



March 2011

REPORT ON

Foundation Investigation and Design Culvert Replacement at Station 16+065.6 Highway 7 from Fowlers Corners Southerly to County Road 28 Peterborough, Ontario G.W.P. 4053-06-00, Site No. 26-215

Submitted to:

D.M. Wills Associates Limited.
452 Charlotte Street
Peterborough, Ontario
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REPORT



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MTO GEOCRES Number: 31D-513



March 30, 2011

Project No. 10-1121-0007 (Rev. No. 1)

Michael Lang, P.Eng.
D.M. Wills Associates Limited
452 Charlotte Street
Peterborough, Ontario
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**FOUNDATION INVESTIGATION AND DESIGN
CULVERT REPLACEMENT AT STATION 16+065.6
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 28
PETERBOROUGH, ONTARIO
G.W.P. 4053-06-00, SITE NO. 26-215, GEOCRETS 31D-513**

Dear Mr. Lang:

We have reviewed the comments provided by the Pavements and Foundations Section of the Ministry of Transportation, Ontario (MTO) that were in a memorandum dated February 14, 2011. The comments, contained in this memorandum pertain to our draft Foundation Investigation and Design Report for the proposed culvert replacement to be constructed as part of the above noted project.

The comments have been discussed and reviewed with our MTO Designated Contact. These comments will be included in our finalized report which will be issued shortly. The associated quality control milestone audit report completed to Milestone 2, submission of final report, will accompany the finalized report. Our responses to the MTO's comments are provided below:

General

Comment 1: MTO GEOCRETS No. 31D-513 has been assigned to the Final Report and Foundation Drawing (BH Locations).

Response 1: GEOCRETS No. 31D-513 will be shown on the front cover and Foundation Drawing of the finalized report.

Comment 2: The Final Foundation Investigation and Design Report and Foundation Drawing must be signed and stamped by two Professional Engineers licensed by PEO, one of which shall be Thurbers Designated Principal Contact identified for MTO Foundation Engineering Projects.

Response 2: The Final Foundation Investigation and Design Report and Foundation Drawings will be signed and stamped by two Professional Engineers licensed by PEO, one of which will be our MTO Designated Principal Contact.

Part 1 – Foundation Investigation Report

Comment 3: Section 4.2.5 (page 6), 1st paragraph – It is specified that “*The sand and gravel content within the cohesive deposit generally decreased with depth.*” However, there are no laboratory test results that confirm this statement. Clarification is required.

Response 3: This sentence will be removed.

Comment 4: Section 4.2.5 (page 6), 2nd paragraph – It is noted that the clayey silt to silty clay is firm to very stiff. However, based on the SPT “N” values noted in the report (i.e., between 3 blows and 16 blows), the clayey silt to silty clay has soft to very stiff consistency. Please clarify.

Response 4: Based on local experience, the nature of the Eastern Ontario clays having blow counts of 3 and 4 blows per 300 millimetres, generally indicate firm constancy.

Comment 5: Section 4.2.5 (pages 6-8) – Why weren’t in situ field vane tests conducted?

Response 5: Based on local experience, typically Eastern Ontario clays that exhibit SPT blow counts in the range of 4 to 16 blows per 300 millimetres will not allow the MTO field vane to turn, which indicates shear strengths greater than 100 kilopascals. No field vanes were attempted at this location, since the SPT blow counts were greater than 3 blows per 300 millimetres within the silty clay deposit.

Comment 6: Section 4.2.6 (page 8) – It is noted that Atterberg Limit Tests were carried out on four samples from the till deposit. However, the table on page 8 shows results from three samples only. Further, based on the results shown on the table for two samples from the Borehole 10-42, these samples should fall within CL-ML zone on the plasticity chart (instead of ML zones). Clarification is required.

Response 6: The text will be changed to three tests. On figure A2, the data point for borehole 06-2, sample 2 will be removed. The results reported in the table round the Plastic and Liquid Limits to the nearest whole number, thus producing a Plastic Index of 3 and 4. Figure A2 was produced using LL and PI values to the nearest tenth of a percent. This figure will be revised using whole numbers. An error was discovered in the table and the plastic limit and plasticity index for borehole 10-42 sample number 12 has been revised.

Comment 7: Appendix A, Record of Borehole No. 10-42 – The consistency of the upper layer of silty clay should be stiff based on the “N” value (i.e., 9 blows) noted on the borehole log. Similarly, the consistency of the lower silty clay and the silty clay to clayey silt should be firm to stiff and firm respectively based on the “N” values specified on the borehole log. Please clarify.

Response 7: Based on local experience, the nature of the Eastern Ontario clays having a blow count of 9 blows per 300 millimetres is indicated as having a stiff to very stiff consistency.

Comment 8: Appendix A, Record on Borehole No. 10-42 – Groundwater level is not specified in the borehole log. Please clarify.

Response 8: The water level will be added to the borehole record.

Comment 9: Appendix A, Record of Borehole No. 10-43 – The consistency of the silty clay should be firm to stiff based on “N” values noted on the borehole log. Please clarify.

Response 9: See Response 7, above.

Comment 10: Appendix A, Record of Borehole No. 10-44 – The consistency of the clayey silt is not noted on the borehole log. The consistency of the silty clay at depths between 3.7 m and 5.2 m below ground should be stiff (instead of stiff to very stiff). Clarification is required.

Response 10: The consistency will be added to the Record of Borehole No. 10-44.

Comment 11: Appendix A, Record of Borehole No. 10-45 – The consistency of the clayey silt should be very stiff based on the “N” value noted on the borehole log. The consistency of the silty clay and the silty clay to clayey silt is not specified on the borehole log. The relative density of the silty sand should be loose to compact based on the “N” values noted on the borehole log. Clarification is required.

Response 11: The consistency of the silty clay layers will be added and the relative density of the lower silty sand will be revised

Part 2 – Foundation Design Report

Comment 12: Section 6.2.1.1 (page 14) – It is specified that the factored ULS and SLS values provided for the box culvert are for preliminary design. The recommendations for this assignment should be given for the detail design. Please clarify.

Response 12: The wording will be revised to detailed design.

Comment 13: Section 6.2.1.2 (page 15) – It is specified that the factored ULS and SLS values provided for the box culvert are for preliminary design. The recommendations for this assignment should be provided for the detail design. Please clarify.

Response 13: The wording will be revised to detailed design.

Comment 14: Section 6.2.1.2 (page 15) – Based on the consistency of the founding soils, the recommended factored ULS and SLS values seem high.

Response 14: Based on the SPT N values in the weathered crust of the silty clay, the factored ULS and SLS values are considered appropriate. To further support these design values, the preconsolidation pressure indicated from the oedometer testing in borehole 06-2, which is located just below the proposed foundation level, indicates that the clay has been preconsolidated by about 290 kilopascals. The SLS value of 200 kilopascals will be within the preconsolidation range of this soil.

Comment 15: Section 6.2.1.2 (page 15) – SP 902S01 is no longer valid. This should be replaced with OPSS902.

Response 15: SP 902S01 will be replaced with OPSS 902.

Comment 16: Section 6.2.1.3 (page 16) – The apparent cohesion value for the Native clayey silt to silty clay seems high. Please clarify.

Response 16: See Response 5, above.

Comment 17: Section 6.5 (page 21) – The bulk unit weights for the clayey silt to silty clay and Till are inconsistent with those noted under Section 6.2.1.3. Further, there's no indication in the report that bedrock was encountered. Clarification is required.

Response 17: The unit weights for the various soils throughout the report will be standardized.

Comment 18: Section 6.7 (page 21) – The report specified that embankment construction should be in accordance with Special Provision No. 206S03. The SP specifies that stripping is required for embankments of 1.2 m or less in height. Please confirm if this meets the project requirements. If not please provide proper wording for NSSP.

Response 18: The native upper soils contain organics which will cause long term settlements and possible instabilities, therefore these soils need to be removed. Therefore this special provision is appropriate for this structure.

Comment 19: Section 6.7.6 (page 24) – Special Provision No. 105S19 is no longer valid. This should be replaced with OPSS539.

Response 19: Special Provision 105S19 will be replaced with OPSS 539.

We trust these responses sufficiently address MTO's comments. If you have any questions or comments, please contact us.

GOLDER ASSOCIATES LTD.



Bruce D. Goddard, P.Eng.
Senior Geotechnical Engineer



Fintan J. Heffernan, P.Eng.
Designated MTO Contact

BDG/FJH/tm

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APPENDIX B

Record of Borehole Sheets and Detailed Laboratory Test Results Previous Investigation



FOUNDATION INVESTIGATION

PART A

FOUNDATION INVESTIGATION REPORT
DETAILED DESIGN
CULVERT REPLACEMENT AT STATION 16+065.6
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 28
PETERBOROUGH, ONTARIO
G.W.P. 4053-06-00, SITE NO. 26-215



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by D.M. Wills Limited on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with proposed highway operational improvements and future four laning of Highway 7 from Fowlers Corners Southerly to County Road 28 in Peterborough, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P0-1121-0007, dated February 2010, that forms part of the consultant's agreement (GWP 4053-06-00/245-00-01/345-01-01). This report addresses the detailed foundation investigation carried out for the proposed culvert replacement at station 16+065.5 (site no. 26-215, WP 4053-06-00) as part of the highway 7 improvement project. Detailed foundation investigation and design services are required for a total of four structures (i.e. Jackson Creek Bridge, CNR overhead site, and two structural culvert sites) and one high fill section in five separate reports for this project.

The purpose of this investigation is to establish the subsurface conditions at the proposed culvert replacement site by borehole drilling, in situ testing and laboratory testing on selected samples. A plan drawing of the existing culvert location was provided to Golder by D.M. Wills in July 2010.

The current investigation was supplemented with information from a previous investigation at this site, as follows:

- Golder's report titled "Foundation Investigation and Design, Preliminary Design, Culvert Replacement at Station 16+065.5, Southerly to County Road 28, Peterborough, Ontario, G.W.P. 73-99-00, Site No. 26-215", dated July 2007.



2.0 SITE DESCRIPTION

The site is located on Highway 7, approximately 125 m south of Brown Line (11th Line) in Peterborough, Ontario (see key plan on Drawing 1). The existing highway in this area has two lanes, one lane each for northbound and southbound traffic.

Within the existing MTO right-of-way, the site generally consists of the raised highway embankment and developed commercial properties on the west side of the highway and low-lying grassy area on the east side of the highway. The west side of the highway is generally flat with gently sloping grades that lead to storm drains and pipes that empty into the Trout Creek tributary, just downstream of the existing culvert outlet. The east side of the highway embankment consists of grass-covered side-slopes (sloped at about 1.5H:1V to 2H:1V) that lead into ditches that drain into the upstream side of the culvert. The existing culvert structure consists of an open footing concrete culvert measuring about 3.7 m wide x 1.5 m high x 18.4 m long. The assumed founding level of the footings is about elevation 197.0 metres. The existing culvert structure allows passage of the Trout Creek tributary from east to west beneath Highway 7 in this area. It is proposed that the existing structure is to be replaced as part of the highway intersection improvements and five-laning (including a turning lane) of Highway 7 at this location.

Based on the drawing provided to us by D.M. Wills titled "Springville Culvert, General Arrangement", dated November 2010, the existing Highway 7 road surface at the site is at about Elevation 200.4 m, the existing culvert obvert is at about Elevation 199.5 m, and the tributary creek bed within the existing culvert is at about Elevation 198.3 m.



3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for this culvert investigation was carried out between September 30 and October 6, 2010 during which time four (4) boreholes were advanced. The boreholes from the current study, numbered 10-42 to 10-45, inclusive, were advanced at the approximate locations shown in plan on Drawing 1. The approximate locations of the boreholes from the 2007 investigation are also shown on Drawing 1.

The current field investigation was carried out using a Diedrich D-50 track mounted drill rig supplied and operated by Walker Drilling Limited of Utopia, Ontario. The boreholes were advanced using 105 mm inside diameter (I.D.) hollow stem augers. Soil samples were obtained at intervals of 0.75 m in depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Boreholes 10-44 and 10-45 were terminated at 8.2 m depth below ground surface, while boreholes 10-42 and 10-43 were terminated at 10.5 m depth below ground surface. All boreholes were backfilled with a mixture of bentonite hole plug and soil cuttings to seal the boreholes from artesian groundwater.

The groundwater conditions in the open boreholes were observed during the drilling operations and a piezometer was installed in borehole 10-43 to permit monitoring of the groundwater level at this location. The piezometer consisted of 51 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. The boreholes and annulus surrounding the piezometer pipe were backfilled to the surface with bentonite pellets in accordance with Ontario Regulation (O.Reg.) 903 amended to O.Reg. 128/03 of the Ontario Water Resources Act. The piezometers will require decommissioning prior to or during construction in accordance with O.Reg 903. The piezometer 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 by a member of our engineering technical staff, who located the boreholes, arranged for the clearance of underground services, 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 Ottawa geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on select samples.

Two samples of soil, one from borehole 10-42 and one from borehole 10-43, were submitted to Exova Laboratories Ltd. in Ottawa, Ontario for chemical analysis related to potential corrosion of buried steel elements and potential sulphate attack on buried concrete elements.

The borehole locations were staked in the field by Golder relative to on-site features. Upon completion of drilling operations, the borehole locations (i.e. MTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to geodetic datum) were surveyed by Golder Associates personnel using an R-8 Trimble GPS unit and are summarized below and on Drawing 1. The locations and ground surface elevations of the boreholes from the previous 2007 Golder investigation are also included below and on Drawing 1.



FOUNDATION INVESTIGATION

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-1	4900609.0	392478.2	198.6
06-2	4900606.9	392441.9	200.2
10-42	4900603.6	392473.3	200.2
10-43	4900608.7	392449.0	200.1
10-44	4900635.3	392462.8	200.1
10-45	4900576.5	392483.1	200.1



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, the study area for this assignment lies within the physiographic region known as the Peterborough Drumlin Field.

The surficial soils in the Peterborough Drumlin Field consist of drumlinized till. Toward the southwestern portion of this physiographic region, near the Oak Ridges Moraine, the till is typically sandy. Some of the drumlins in this area have shallow coverings of silt and fine sand, between about 0.5 m and 2.5 m in thickness. “Wave-washed” drumlins, with exposed bouldery surfaces, are also present near the Simcoe Lowlands immediately south and east of Lake Simcoe. Localized swampy areas and deposits of silt, clay and peat are found in the low-lying areas between drumlins.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and in Appendix A following the text of this report. The borehole records and laboratory testing results from the 2007 Golder investigation are contained in Appendix B.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes at the culvert replacement and road widening location is shown on Drawing 1.

