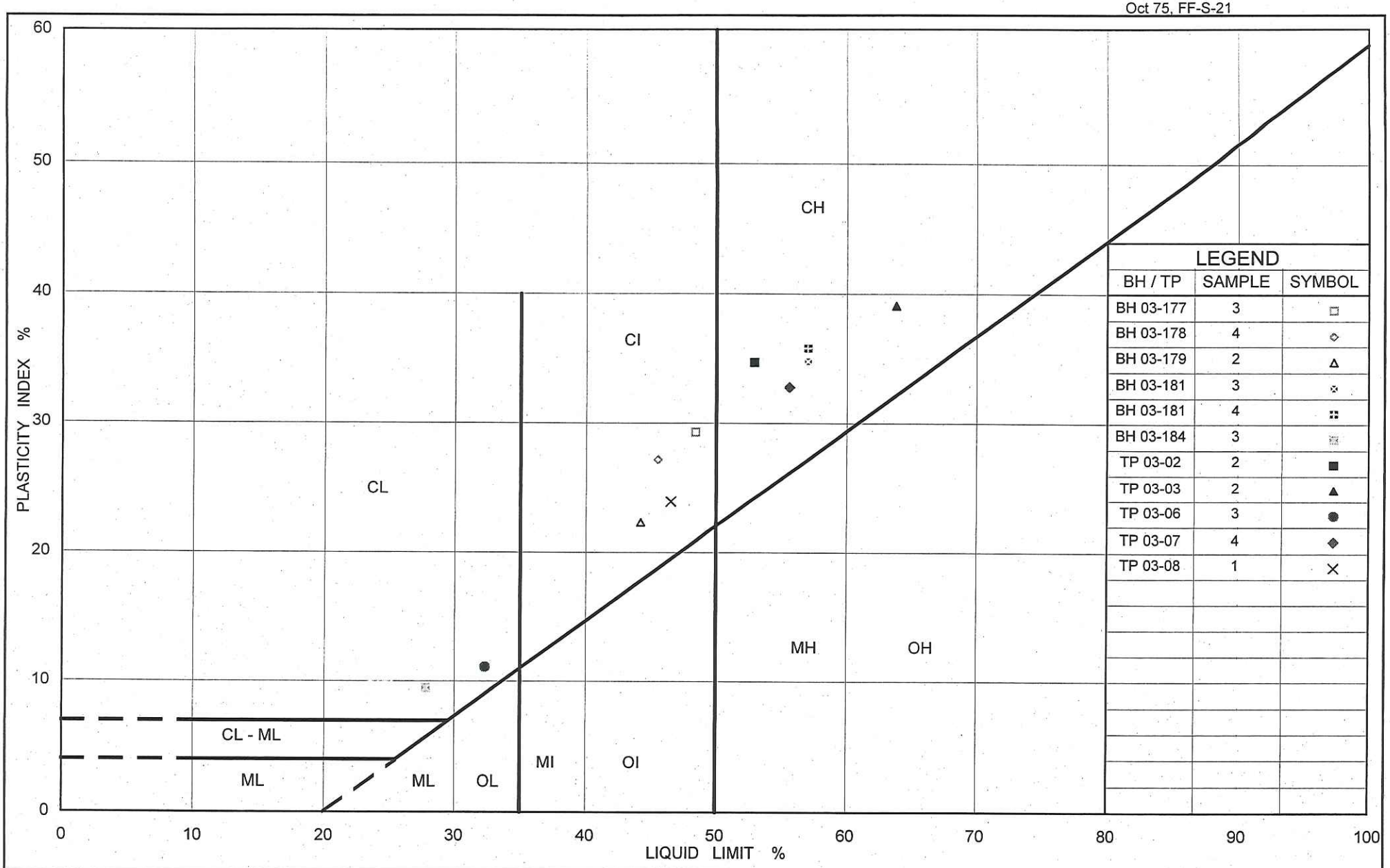
FIGURE A5



LEGEND

SYMBOL	TEST PIT	SAMPLE	ELEVATION (m)
●	03-06	1	267.6
■	03-06	3	265.6

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

FIG No. A6

Project No. 021-1147