In general, the subsoils at the culvert replacement site consist of a surficial roadway embankment fill, underlain by a layer of clayey silt to silty clay. The clayey silt to silty clay is underlain by a deposit of glacial till. At borehole 06-1, which was drilled on the east side of the highway and at the toe of the embankment, a surficial layer of peat was encountered above the clayey silt to silty clay deposit. At borehole 10-43, organic silt underlain by clayey silt was encountered sandwiched between the embankment fill and the silty clay to clayey silt deposit. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Pavement

Asphaltic concrete was encountered at the pavement surface in boreholes 10-42, 10-43, 10-44 and 10-45. The thickness of the asphaltic concrete ranged from about 100 to 200 at the borehole locations.

Granular base and subbase fill materials were encountered beneath the asphaltic concrete in boreholes 10-42, 10-44 and 10-45. Borehole 06-2 was advanced beyond the edge of the asphalt pavement, and roadway granular base and subbase fill materials were encountered beneath the surficial topsoil layer at this location. The granular base and subbase ranged from about 500 to 900 mm in thickness and consisted of sand and gravel.

¹ Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.



Borehole 10-43 was advanced at the edge of the parking area of the nearby gas bar. The base material encountered within borehole 10-43 consists of 0.3 m of sand fill.

4.2.2 Topsoil / Peat / Organic Soils

A surficial layer of peat, about 0.5 m thick, was encountered in borehole 06-1 at the toe of the highway embankment. A surficial topsoil layer was encountered, about 0.2 m thick, in borehole 06-2 at the crest of the embankment. In borehole 10-43, a layer of organic silt, about 0.3 m thick, was encountered immediately below the embankment fill. In all of the other boreholes, the upper portion of the clayey silt to silty clay deposit contained a small amount of organic matter.

4.2.3 Fill

Fill was encountered below the pavement structure in borehole 10-43. This borehole was advanced at the edge of the parking area and near the inlet of the existing culvert. The fill varies in composition from silty sand to sandy silt with varying amounts of gravel, clay and organic matter. The top of the fill was encountered at a depth of 0.4 m (Elevation 199.7 m) and was 1.6 m thick.

Two Standard Penetration Test (SPT) 'N' values recorded within the fill were 2 and 5 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on three samples of the fill ranged from 11 to 30 per cent. A grain size distribution curve on a selected sample of the fill is shown on Figure A3 in Appendix A.

4.2.4 Upper Silty Sand

Underlying the sand and gravel fill, a layer of silty sand containing trace gravel and clay was encountered in borehole 06-2. The top of the silty sand layer was encountered at a depth of 1.8 m (Elevation 198.4 m) and was about 0.3 m thick.

One Standard Penetration Test (SPT) 'N' value recorded within the silty sand layer measured 2 blows per 0.3 m of penetration, indicating a very loose relative density.

4.2.5 Clayey Silt to Silty Clay

A clayey silt to silty clay layer containing trace to some sand and gravel was encountered below the pavement structure, topsoil, peat, organic silt and silty sand in all boreholes (i.e., 06-1, 06-2, 10-42 to 10-45, inclusive). The cohesive layer contained sand and silt seams throughout. The top of the clayey silt to silty clay deposit was encountered at a depth of about 0.5 m and 2.3 m (Elevation 197.8 to 199.5 m) and the deposit ranged from about 3.3 to 4.5 m in thickness. The colour of the cohesive deposit generally transitioned from brown to grey with depth.

Standard Penetration Testing (SPT) 'N' values recorded within the clayey silt to silty clay deposit ranged between 3 blows and 16 blows per 0.3 m of penetration, indicating a firm to very stiff consistency. The 'N' values within the clayey silt to silty clay generally decreased with depth, below the upper crust.

The natural water content measured on select samples of the clayey silt to silty clay layer ranged between about 17 per cent and 31 per cent. Grain size distribution curves on select samples of the clayey silt to silty clay deposit are shown on Figure A4 in Appendix A.



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The grain size distribution curve for sample 4 from borehole 10-42 indicated that the sample had a significant amount of sand. Upon further review, it was determined that the portion of the sample taken for the grain size analysis included a sand layer, which skewed the results of the analysis. The remaining portion of the sample was visually identified as clayey silt.

The results of Atterberg Limits testing carried out on 12 samples of the clayey silt to silty clay deposit are illustrated on the plasticity chart on Figure A1 in Appendix A. The test results are summarized below and indicate the clayey silt to silty clay is of low to medium plasticity.

Borehole	Sample	Elevation (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
06-1	4	194.9 – 195.6	21	14	7
06-2	4	196.6 - 197.2	40	20	20
06-2	5	195.0 – 195.6	24	15	9
10-42	3	197.3 – 197.9	27	14	13
10-42	4	196.5 – 197.2	29	17	12
10-42	6	195.0 – 195.6	21	16	5
10-43	3	197.2 – 197.8	23	12	11
10-43	4	196.4 – 197.1	40	19	21
10-43	6	194.9 – 195.5	22	14	8
10-44	3	197.2 – 197.8	29	14	15
10-44	5	195.7 – 196.3	33	16	17
10-45	3	197.2 – 197.8	31	18	13

A laboratory consolidation test was carried out on a single specimen of the clayey silt to silty clay deposit obtained from a Shelby tube sample. The results are summarized below.

Borehole/ Sample No.	Sample Depth/Elev.	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
06-2, Sa#4	3.3 m/196.9 m	51	341	290	7	0.19	0.02	0.74	4.5×10^{-2}

Note: * For stress range of $40 \leq \sigma_v' \leq 160$ kPa

Where: σ_{vo}' is the effective overburden pressure in kPa
 σ_p' is the apparent preconsolidation pressure in kPa
OCR is overconsolidation ratio
 e_o is initial void ratio
 C_c is the compression index
 C_r is the recompression index
 c_v is the coefficient of consolidation in cm²/s



Based on the test results, the silty clay sample used for the consolidation test (i.e. from borehole 06-2, Sa#4) is considered to be over-consolidated. A bulk unit weight of about 19.6 kN/m^3 and a specific gravity of 2.76 were measured on the consolidation test specimen. The consolidation test results are shown on Figures B1 to B4 (inclusive) in Appendix B.

4.2.6 Till

A deposit of till was encountered below the clayey silt to silty clay deposit in boreholes 10-42 to 10-45, inclusive. In boreholes 06-1 and 06-2 from the previous 2007 investigation, this deposit was identified as the lower silty sand. Till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand with trace to some clay. The top of this deposit was encountered at depths ranging from 4.3 m (Elevation 194.3 m) to 5.8 m (Elevation 194.5 m).

The till was penetrated about 3.0, 3.8, 4.9 and 5.0 m in boreholes 10-44, 10-45, 10-43, and 10-42, respectively, and all boreholes were terminated within this deposit. In boreholes 06-1 and 06-2, the silty sand was penetrated about 2.4 m and 0.9 m (in boreholes 06-1 and 06-2, respectively) and both boreholes were terminated within this deposit, below which depth Dynamic Cone Penetration Tests (DCPT) were advanced in both boreholes. The DCPTs were terminated upon effective refusal at depths of about 12.2 m (Elevation 186.4 m) and 9.5 m (Elevation 190.7 m) for boreholes 06-1 and 06-2, respectively.

Standard Penetration Testing (SPT) 'N' values recorded within the till deposit ranged from 4 blows to 52 blows per 0.3 m of penetration, indicating the till is loose to very dense.

The natural water content measured on select samples of the till (silty sand) deposit ranged between about 7 per cent and 12 per cent. Grain size distribution curves on select samples of the till (silty sand) from the current investigation are shown on Figures A5, A6, A9, A11, A13, A14 in Appendix A. A grain size distribution curve on a selected sample of the lower silty sand from the previous 2007 investigation is shown on Figure B5 in Appendix B.

The results of Atterberg Limits testing carried out on the fines portion of three samples of the till deposit are illustrated on the plasticity chart on Figure A2 in Appendix A. The test results are summarized below and indicate the silt and clay size particles within the till is of low plasticity.

Borehole	Sample	Elevation (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
10-42	8	193.5 – 194.1	12	8	4
10-42	12	190.4 – 191.1	12	9	3
10-45	6	194.9 – 195.5	15	10	5



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4.2.7 Groundwater Conditions

The water levels were noted within the open boreholes at the time of the drilling operations. Piezometers were installed in boreholes 06-1 and 10-43. The piezometers were sealed into the till deposit, below the clayey silt to silty clay layer. Details of the piezometer installations are shown on the Record of Borehole Sheet following the text of this report. The water levels measured in the piezometers and open boreholes upon completion of drilling are summarized below.

Borehole	Installation	Ground Surface Elevation (m)	Depth to Water Level (m)	Water Level Elevation (m)	Date
06-1	Piezometer	198.6	4.5	194.1	July 7, 2006
			- 0.1*	198.7	July 10, 2006
			- 0.9*	199.5	July 31, 2006
			- 0.9*	199.5	Aug. 18, 2006
06-2	Open Borehole	200.2	1.6	198.6	July 5, 2006
10-42	Open Borehole	200.2	3.7	196.5	September 30, 2010
10-43	Piezometer	200.1	2.4	197.7	October 6, 2010
			0.2	199.9	October 16, 2010
			0.2	199.9	December 13, 2010
10-44	Open Borehole	200.1	3.0	197.1	September 30, 2010
10-45	Open Borehole	200.1	3.7	196.4	September 30, 2010

Note : * Artesian Conditions

It should be noted that the piezometer readings indicate artesian conditions from within the till up to about 0.9 m above the ground surface (Elevation 199.5 m) at borehole 06-1. The piezometer readings taken in borehole 10-43 indicate the groundwater level at about 0.2 m depth (Elevation 199.9 m). Groundwater levels at the site of the culvert will depend on rainfall and snowmelt conditions and are expected to fluctuate seasonally.



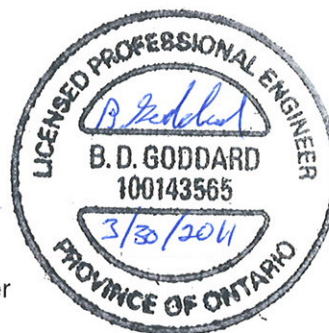
5.0 CLOSURE

The field technician supervising the drilling program was Mr. Harold Cameron. This report was prepared by Mr. Nicolas R. LeBlanc, EIT and reviewed by Mr. Bruce D. Goddard, P.Eng., a senior geotechnical engineer. Mr. Fintan J. Heffernan, Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

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NRL/BDG/FJH/ca/tm

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FOUNDATION INVESTIGATION

PART B

FOUNDATION DESIGN REPORT
DETAILED DESIGN
CULVERT REPLACEMENT AT STATION 16+065.6
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 28
PETERBOROUGH, ONTARIO
G.W.P. 4053-06-00, SITE NO. 26-215



6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the detailed design of the proposed culvert extension / replacement as part of the proposed highway operational improvement plan. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the detailed as well as the preliminary subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out design of the proposed culvert foundations. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project.

6.1 General

As mentioned previously, the existing Highway 7 road surface at the site is at about Elevation 200.2 m and the existing open footing culvert obvert is at about Elevation 199.5 m. The existing structure consists of an open footing concrete culvert that measures about 3.7 m wide x 18.4 m long. The elevation of the creek bed at the inlet and outlet of the culvert is about Elevation 198.3 m, resulting in a height of about 1.2 m from the creek bed to culvert obvert. The existing open footing foundation width and depth was not known at the time of this report and should be confirmed prior to construction. Based on the results of a visual inspection of the existing culvert structure (performed by Harmer Podolak Engineering Consultants during preliminary design and by D.M. Wills under the current design phase), we understand that the culvert is approaching the end of its service life and will need to be replaced as part of the operational improvements. The culvert will also need to be lengthened in order to permit widening the highway embankment to accommodate five-laning and intersection improvements (see Drawing 1).

According to the pavement report prepared by MTO, there will be a 45 mm grade raise resulting from the proposed pavement overlay of the existing pavement. Based on the existing topography, the new embankment widening is expected to be up to about 2 m high (i.e. above the existing ground surface). The culvert should be designed to withstand the maximum anticipated overburden, lateral pressures and live loads. The effects of frost should also be considered in the structural design if frost susceptible soils are located within the depth of frost penetration.

Three culvert type and configuration alternatives for the culvert replacement have been considered based on the information presented in the D.M. Wills Draft Structural Design Report, dated November 2010. The culverts that were discussed were the following:

- 6.0 metre span Precast Concrete Open Footing Culvert;
- 3.6 metre span Precast Concrete Open Footing Culvert; and,
- 3.6 metre span Precast Concrete Closed Bottom Box Culvert.

The preferred replacement structure considering the groundwater conditions is a closed bottom box precast concrete culvert measuring about 3.6 m wide x 1.8 m high x 25.7 m in length.



6.2 Culvert Foundation Options

Apart from the existing topsoil and fill placed on the west side of the highway (i.e. previous grade raise) and existing road embankment fill, the subsurface conditions generally consist of a surficial layer of peat, underlain by a layer of clayey silt to silty clay. The clayey silt to silty clay was underlain by till. At boreholes which were drilled on top of the embankment and roadway area, a surficial layer of topsoil (on the grass shoulder) or asphaltic pavement (on the roadway surfaces) underlain by sand and gravel fill was generally encountered above the clayey silt to silty clay deposit.

The depth of roadway fill generally extends to elevation 199.0 to 199.5 metres approaching the culvert location and extends to elevation 198.1 metres near the culvert location. The depth of organic soils generally extends to elevations ranging from elevation 197.8 to 198.8 metres. The underlying layer of clayey silt to silty clay extends to elevations ranging from elevation 194.5 to 195.7 metres. Below the layer of clayey silt to silty clay, the glacial till extends beyond the termination elevations of the boreholes, ranging from elevation 191.9 to 186.4 metres.

As reported in the preliminary investigation an up gradient (artesian) groundwater condition exists in the underlying glacial till. This condition was confirmed in the standpipe piezometer that was installed during the current investigation on the west side of the existing culvert. Groundwater levels noted within the underlying glacial till ranged from Elevation 199.5 metres (noted in the piezometer installed within borehole 06-1 on the east side in August 2006) to 199.9 metres (noted in the piezometer installed within borehole 10-43 on the west side in October 2010).

Based on the subsurface information obtained, the upper portion of the native firm to very stiff clayey silt to silty clay soils located below the surficial topsoil, peat, fill and very loose silty sand deposits are considered suitable for the support of the proposed culvert foundation. Table 1 provides a comparison of culvert foundation options based on advantages, disadvantages, relative costs and risks/consequences. Both open footing and closed box culverts have been included as options. The options also assume that open footings will be cast-in-place and the culvert structure placed or constructed on top of the footing as either a precast or cast-in-place unit. Closed box culverts can be either precast or cast in place as a single unit. From a foundations perspective, the preferred culvert replacement structure type at this location is closed bottom box culvert with precast culvert units. Given that the culvert replacement is to be located at the same location as the existing culvert, the preferred alternative will require a temporary diversion of the stream during construction. Some localized stripping of the topsoil, peat and any very loose or highly organic soils that are present at this site will also be required as part of the embankment construction.

6.2.1 Geotechnical Resistance

6.2.1.1 Preferred Alternative – Closed Bottom Box Culvert

Based on Table 1 and the subsurface conditions, a box culvert is the preferred culvert type. The approximate invert elevation, recommended level of subexcavation, and the founding soil type for a box culvert is presented below.



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Approximate Culvert Station	Relevant Boreholes	Approximate Creek Bed Elevation (m)	Recommended Subexcavation Level for Proposed Culvert (m)	Founding Soil Type
16+065.6	06-1, 06-2, 10-42, 10-43	197.8	197.2	Stiff to Very Stiff Clayey Silt to Silty Clay

The above recommended subexcavation level indicates the estimated target elevation required to reach the appropriate founding subgrade soil. The actual founding level will depend on frost protection requirements and the depth of the granular bedding and/or engineered fill required under the culvert (see Section 6.7.3).

Assuming the founding levels noted above, the factored geotechnical resistance at Ultimate Limit States (ULS) and the unfactored geotechnical resistance at Serviceability Limit States (SLS) to be used for detailed design of the box culvert is given below.

Approximate Culvert Station	Culvert	Total Proposed Culvert Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa) for 25 mm settlement
16+065.6	Closed Bottom Box Culvert	4.1	300	200

The geotechnical resistance values assume the culvert is founded on granular bedding of 200 mm of Granular "A" supported on undisturbed native clayey silt to silty clay soils which for design may be assumed to be at the founding elevations levels indicated above. Lower strength soils were encountered below about Elevation 196 m within the cohesive deposit. In addition, artesian groundwater conditions were encountered within the glacial till (located within about 2.5 m of the proposed founding elevation for the box culvert). As a result, it is recommended that subexcavation / founding depths be as high as possible to reduce the risk of uplift of the clayey silt to silty clay layer and additional dewatering efforts.

If wing walls are employed, a construction joint should be provided between the walls and the box culvert to account for possible differential settlement.

It should be noted that the geotechnical resistance values provided above for the culvert replacement assume the existing footings are located at or above the design founding level. Disturbance to the founding subgrade soils should be anticipated during the removal of the existing culvert footings. As a result, any new footing areas founded above the existing footing level will require sub-excavation of any loosened or disturbed soil caused by removal of the existing culvert footings and replacement with approved engineered fill as described in Section 6.7.3.

6.2.1.2 Open Footing Culvert

As an alternative to a closed box culvert, an open footing culvert could be used. The approximate invert (creek bed) elevation, recommended footing founding level (i.e. depth of subexcavation), and the founding soil type at the footing level for the proposed open footing culvert replacement / extension are presented below.



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Approximate Culvert Station	Relevant Boreholes	Approximate Creek Bed Elevation (m)	*Recommended Founding Levels for Proposed Culvert Footing (m)	Founding Soil Type
16+065.6	06-1, 06-2, 10-42, 10-43	197.8	196.2	Firm to Stiff Clayey Silt to Silty Clay

*Assumed to be at or below the existing culvert footing level

The above recommended founding levels indicate the estimated target elevation required to reach the appropriate founding subgrade soil(s). The actual founding level will depend on the final location of the culvert, the design invert level and frost protection requirements and the elevation of the existing footings.

Assuming the founding levels noted above, the factored geotechnical resistance at Ultimate Limit States (ULS) and the unfactored geotechnical resistance at Serviceability Limit States (SLS) to be used for detailed design of the open footing culvert is given below.

Approximate Culvert Station	Culvert	Assumed Open Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa) for 25 mm settlement
16+065.6	Open Footing Culvert	1.2	250	150

The geotechnical resistance values assume the culvert is founded on the undisturbed native silty clay to clayey silt soils. It should be noted that lower strength clayey soils were encountered below about Elevation 196 m, (i.e. below the apparent crust) within the cohesive deposit. In addition, artesian groundwater conditions were encountered within the glacial till layer (located within about 1.5 m of the proposed founding elevation). As a result, it is recommended that subexcavation / founding depths be as high as possible to reduce the risk of unstable founding soils and to reduce dewatering efforts. It should be noted that, depending on design founding elevation and the water levels at the time of construction, dewatering may be required to limit the potential for boiling, loosening, and/or disturbance of the founding clayey soils at the founding level. It is recommended that the water level in the till be checked before excavation, i.e., in the Piezometer 1H Boreholes 10-43. As a result, excavation for the construction of the open footings should be in accordance with OPSS 902.

6.2.1.3 Wing Wall Foundations

Based on conversations with the designers the extension of the culvert to eliminate the need for wing walls is not feasible at the west end of the culvert due to its location. We understand that an armour stone wall is being considered to treat the west end of the culvert. The east end of the culvert will be over extended and will not have an end/wing wall section. An armour stone wall system typically consists of dry stacking of large stone blocks, typically from quarried bedrock. This wall system is typically considered a gravity wall, which gathers its stability from the weight of the retaining wall and from the enlarged base. This wall system does not have a separate foundation, but does require a suitable bearing surface. Typically the first course of stone needs to be embedded into the foundation soils. The depth of this course needs to be below the level of anticipated scour.



For this culvert, scour is anticipated to be minimal, but this should be confirmed. Therefore, the first course of stone should be placed no higher than Elevation 197.7 m. At this foundation level the existing native soils could contain organic materials and are considered unsuitable for foundation soils of this wall system, therefore these native organic soils will need to be subexcavated and replaced with the same bedding materials used for the culvert, which are described in section 6.7.3 of this report.

For design of this armour stone wall, the factored geotechnical resistance at ULS may be taken as 250 kPa, assuming that the armour stone wall has a minimum base width of 1.7 m. The geotechnical resistance at SLS, for 25 mm of settlement, may be taken as 200 kPa.

The design and construction of an armour stone wall adjacent to the culvert extension areas should incorporate placement of a suitable non-woven geotextile fabric immediately below and behind the armour stone, to minimize the potential for migration of fine soil particles into the voids between the stones and will result in a loss of ground between the wall as well as maintain a free draining wall system. The backfill on the armour stone wall should consist of a Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II, but with less than 5 percent passing the No. 200 sieve, should be used as backfill behind the walls. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.

The global stability of the armour stone wall up to 2.1 m in height has been assessed using the commercially available program SLOPE/W (Version 7.17), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. The ranges of parameters used for the foundation soils in the stability analyses are summarized below:

Soil Type	Bulk Density, γ (kN/m ³)	Apparent Cohesion, kPa	Internal Friction Angle, ϕ'
New Roadway Fill	22	0	32°
New Retaining Wall Backfill	21	0	32°
Native Clayey Silt to Silty Clay	19	100	0°
Native Till	21	0	34°

The results of the global stability analyses indicate that a factor of safety of 1.5 or greater is achieved for an armour stone wall up to 2.3 m in height with a base of 1.8 m wide at this culvert location. The design of the armour stone wall should be designed by the supplier and checked for internal stability.

6.2.2 Resistance to Lateral Loads

Resistance to lateral forces (i.e. sliding resistance) between the base of the concrete culvert foundation and the undisturbed native materials should be calculated in accordance with Section 6.7.5 of the CHBDC. For the concrete box option, assuming the culvert is precast concrete and is placed on compacted granular bedding, a coefficient of friction value ($\tan \delta$) of 0.5 can be used for design. The coefficient of friction value for this option can be increased to 0.58 if cast-in-place concrete is placed on compacted granular bedding.

For the open footing culvert option, assuming the footings are cast-in-place and founded on undisturbed clayey silt to silty clay, a coefficient of friction value ($\tan \delta$) of 0.43 can be used for design. In accordance with the



CHBDC, a factor of 0.8 is to be applied to the coefficient of friction value when calculating the horizontal resistance.

6.2.3 Frost Protection

The design frost penetration depth in the area of the proposed culvert is 1.6 m. All shallow foundations should be provided with a minimum of 1.6 m of soil cover or equivalent thermal insulation for frost protection.

6.3 Lateral Earth Pressures

The lateral earth pressures acting on the new structure and any associated foundation walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II, but with less than 5 percent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the culvert / wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.6 m behind the back of the wall stem (Case I in Figure C6.9.1(I)(i) of the Commentary to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(I)(ii) of the Commentary to the CHBDC).
- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade Material (SSM) for the new portions of the approach embankments.

SSM

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

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



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

- If the culvert structure allows lateral yielding of the culvert walls, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (such as typically the case for a rigid concrete box culvert), at-rest earth pressures should be assumed for geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the culvert wall and any retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.1 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio (A) for Peterborough is 0.05. Based on experience, for the overburden soils at the site and embankment heights of up to about 2.5 m, a 10 to 20 percent amplification of the ground motion may occur, resulting in an increase in the ground surface acceleration from 0.05g to between 0.055g and 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.06$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.09$). The seismic active earth pressure coefficients are also dependent on the vertical component of the earthquake acceleration, k_v . Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2.3k_h$, $k_v = 0$, and $k_v = -2/3$.
- The following seismic active earth pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

Seismic Active Pressure Coefficients, K_{AE}			
	Case I (SSM)	Case II	
		Granular A	Granular B Type II
Yielding wall	0.32	0.26	0.26
Non-yielding wall	0.37	0.30	0.30

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



- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to $250A$ (mm), where A is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to 15 mm at this site.
- The earthquake-induced dynamic active lateral pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

Where: p is the total (static plus seismic) pressure distribution (kPa)
 K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
 K_{AE} is the seismic active earth pressure coefficient;
 γ' is the effective unit weight of the soil (kN/m^3)
(given above for fill materials)
 d is the depth below the top of the wall (m); and
 H is the height of the wall above the toe (m).

6.4 Settlement

Referring to Drawing 1, we understand that the existing embankment will be widened by up to about 4 metres on each side of the roadway. Based on conversations with the designer, it is understood that the existing roadway and new embankment will be raised about 45 mm (about Elevation 200.4 m). From the boreholes and surrounding topography, the existing ground surface is typically less than 0.2 metres below the proposed road surface on the west side of the culvert replacement (outside of the tributary creek valley) and generally about 1.9 metres below the road surface on the east side of the culvert at the borehole locations.

Provided that the culvert is founded on undisturbed native clayey silt to silty clay, the total settlement of the foundation soils at the culvert replacement / extension along the central portion of the highway is expected to be less than 25 millimetres.

For the proposed culvert replacement and embankment widening on both sides of the existing highway, settlements are anticipated to be greater due to the increased loading imposed on the clayey subsoils from the new embankment loading. Assuming the new embankment footprint will be stripped of topsoil and peat and replaced with engineered fill, the total thickness of the new embankment fill on the east side is anticipated to be up to about 2.0 metres. The total net loading on the foundation soils (after stripping, backfilling and embankment fill construction) from the embankment widening on the east side was estimated to be about 44 kilopascals.

A settlement analysis was performed using the commercially available program Settle3D (Version 2.01) produced by Rocscience Inc. The soil parameters used for the analysis were based on the laboratory and in situ test data collected during the preliminary and current investigations. The apparent preconsolidation pressure and over-consolidation ratio (OCR) of the fine grained soils required in the settlement analysis were estimated using the results of the borehole and consolidation test data.



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The immediate compression of the till below the clayey silt to silty clay layer was modelled using elastic moduli. The time dependant, consolidation settlement of the clayey silt to silty clay deposit was modelled using the following parameters.

Soil	Initial Void Ratio e_o	Recompression Index C_r	Compression Index C_c	Apparent Preconsolidation Pressure σ'_p (kPa)
Clayey Silt to Silty Clay	0.74	0.02	0.19	340

The following summarizes the simplified stratigraphy, unit weights and deformation parameters (see Chapter 6, “*Commentary to the CHBDC, 2001*”) employed in the settlement analysis:

Soil	Thickness (m)	Bulk Unit Weight (kN/m ³)	Deformation Properties
Clayey Silt to Silty Clay	3.15	19	See table above
Till	10	21	E' = 15 MPa

Although the net loading due to the placement of the new culvert itself on the foundation soils is expected to be relatively small, the structural design of the culvert sections should consider resistance to the bending moment anticipated to occur along the centreline of the culvert due to the non-uniform embankment geometry and loading conditions (i.e. vertical settlements and horizontal strains).

In the approach areas of the proposed widening, it is predicted the maximum total settlement of the foundation soils is estimated to be about 25 millimetres due to the loading imposed by the new embankment fill. This total is estimated to be comprised of about 10 millimetres of immediate settlement due to compression of the cohesionless soil deposit at depth and about 15 mm of time dependent settlement of the cohesive soil layer.

Settlement of the new granular embankment fill itself is expected to occur rapidly (i.e. during or shortly after construction) and be less than 25 mm if placed and compacted properly.

Considering the central portion of the proposed culvert replacement has been preloaded by the existing roadway embankment, it is estimated that there will be approximately 25 millimetres of differential settlement between the central portions of the new culvert and the outer portions where the embankment has been added. This amount of settlement is considered to be minimal; therefore no cambering of the culvert is needed.

6.5 Stability

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the roadway widening with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) or shallower will have a factor of safety of greater than 1.3 against deep-seated slope instability under static loading conditions.

The slope stability analysis for this embankment configuration were carried out using the following parameters, derived from field and laboratory testing and accepted correlations, using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis.



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Material	Bulk Unit Weight, kN/m ³	Apparent Cohesion, kPa	Effective Angle of Friction, degrees
New Roadway Fill	22	0	32
Existing Roadway Embankment Fill	21	0	32
Clayey Silt to Silty Clay	19	100	0
Till	21	0	34

Pseudo-static seismic slope stability analysis for the above configurations also indicate that the roadway widening side slopes will have a factor of safety greater than 1.1 against deep-seated slope instability based on an acceleration of 0.1g.

6.6 Base Heave

As previously mentioned, artesian groundwater conditions were encountered within the till at about 4 to 6 metres depth (about Elevation 195.0 m). Temporary subexcavations during culvert installation should be maintained as shallow as possible to reduce the risk of basal heave or unstable founding soil conditions within the clayey silt to silty clay soils containing seams of silt and sand. Given the artesian water conditions encountered during our investigations within the underlying till, calculations indicate that a soil thickness of at least 2.5 metres must be maintained for a Factor of Safety against basal heave of about 1.0. Based on a total weight calculation, additional resistance to heave will be provided by the shear strength of the cohesive deposit along the sides of the soil that is being subjected to upward water pressure. This will raise the Factor of Safety to above 1.3. As such, subexcavation to about Elevation 197.0 m, the level recommended for the box culvert replacement options (see Section 6.2.1), can be carried out with an adequate factor of safety against base heave. The excavation should be limited to short sections and the culvert placed and partially backfilled before proceeding to the next section. Deeper subexcavations to lower elevations to remove the existing culvert footings and achieve adequate frost protection for wing walls will most likely require some form of dewatering techniques to lower the artesian pressures prior to excavation. The level of artesian pressure should be determined prior to construction. Depending on level of artesian pressure at the time of construction, relief wells or other significant groundwater controls may be required to temporarily lower the artesian pressures and allow for stable working conditions.

6.7 Considerations for Culvert Construction

6.7.1 Subgrade Preparation and Excavation

Prior to the placement of any foundations, engineered fill, bedding, or new embankment construction, all surficial peat, topsoil, organics, and softened or loosened soils should be stripped from below the proposed culvert and embankment widening footprint and wasted/reused for landscaping. All subgrade soils should be inspected or proofrolled prior to placement of foundations or engineered fill and embankment fill should be placed in accordance with SP206S03.

Based on the boreholes and design founding elevations, it is anticipated that excavations up to about 3.0 m below the existing road and ground surface at the west widening area and up to about 2.0 m at the east widening area will be required to remove the topsoil, peat, and existing fill and silty sand soils, and expose the native clayey silt and silty clay soils.



Excavation work should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities, and follow the guidelines outlined in OPSS 902.

It is noted that the soils in which the excavations will be formed are susceptible to disturbance due to groundwater seepage, upwellings from the underlying artesian water conditions, and construction traffic. Groundwater and surface water control will be required.

6.7.2 Groundwater and Surface Water Control

The founding soils for the culverts are susceptible to disturbance due to seepage, artesian water conditions, water ponding, and / or construction traffic. Provided that the existing culvert is to remain in use during construction of the culvert replacement, the majority of the Trout Creek tributary can be diverted around the construction area. If the existing culvert is to be removed prior to completion of the new culvert, a system of sumps and pumps or a temporary CSP will be required to divert the creek from one side of the road to the other. Groundwater seepage into the excavations is expected. The severity of the groundwater conditions is dependent upon many factors including the season during which construction occurs, artesian water pressures, and the flow rate of the Trout Creek tributary. In general, pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works unless dewatering is required to maintain base stability during excavation. Ditches to divert perched water and/or storm water flows around the construction area will also be required to help permit construction and placement of concrete (for open footings) in the dry.

More extensive groundwater control (e.g. relief wells) may be required for excavations that extend to within 1 metre of the till interface or deeper into the underlying till. In such cases, the piezometric water level within the till should be lowered to a minimum of 0.5 m below the base of the proposed excavation level prior to initiation of the excavation in order to limit the potential for disturbance of the native soils and allow placement of concrete in the dry.

6.7.3 Bedding and Backfill

For the concrete culverts being considered, the clayey silt to silty clay soils located within the founding elevations provided in Section 6.2.1 are considered suitable for the support of the bedding for the proposed culvert replacement. Stripping of any existing fills, topsoil, peat, very loose and highly organic soils will be required.

For the box culvert option, the bedding, levelling pad and backfill requirements for the culvert replacement should be in accordance with OPSS 422 for precast concrete rigid frame culverts. The box culvert should be provided with at least 200 millimetres of OPSS Granular 'A', or Granular 'B' Type 2 (OPSS 1010) material if in wet ground conditions, for bedding purposes and partial frost protection. If Granular 'B' Type 2 is used, the placement of a geotextile filter between the bottom of the bedding and native soils may be required depending on the actual in situ ground conditions. The bedding should be placed in lifts not exceeding 150 mm in loose thickness, and compacted to at least 95 per cent of the Standard Proctor maximum dry density. In addition, for closed box culverts, a minimum 75 mm thick uncompacted levelling pad of Granular 'A' or fine aggregate (OPSS 1002) should be provided.

Frost treatment for the culvert structure should follow the guidelines provided in OPSD 803.010 for box and open footing culverts. In order to reduce the potential for frost damage to the culvert (i.e. differential heave and



settlement), the combined thickness of backfill and bedding should be at least 1.6 m (i.e. equal to the depth of frost penetration).

Depending on the culvert base thickness and actual subexcavation depth during construction, the subgrade may require placement of engineered fill to raise grades to the bedding level. This can be achieved by placing additional lifts of the properly placed and compacted bedding material (i.e. Granular 'A' or Granular 'B' Type II).

Compaction equipment should be used in accordance with Special Provision No. 105S10. Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations to ensure that appropriate materials are used, and that adequate levels of compaction have been achieved.

6.7.4 Erosion Protection

Typically, the existing subsoils at the invert level of the culvert site consist of peat or silty sand deposits. It is assumed that all peat and highly organic soils will be removed and replaced with engineered fills within the culvert footprint. At the culvert inlet and outlet locations, if the anticipated water flow velocities are sufficiently high, provision should be made for scour and erosion protection.

In order to prevent water from flowing either beneath the culvert (potentially causing undermining and scouring for box culverts) or around the culvert (creating seepage through the embankment fill and potentially causing erosion and loss of fine particles), a clay seal or cut-off headwall should be provided at both ends of the culvert. If cut-off headwalls are used, the backfill around these headwalls should consist of fine grained cohesive soils with low permeability, such as clay or silty clay.

Erosion protection should be provided upstream and downstream of the culvert as appropriate. Consideration could be given to the use of suitable non-woven geotextiles and rip-rap to provide erosion protection based on hydraulic requirements.

6.7.5 Corrosion Potential

Two samples of soil from boreholes 10-42 and 10-43 were submitted to Exova Accutest Laboratories Ltd. for chemical analysis related to potential corrosion of exposed buried steel and concrete elements (corrosion and sulphate attack). The results of this testing are provided in Appendix A. The results indicate that concrete made with Type GU Portland cement should be acceptable for substructures. The result of the pH, chlorides and resistivity testing indicate a mild potential for corrosion of exposed ferrous metal. These results should be considered when selecting protective coatings for any buried steel objects.

6.7.6 Temporary Roadway Protection

It is understood that the culvert will be installed in two sections with the roadway moved slightly to allow this construction. Boreholes 10-44 and 10-45 were put down along the roadway to enable the design of the temporary roadway production.

The temporary excavation support system should be designed and constructed in accordance with OPSS 539. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539. The contractor is responsible for the complete detailed design of the protection system.



The design of braced soldier pile and lagging walls should be based on rectangular earth pressure distribution using the design parameters given below. Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of any sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter.

The unfactored triangular earth pressure distribution (p in kN/m^2 ; increasing with depth), can be calculated as follows:

$$p = K_a (H + q)$$

Where H = the height of the excavation at any point in metres

$$K_a = \text{active coefficient of earth pressure}$$

$$= \text{soil unit weight}$$

$$q = \text{surcharge for traffic and other loading}$$

For the granular fill, the unfactored rectangular earth pressure distribution (p in kN/m^2 ; constant with depth), can be calculated as follows:

$$p = 0.65 (K_a H + q)$$

Where H = the total height of the excavation

$$K_a = \text{active coefficient of earth pressure}$$

$$= \text{soil unit weight}$$

$$q = \text{surcharge for traffic and other loading}$$

For the cohesive clayey silt to silty clay stratum, the unfactored rectangular earth pressure distribution (p in kN/m^2 ; varying with depth); can be calculated as follows:

$$p = H - m \gamma_u (H - 0.25 H)$$

$$p = \frac{H^2}{2} \gamma_u (H < 0.25 H)$$

Where H = the total height of excavation

$$H = \text{the height of the excavation at any point } < 0.25H$$

$$= \text{soil unit weight}$$

$$q = \text{surcharge for traffic and other loading}$$

$$m = \begin{matrix} 0.4 & \text{if a soft clay layer underlies the excavation} \\ 1.0 & \text{if more resistant layer at excavation base} \end{matrix}$$

$$\gamma_u = \text{undrained shear strength}$$



FOUNDATION INVESTIGATION

The support systems may be designed using the following parameters:

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Unit Weight (kN/m ²)
	Active K _a	At rest, K _o	Passive K _p		
Granular Fill	0.33	0.50	3.0	32	20
Clayey Silt to Silty Clay	0.33	0.55	2.7	27*	19
Glacial Till	0.33	0.50	3.0	34	21

*The support system design should also be checked with an undrained shear strength of 50 kPa.

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly.

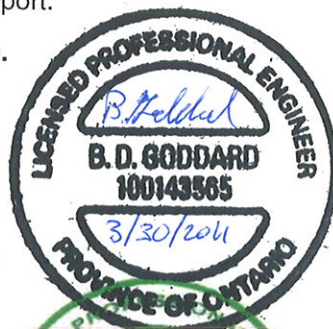


7.0 CLOSURE

This report was prepared by Mr. Bruce Goddard, P.Eng., a senior geotechnical engineer. Mr. Fintan J. Heffernan, P.Eng., and a Designated MTO Contact with Golder, reviewed the technical aspects and conducted a quality control review of the report.

GOLDER ASSOCIATES LTD.

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Senior Geotechnical Engineer



Fintan J. Heffernan, P.Eng.,
Designated MTO Contact



BDG/FJH/tm

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- Ontario Provincial Standards. 2005. "Ontario Provincial Standard Drawing OPSD 3101.150 – Walls Retaining, Backfill Minimum Granular Requirement", November.
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FOUNDATION INVESTIGATION

**Table 1: Evaluation of Culvert Foundation Alternatives
Highway 7 Culvert Replacement (Site No. 26-215)
G.W.P. 4053-06-00**

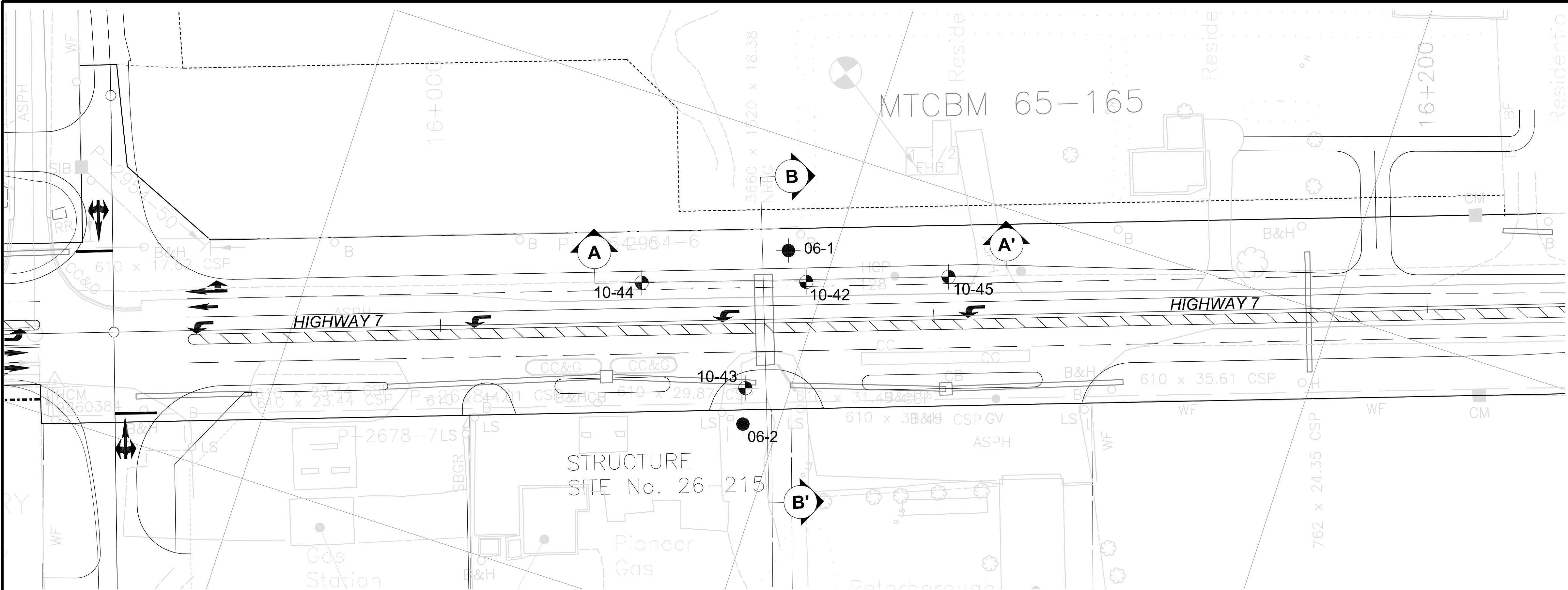
Option	Option No.	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Closed Concrete Box	1	<ul style="list-style-type: none"> Routine excavation and construction procedure; Shallow sub-excavation depth. 	<ul style="list-style-type: none"> Dewatering required to place and compact bedding layer and/or engineered fill; 	<ul style="list-style-type: none"> Lower cost than Option No. 2. 	<ul style="list-style-type: none"> Risk of base heave with deeper excavations; Temporary diversion to natural stream required to place box units.
Open Footing Culvert	2	<ul style="list-style-type: none"> Routine excavation and construction procedure; Proposed founding elevation is likely at or below existing open footing founding level; Reduced construction time for this option if precast culvert units are used as opposed to cast-in-place. 	<ul style="list-style-type: none"> Basal Instability is of concern due to the deeper excavations needed for frost protection; Dewatering required in order to pour concrete footings "in the dry"; Limited soil cover above spread footings for frost protection. Longer construction time for cast-in-place versus precast culvert units. 	<ul style="list-style-type: none"> Higher costs than Option No. 1 due to increased potential for dewatering and requirement to place concrete footings "in the dry", especially if precast culvert unit is used; Higher costs if footing and 3-sided box are cast-in-place. 	<ul style="list-style-type: none"> A higher risk of base heave due to the required deeper excavations; Reduced risk of environmental disturbance to stream if existing culvert can be used during construction of a new, wider culvert open footing.
Deep Foundations (i.e. Piles or Caissons)	NP		<ul style="list-style-type: none"> Pile/caisson tip elevations anticipated to be in excess of 10 m below ground surface. 	<ul style="list-style-type: none"> Much higher costs than Option No. 1 and No. 2. 	

NP = not feasible or not practical

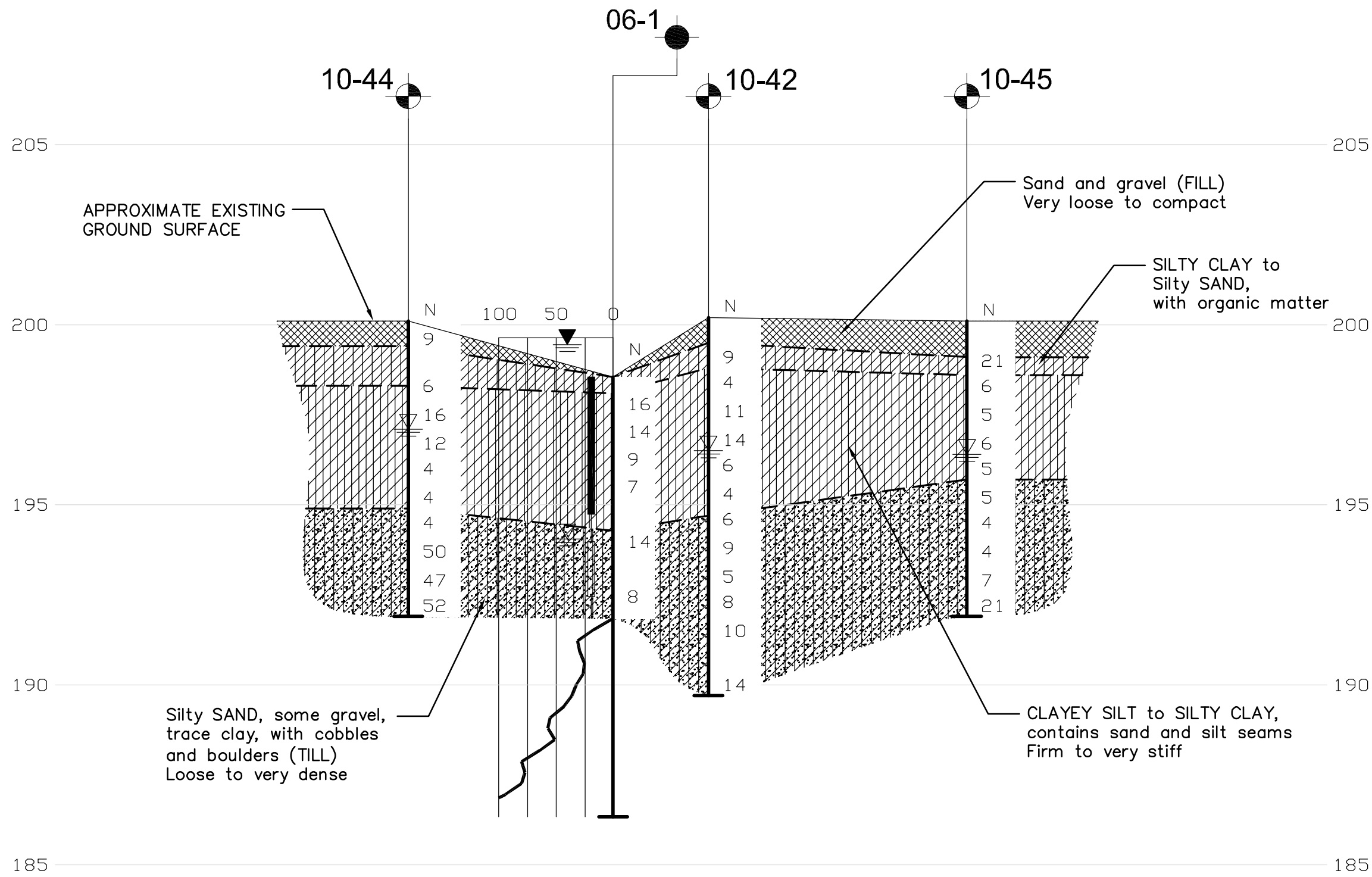
Additional Notes:

- Preferred option as determined from structural, basal instability and dewatering consideration is a Precast Concrete Box culvert.
- If existing culvert footings are founded at depth, there is increased risk for basal heave during removal, sub-excavation, and placement of new foundations.

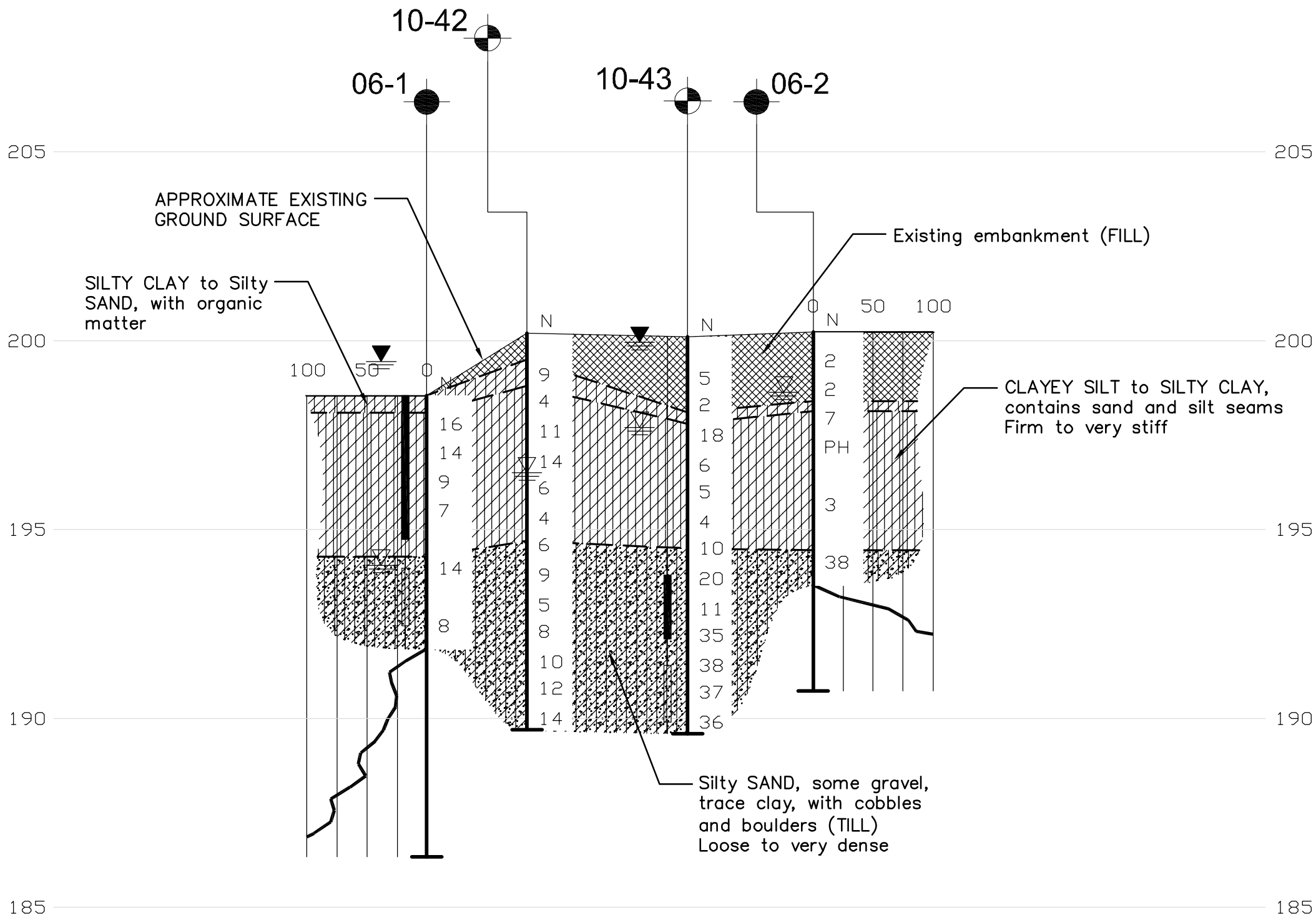
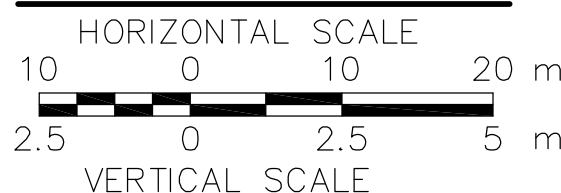
MINISTRY OF TRANSPORTATION, ONTARIO



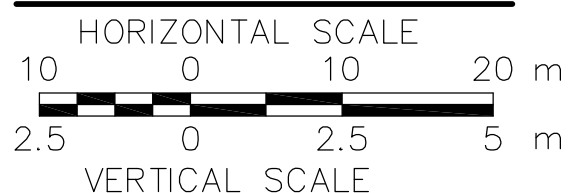
PLAN



PROFILE A-A'

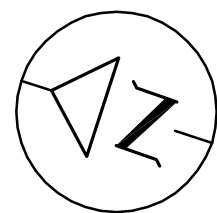


SECTION B-B'



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

CONT No.
WP No. 4053-06-00

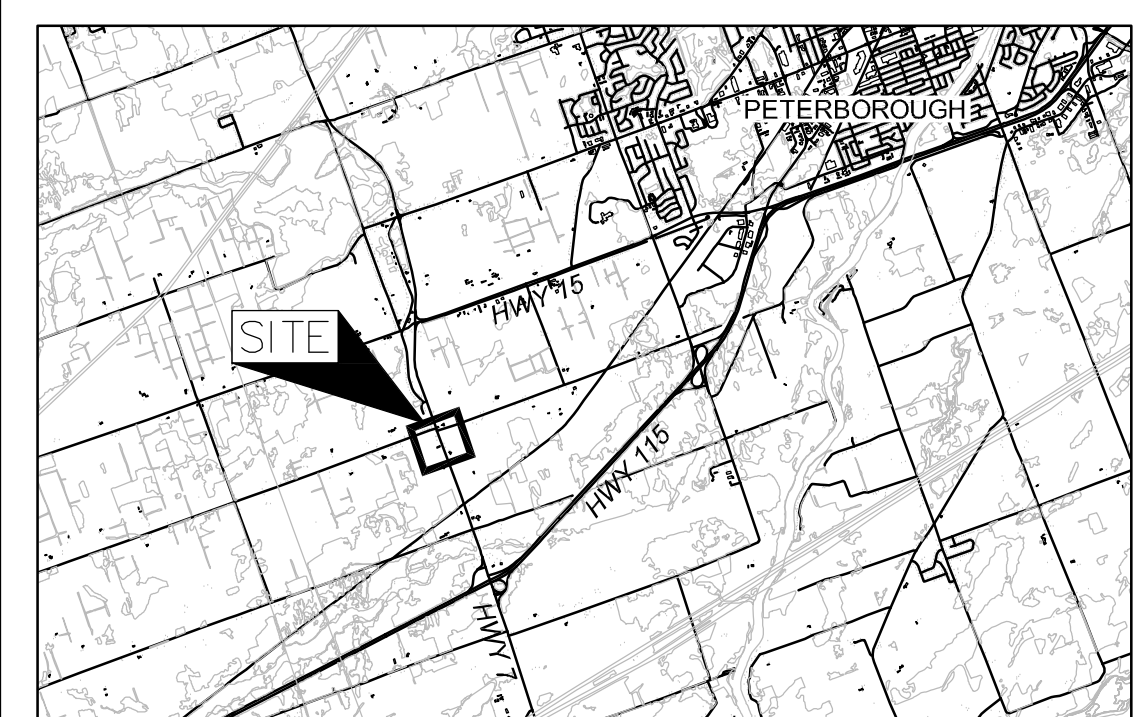


SPRINGVILLE CULVERT
STRUCTURE No. 26-215
HIGHWAY 7
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
1



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN



LEGEND

- Borehole - Current Investigation
- Borehole - Previous MTO Investigation (Geocres No. 31D-428)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer
- WL in piezometer
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
10-42	200.2	4900603.6	392473.3
10-43	200.1	4900608.7	392449.0
10-44	200.1	4900635.3	392462.8
10-45	200.1	4900576.5	392483.1
06-1	198.6	4900609.0	392478.2
06-2	200.2	4900606.9	392441.9

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by MRC (Drawing File No. "73-99-00 PR-2 Property Request Part C.dwg", received July 13, 2010).

NO.	DATE	BY	REVISION
Geocres No. 31D-513			
HWY. 7	PROJECT NO. 10-1121-0007		DIST. 43
SUBM'D. NRL	CHKD. BDG	DATE: DEC. 2010	SITE: 26-215
DRAWN: JM	CHKD. TJN	APPD. FJH	DWG. 1



APPENDIX A

List of Abbreviations and Symbols

Record of Borehole Sheets and Detailed Laboratory Test Results

Present Investigation

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		III. SOIL DESCRIPTION	
AS	Auger sample	(a)	Cohesionless Soils
BS	Block sample		
CS	Chunk sample		
DO	Drive open	Density Index	N
DS	Denison type sample	(Relative Density)	<u>Blows/300 mm</u>
FS	Foil sample		<u>Or Blows/ft.</u>
RC	Rock core	Very loose	0 to 4
SC	Soil core	Loose	4 to 10
ST	Slotted tube	Compact	10 to 30
TO	Thin-walled, open	Dense	30 to 50
TP	Thin-walled, piston	Very dense	over 50
WS	Wash sample	(b)	Cohesive Soils
DT	Dual Tube sample	Consistency	C _u or S _u
II. PENETRATION RESISTANCE			
Standard Penetration Resistance (SPT), N:		<u>Kpa</u>	<u>Psf</u>
The number of blows by a 63.5 kg. (140 lb.)		Very soft	0 to 12
hammer dropped 760 mm (30 in.) required		Soft	12 to 25
to drive a 50 mm (2 in.) drive open		Firm	25 to 50
Sampler for a distance of 300 mm (12 in.)		Stiff	50 to 100
DD- Diamond Drilling		Very stiff	100 to 200
Dynamic Penetration Resistance; N_d:		Hard	Over 200
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	IV. SOIL TESTS	
PM:	Sampler advanced by manual pressure	w	water content
WH:	Sampler advanced by static weight of hammer	w _p	plastic limited
WR:	Sampler advanced by weight of sampler and rod	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 test (LV-laboratory vane test)
		γ	unit weight

Note:

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1 \sigma_2 \sigma_3$	principal stresses (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 ($D_R = p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is p . Unit weight symbol is γ where $\gamma = pg$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

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 (overconsolidated 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	Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

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

PROJECT <u>10-1121-0007</u>		RECORD OF BOREHOLE No 10-42		1 OF 2 METRIC	
G.W.P. <u>4053-06-00</u>		LOCATION <u>N 4900603.6 ; E 392473.3</u>		ORIGINATED BY <u>HEC</u>	
DIST <u>43</u> HWY <u>7</u>		BOREHOLE TYPE <u>Power Auger, 200mm Diam. Hollow Stem</u>		COMPILED BY <u>JM</u>	
DATUM <u>Geodetic</u>		DATE <u>Sept. 30, 2010</u>		CHECKED BY <u>NRL</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED											
200.2	GROUND SURFACE																			
0.0	ASPHALTIC CONCRETE																			
0.1	Sand and gravel (FILL) Brown																			
199.5																				
0.7	SILTY CLAY, with organic matter Stiff to very stiff Dark grey to black Moist		1	SS	9															
198.8																				
1.4	SILTY CLAY, trace sand and gravel, occasional thinly bedded sand seams (Weathered Crust) Stiff to very stiff Grey-brown Moist to wet		2	SS	4															
			3	SS	11											0 3 55 42				
			4	SS	14											0 60 30 10				
196.1			5	SS	6															
4.1	SILTY CLAY to CLAYEY SILT Stiff Grey Wet		6	SS	4															
194.7			7	SS	6															
5.5	Silty SAND, trace gravel and clay (TILL) Loose Grey Wet		8	SS	9											7 55 26 12				
			9	SS	5															
			10	SS	8											13 53 26 8				
192.0																				
8.2	Silty SAND, some gravel, trace clay (TILL) Compact Grey Wet		11	SS	10															
			12	SS	12															

Continued Next Page

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

MIS-MTO 001 10-1121-0007.GPJ GAL-MISS.GDT 3/28/11 JM



PROJECT		RECORD OF BOREHOLE No 10-42				2 OF 2 METRIC										
G.W.P. 4053-06-00		LOCATION N 4900603.6 ; E 392473.3				ORIGINATED BY HEC										
DIST 43 HWY 7		BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem				COMPILED BY JM										
DATUM Geodetic		DATE Sept. 30, 2010				CHECKED BY NRL										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
189.7	End of Borehole		13	SS	14		190									
10.5	Note: Water level in open borehole observed at 3.7 m depth (Elev. 196.5 m) upon completion of drilling.															



MIS-MTO 001 10-1121-0007.GPJ GAL-MISS.GDT 3/28/11 JM

1 OF 2 **METRIC**

CHECKED BY NRL

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

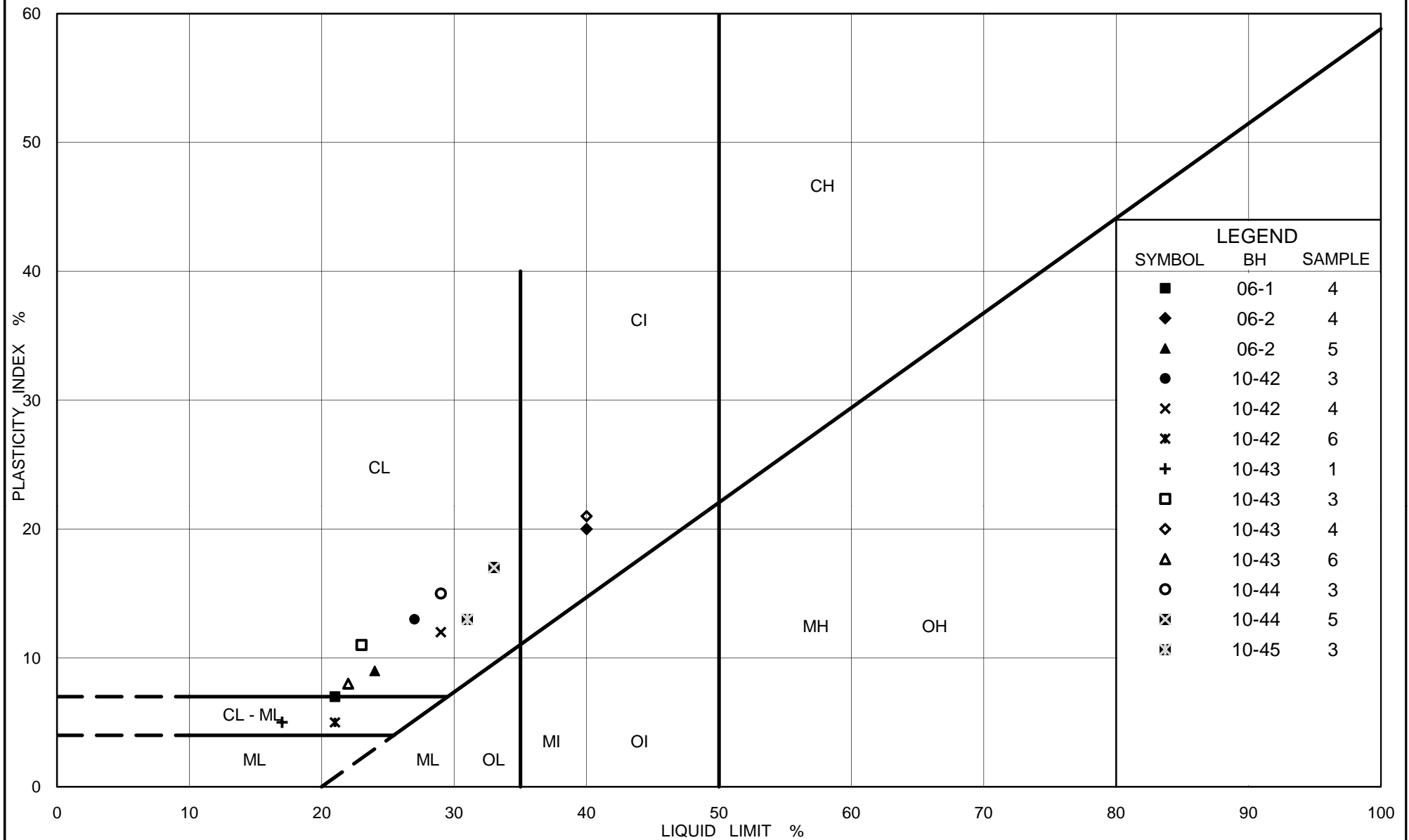
MIS-MTO 001 10-1121-0007.GPJ GAL-MISS.GDT 3/28/11 JM

PROJECT <u>10-1121-0007</u>		RECORD OF BOREHOLE No 10-43				2 OF 2 METRIC										
G.W.P. <u>4053-06-00</u>		LOCATION <u>N 4900608.7 ; E 392449.0</u>				ORIGINATED BY <u>HEC</u>										
DIST <u>43</u> HWY <u>7</u>		BOREHOLE TYPE <u>Power Auger, 200mm Diam. Hollow Stem</u>				COMPILED BY <u>JM</u>										
DATUM <u>Geodetic</u>		DATE <u>Oct. 6, 2010</u>				CHECKED BY <u>NRL</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
189.6 10.5	End of Borehole Note: 1. Water level in open borehole at 2.4 m depth (Elev. 197.7 m) upon completion of drilling. 2. Water level in well screen at 0.2 m depth (Elev. 199.9 m) on Oct. 16, 2010. 3. Water level in well screen at 0.2 m depth (Elev. 199.9 m) on Dec. 13, 2010.		13	SS	36		190									

PROJECT 10-1121-0007			RECORD OF BOREHOLE No 10-44			1 OF 1 METRIC														
G.W.P. 4053-06-00			LOCATION N 4900635.3 ; E 392462.8			ORIGINATED BY HEC														
DIST 43 HWY 7			BOREHOLE TYPE Power Auger, 105mm Diam. Solid Stem			COMPILED BY JM														
DATUM Geodetic			DATE Sept. 30, 2010			CHECKED BY NRL														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			UNIT WEIGHT γ kN/m ³	GR SA SI CL			
							20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100							
200.1	GROUND SURFACE																			
199.9	ASPHALTIC CONCRETE																			
0.2	Sand and gravel (FILL) Brown																			
199.4																				
0.7	CLAYEY SILT, some sand and organic matter Stiff to very stiff Dark brown to black Moist		1	SS	9															
198.3																				
1.8	SILTY CLAY, some sand Stiff to very stiff Grey-brown Moist to wet		2	SS	6															
198.3																				
			3	SS	16															
			4	SS	12															
196.4																				
3.7	SILTY CLAY, trace sand and gravel Stiff Grey Wet		5	SS	4															
196.4																				
			6	SS	4															
194.9																				
5.2	Silty SAND, some gravel, trace clay (TILL) Loose Grey Wet		7	SS	4															
194.9																				
194.0																				
6.1	Silty SAND and GRAVEL, trace clay, occasional cobbles and small boulders (TILL) Dense to very dense Grey Wet		8	SS	50															
194.0																				
			9	SS	47															
			10	SS	52															
191.9																				
8.2	End of Borehole																			
	Note: Water level in open borehole observed at 3.0 m depth (Elev. 197.1 m) upon completion of drilling.																			

PROJECT 10-1121-0007			RECORD OF BOREHOLE No 10-45			1 OF 1 METRIC											
G.W.P. 4053-06-00			LOCATION N 4900576.5 ; E 392483.1			ORIGINATED BY HEC											
DIST 43 HWY 7			BOREHOLE TYPE Power Auger, 105mm Diam. Solid Stem			COMPILED BY JM											
DATUM Geodetic			DATE Sept. 30, 2010			CHECKED BY NRL											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L WATER CONTENT (%)			γ kN/m ³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	25 50 75								
200.1	GROUND SURFACE																
0.0	ASPHALTIC CONCRETE						200										
0.1	Fine sand and gravel, trace cobbles (FILL) Brown Moist																
199.1							199										
1.0	CLAYEY SILT, some sand, trace gravel, with organic matter Stiff to very stiff Dark grey-brown to black Moist		1	SS	21												
198.6																	
1.5	SILTY CLAY, trace sand (Weathered Crust) Stiff Grey-brown Moist to wet		2	SS	6		198										
			3	SS	5												0 34 44 22
			4	SS	6		197										
196.4																	
3.7	SILTY CLAY to CLAYEY SILT, with grey fine sand, trace gravel Stiff Grey Wet		5	SS	5		196										
195.7																	
4.4	Silty SAND, some gravel and clay (TILL) Loose to compact Grey Wet		6	SS	5		195										11 48 28 13
			7	SS	4		194										
			8	SS	4												
			9	SS	7		193										
			10	SS	21		192										25 44 20 11
191.9																	
8.2	End of Borehole																
	Note: Water level in open borehole observed at 3.7 m depth (Elev. 196.4 m) upon completion of drilling.																

MIS-MTO 001 10-1121-0007.GPJ GAL-MISS.GDT 3/28/11 JM



Ontario

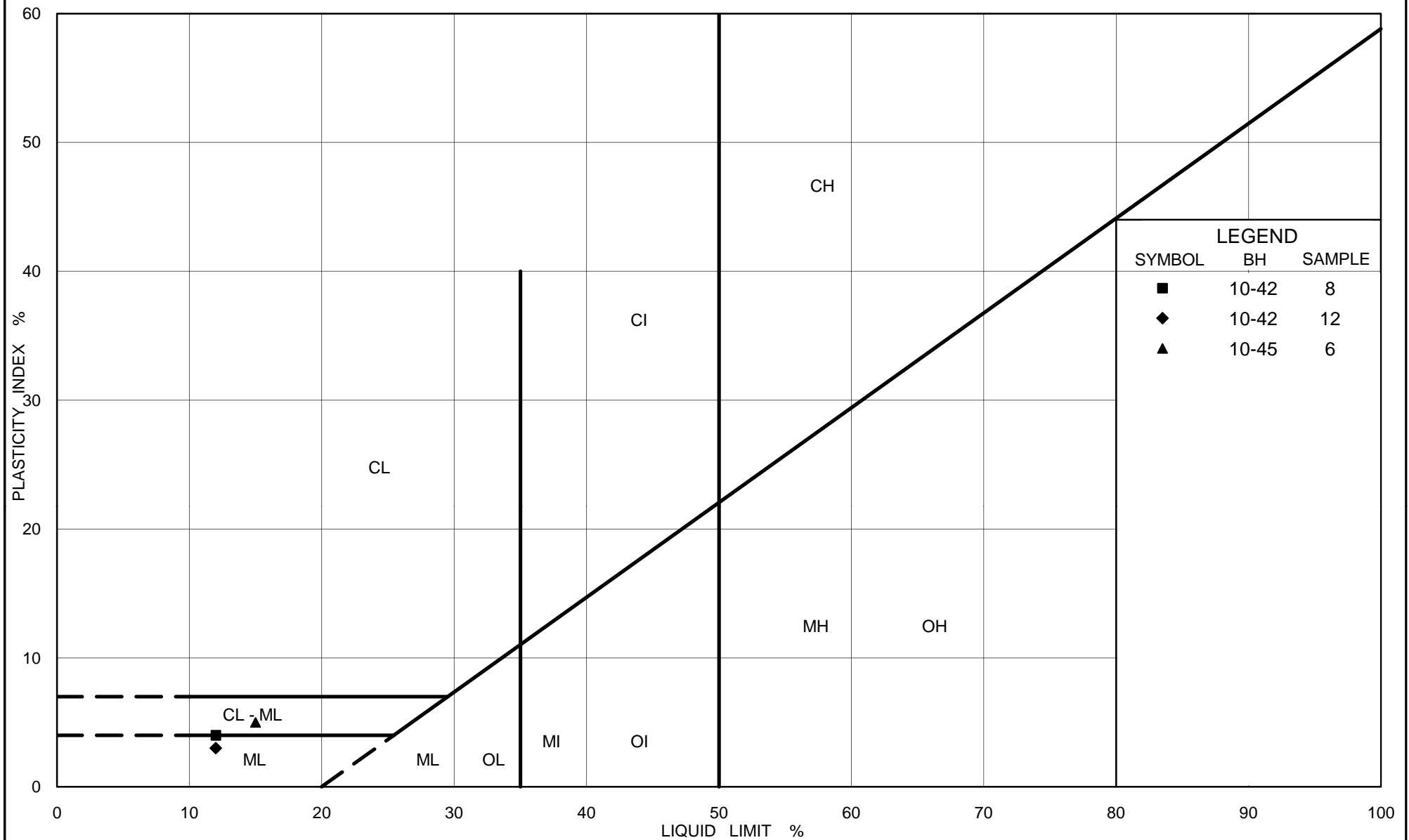
Ministry of Transportation

PLASTICITY CHART

Silty Clay to Clayey Silt

FIG No. A1

Project No. 10-1121-0007



Ontario

Ministry of Transportation

PLASTICITY CHART Till

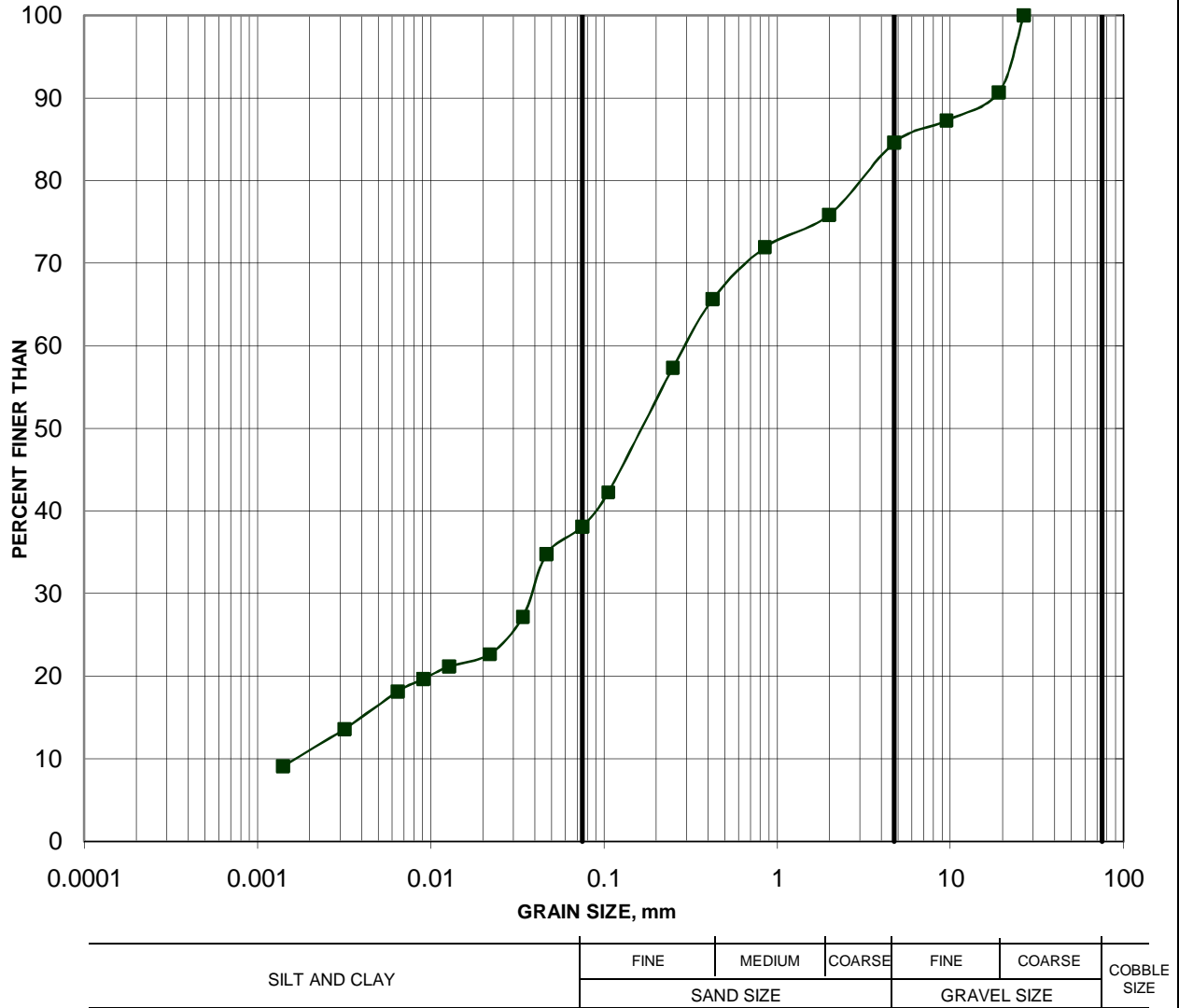
FIG No. A2

Project No. 10-1121-0007

GRAIN SIZE DISTRIBUTION

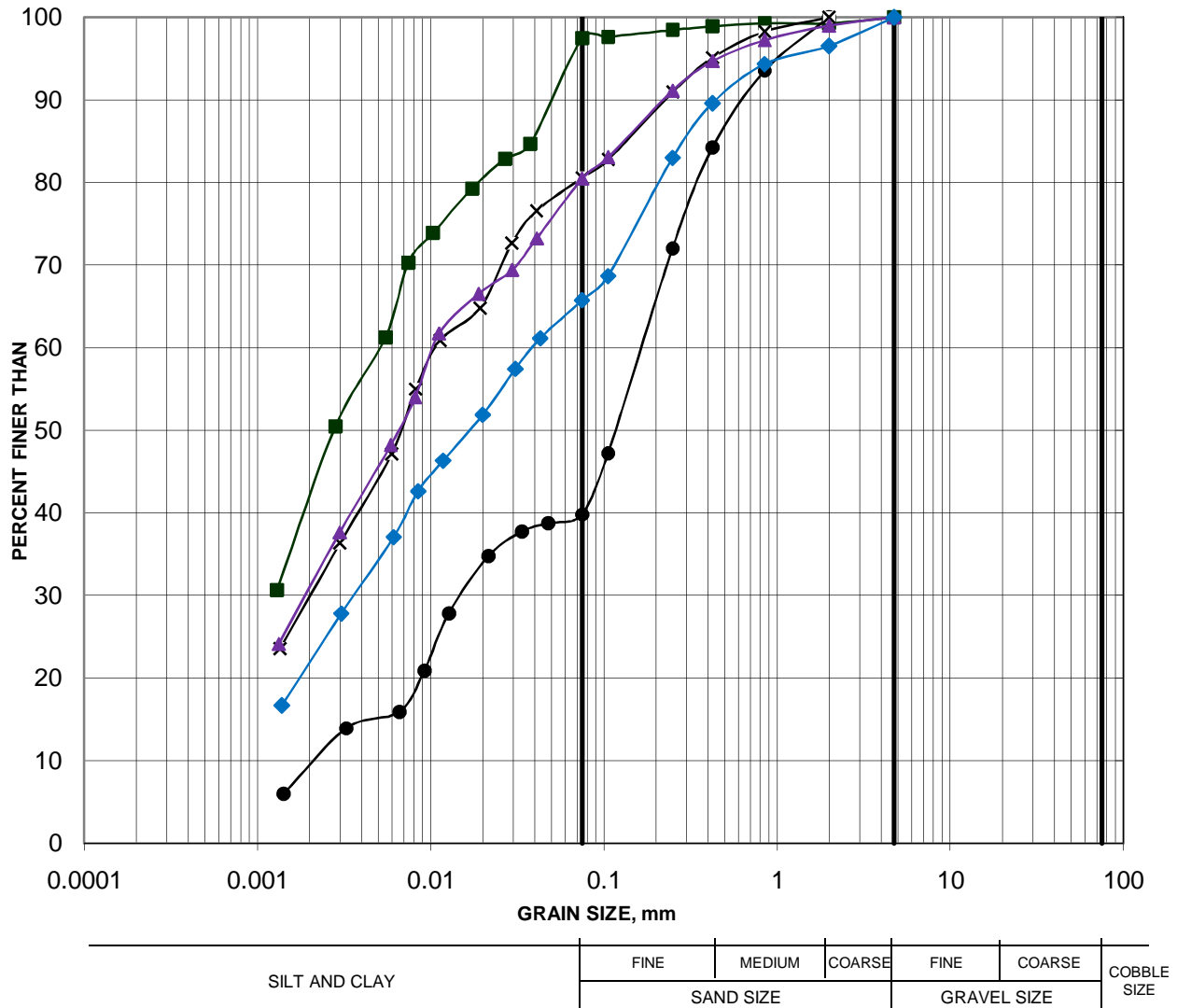
FIGURE A3

SANDY SILT SAND WITH ORGANIC MATTER (FILL)

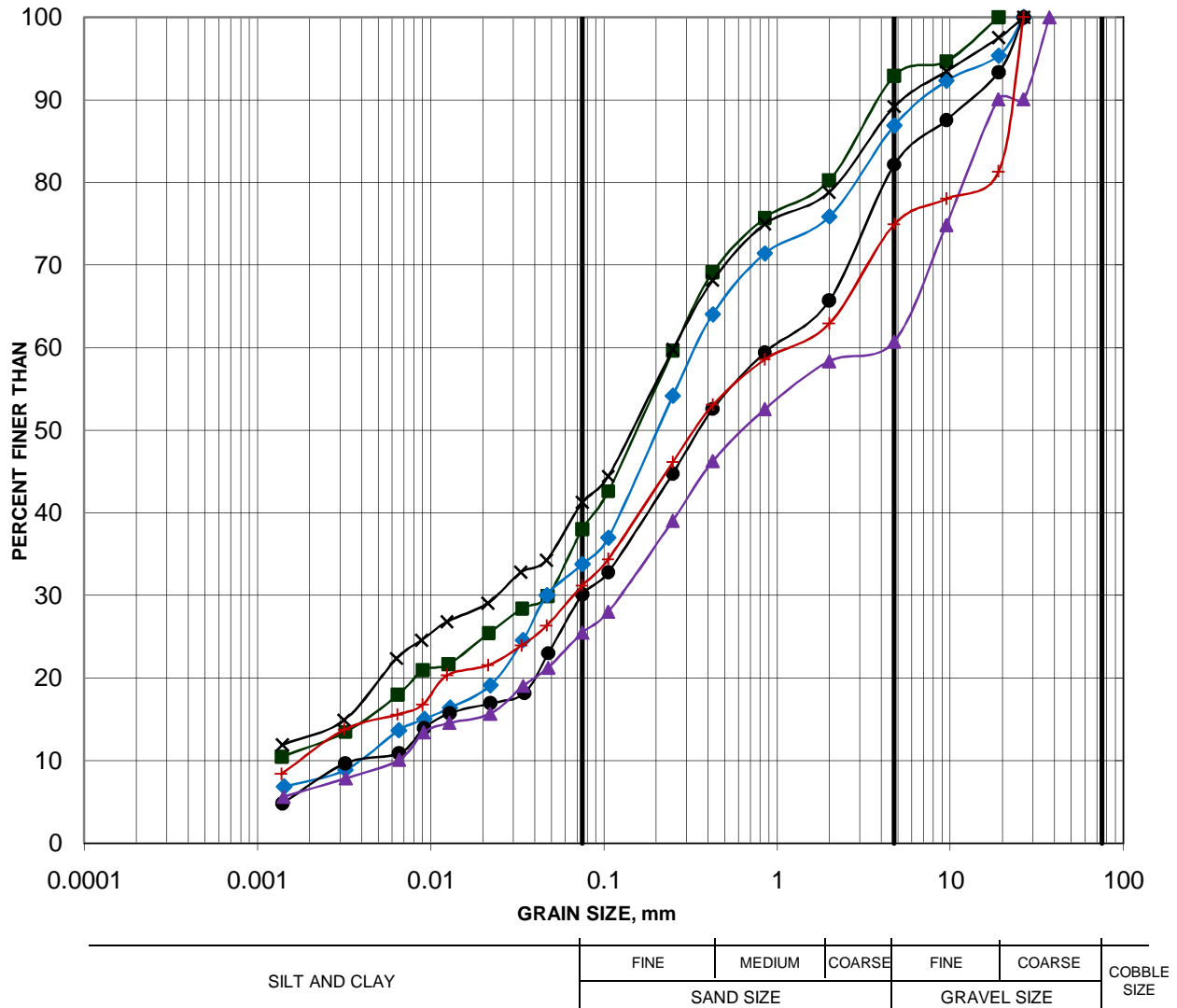


Borehole	Sample	Depth (m)
10-43	1A	1.07-1.37

CLAYEY SILT TO SILTY CLAY, SOME SAND



SILTY SAND, SOME GRAVEL, TRACE CLAY (TILL)



Borehole	Sample	Depth (m)
10-42	8	6.10-6.71
10-42	10	7.62-8.23
10-43	12	9.15-9.76
10-44	9	6.86-7.47
10-45	6	4.57-5.18
10-45	10	7.62-8.23

Client: Golder Associates Ltd. (Ottawa)
 32 Steacie Drive
 Kanata, ON
 K2K 2A9

Attention: Mr. Bruce Goddard

Report Number: 1029635
 Date: 2010-12-07
 Date Submitted: 2010-12-03

Project: 10-114-0007-200

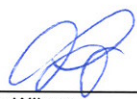
Chain of Custody Number: 127476

P.O. Number:
 Matrix: Soil

			LAB ID:	850241	850242	850243	850244	GUIDELINE		
			Sample Date:	2010-12-02	2010-12-02	2010-12-02	2010-12-02			
			Sample ID:	10-42/S3	10-43/S-5	10-30/S-1	10-30/S-3			
PARAMETER	UNITS	MRL						TYPE	LIMIT	UNITS
Chloride	%	0.002	0.014	0.003	0.005	0.072				
Electrical Conductivity	mS/cm	0.05	0.40	0.39	0.23	1.22				
pH			8.1	7.9	8.7	7.8				
Resistivity	ohm-cm	1	2500	2560	4350	820				
Sulphate	%	0.01	0.05	0.08	0.04	0.02				

MRL = Method Reporting Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration
 Comment:

Methods references and/or additional QA/QC information available on request.

APPROVAL: 
 Lorna Wilson
 Agriculture Lab Supervisor



APPENDIX B

Record of Borehole Sheets and Detailed Laboratory Test Results Previous Investigation

PROJECT 04-1111-024C			RECORD OF BOREHOLE No 06-1			1 OF 2 METRIC		
G.W.P. 73-99-00			LOCATION N 4900609.0 ; E 392478.2			ORIGINATED BY SB		
DIST _____ HWY 7			BOREHOLE TYPE Power Auger, 107 mm O.D. Solid Stem Augers			COMPILED BY DD		
DATUM Geodetic			DATE July 7, 2006			CHECKED BY SLP		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)
198.6 0.0	GROUND SURFACE PEAT							
198.1 0.5	CLAYEY SILT to SILTY CLAY, trace to some sand and gravel, contains sand and silt seams Firm to very stiff Brown to grey Moist		1	SS	16		198	
			2	SS	14		197	
	Grey at 2.1 m depth (Elevation 196.5 m)		3	SS	9		196	
			4	SS	7		195	
194.3 4.3	SILTY SAND, some clay, trace to some gravel Loose to compact Grey Moist to wet		5	SS	14		194	
							193	
191.8 6.7	End of Borehole Start of Dynamic Cone Penetration Test (DCPT)		6	SS	8		192	
							191	
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186.4 12.2							186	
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Continued Next Page

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

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 12/16/10 DD



PROJECT		RECORD OF BOREHOLE No 06-1				2 OF 2		METRIC					
G.W.P.		LOCATION				ORIGINATED BY		SB					
DIST		BOREHOLE TYPE				COMPILED BY		DD					
DATUM		DATE				CHECKED BY		SLP					
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
--- CONTINUED FROM PREVIOUS PAGE ---													
	End of DCPT												
	Notes:												
	1. Water level measured in piezometer at 4.5 m depth (Elevation 194.1 m) upon completion of installation.												
	2. Water level measured in piezometer at 0.1 m above ground surface (Elevation 198.7 m) on July 10, 2006.												
	3. Water level measured in piezometer at 0.9 m above ground surface (Elevation 199.5 m) on July 31, 2006 and August 18, 2006.												

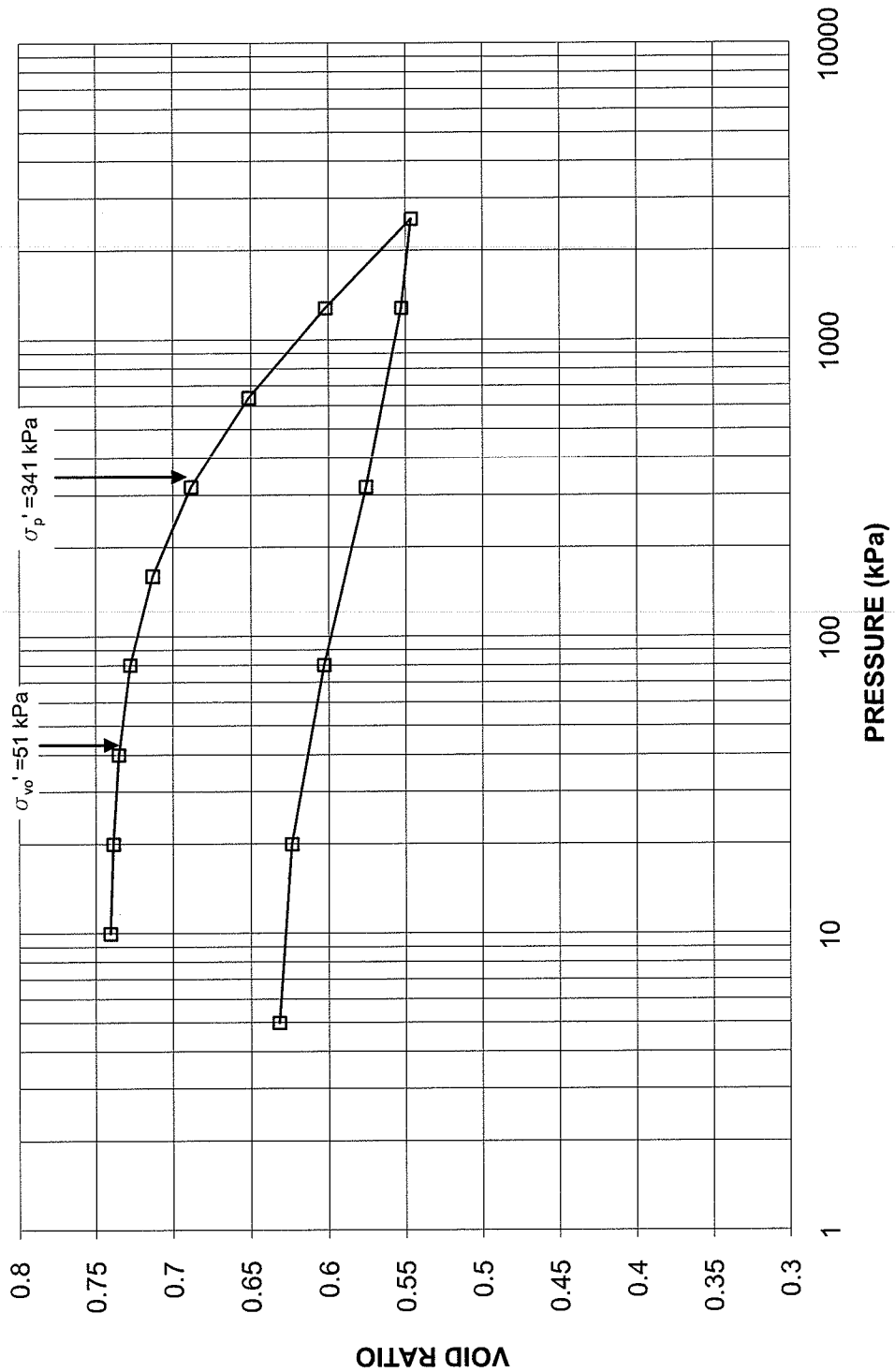
PROJECT 04-1111-024C			RECORD OF BOREHOLE No 06-2			1 OF 1 METRIC		
G.W.P. 73-99-00			LOCATION N 4900606.9 ; E 392441.9			ORIGINATED BY SB		
DIST _____ HWY 7			BOREHOLE TYPE Power Auger, 107 mm O.D. Solid Stem Augers			COMPILED BY DD		
DATUM Geodetic			DATE July 5, 2006			CHECKED BY SLP		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
200.2	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
0.0	TOPSOIL							
0.2	Sand and Gravel, trace clay and silt (FILL) Very loose Brown Moist		1	SS	2		200	
198.4							199	
198.1	SILTY SAND, trace gravel and clay Very loose Grey Moist to wet		2	SS	2		198	
2.1	CLAYEY SILT to SILTY CLAY, trace to some sand and gravel, contains sand and silt seams Soft to firm Brown to grey Moist		3	SS	7		197	
			4	SS	PH		196	
			5	SS	3		195	
194.5							194	
5.8	SILTY SAND, trace gravel and clay Dense Grey Moist		6	SS	38		193	
193.5	End of Borehole Start of Dynamic Cone Penetration Test (DCPT)						192	
6.7							191	
190.7	End of DCPT						Effective Refusal - 214 blows/0.3 m	
9.5	Notes: 1. Water level measured in open hole at 1.6 m depth (Elevation 198.6 m) upon completion of drilling. 2. Laboratory oedometer (consolidation) test performed on Sample No. 4.							

See Note 2.

**CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE**

FIGURE A2

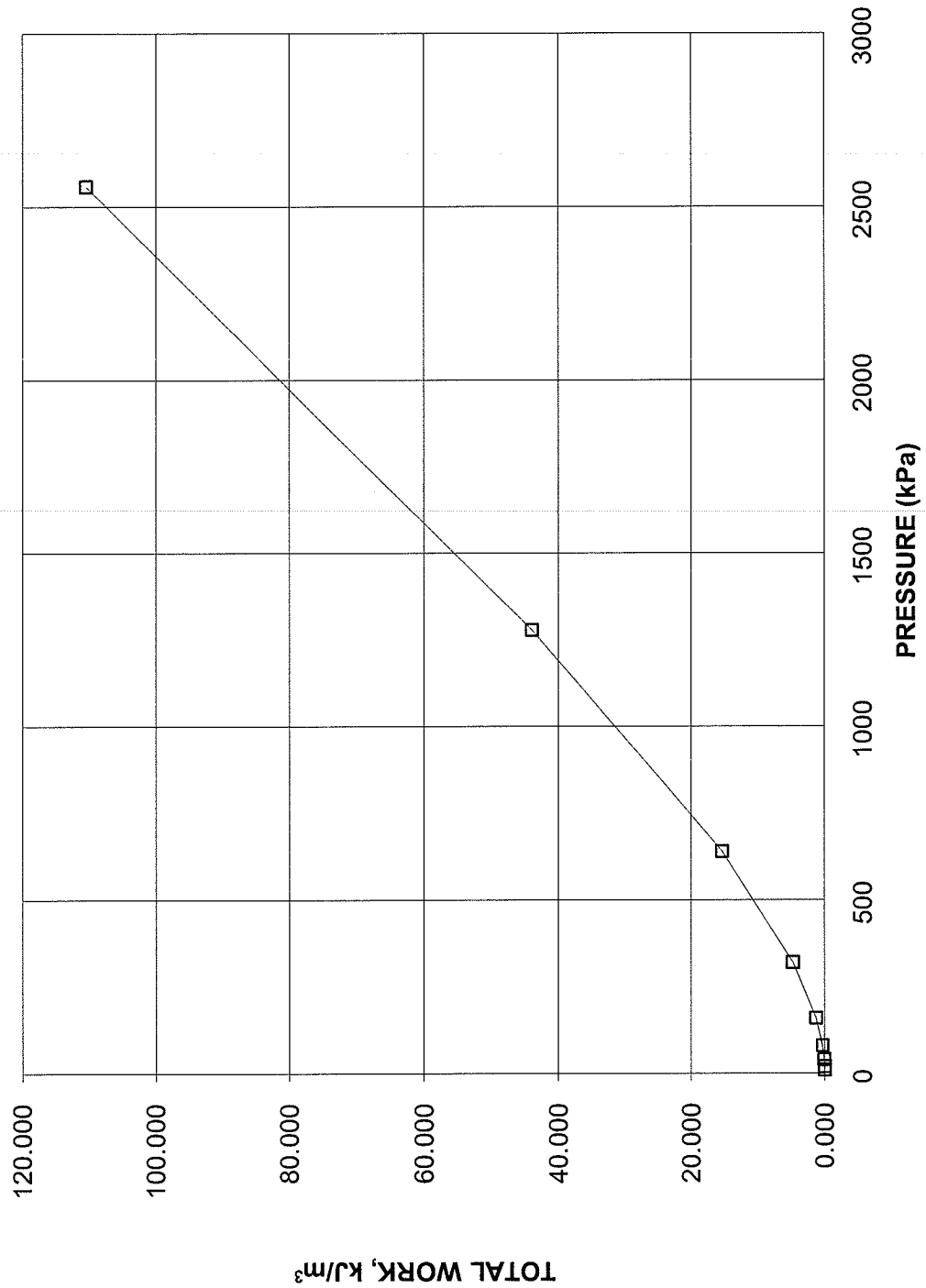
**CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE
BH 06-2, SA 4**



**CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE**

FIGURE A3

**CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 06-2, SA 4**



OEDOMETER CONSOLIDATION SUMMARY

FIGURE A4

SAMPLE IDENTIFICATION

Project Number	04-1111-024	Sample Number	4
Borehole Number	06-2	Sample Depth, m	3.0 - 3.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	08/29/2006		
Date Completed	09/12/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m ³	19.55
Sample Diameter, cm	4.42	Drv Unit Weight, kN/m ³	15.56
Area, cm ²	15.35	Specific Gravity, measured	2.76
Volume, cm ³	29.32	Solids Height, cm	1.098
Water Content, %	25.64	Volume of Solids, cm ³	16.86
Wet Mass, g	58.45	Volume of Voids, cm ³	12.46
Dry Mass, g	46.52	Degree of Saturation, %	95.7

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.910	0.740	1.910				
4.99	1.911	0.740	1.911				
9.98	1.911	0.740	1.911	2	3.87E-01	0.00E+00	0.00E+00
20.00	1.909	0.739	1.910	2	3.87E-01	1.05E-04	3.96E-06
40.00	1.905	0.735	1.907	19	4.06E-02	1.05E-04	4.16E-07
80.00	1.897	0.728	1.901	17	4.51E-02	1.05E-04	4.62E-07
160.00	1.881	0.713	1.889	15	5.04E-02	1.05E-04	5.18E-07
320.00	1.854	0.689	1.868	28	2.64E-02	8.84E-05	2.29E-07
639.95	1.813	0.651	1.834	28	2.55E-02	6.71E-05	1.67E-07
1280.00	1.759	0.602	1.786	31	2.18E-02	4.42E-05	9.44E-08
2560.00	1.698	0.546	1.729	60	1.06E-02	2.50E-05	2.58E-08
1280.00	1.705	0.553	1.702				
320.00	1.730	0.576	1.718				
80.00	1.760	0.603	1.745				
20.00	1.783	0.624	1.772				
4.99	1.792	0.632	1.788				

Note:

Specimen swelled under 4.99kPa

k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.79	Unit Weight, kN/m ³	20.57
Sample Diameter, cm	4.42	Drv Unit Weight, kN/m ³	16.58
Area, cm ²	15.35	Specific Gravity, measured	2.76
Volume, cm ³	27.51	Solids Height, cm	1.098
Water Content, %	24.01	Volume of Solids, cm ³	16.86
Wet Mass, g	57.69	Volume of Voids, cm ³	10.65
Dry Mass, g	46.52		

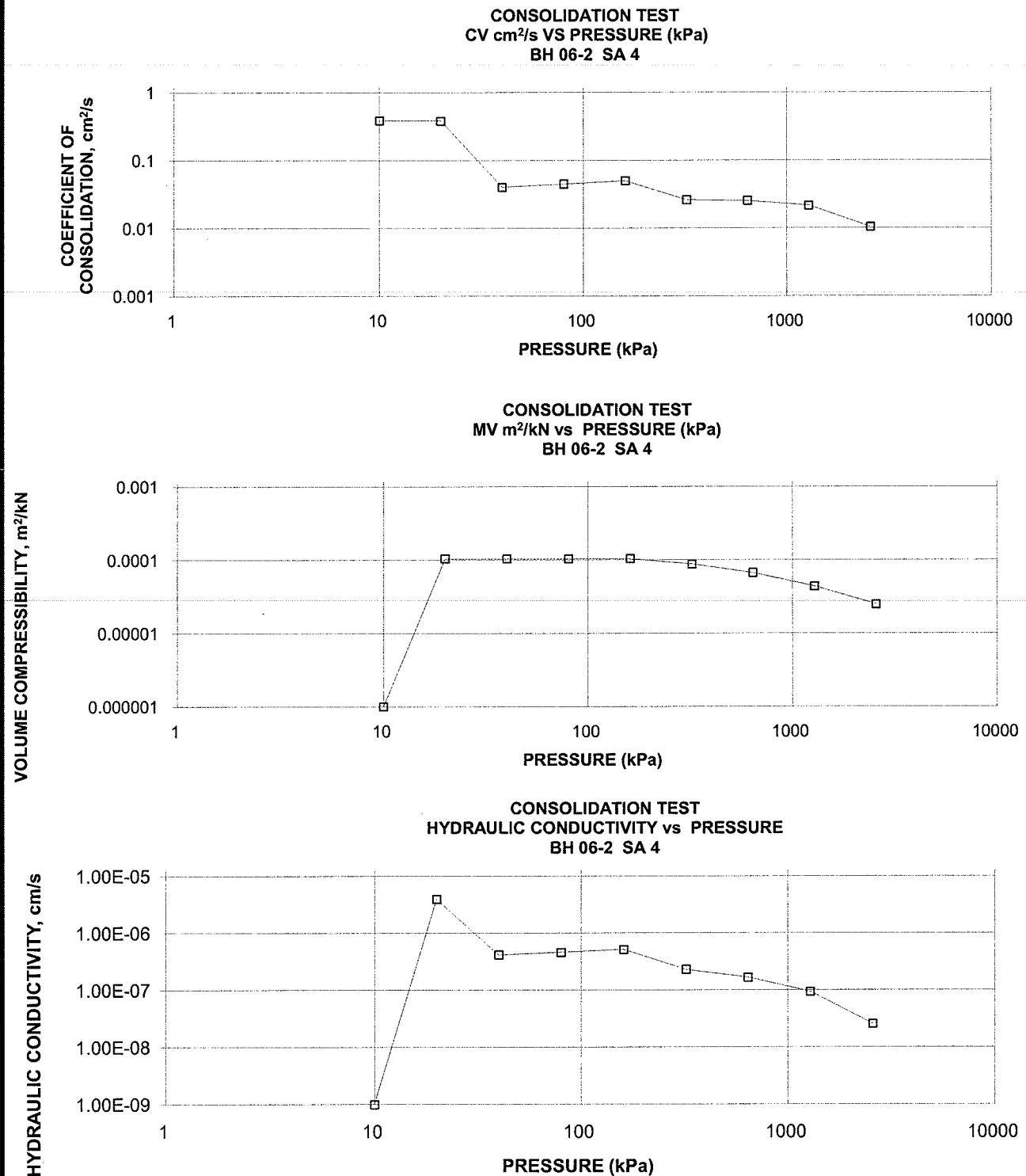
Prepared By: LFG

Golder Associates

Checked By: MM

PLOTS OF CONSOLIDATION TEST RESULTS
Cv, Mv, and K VS. PRESSURE

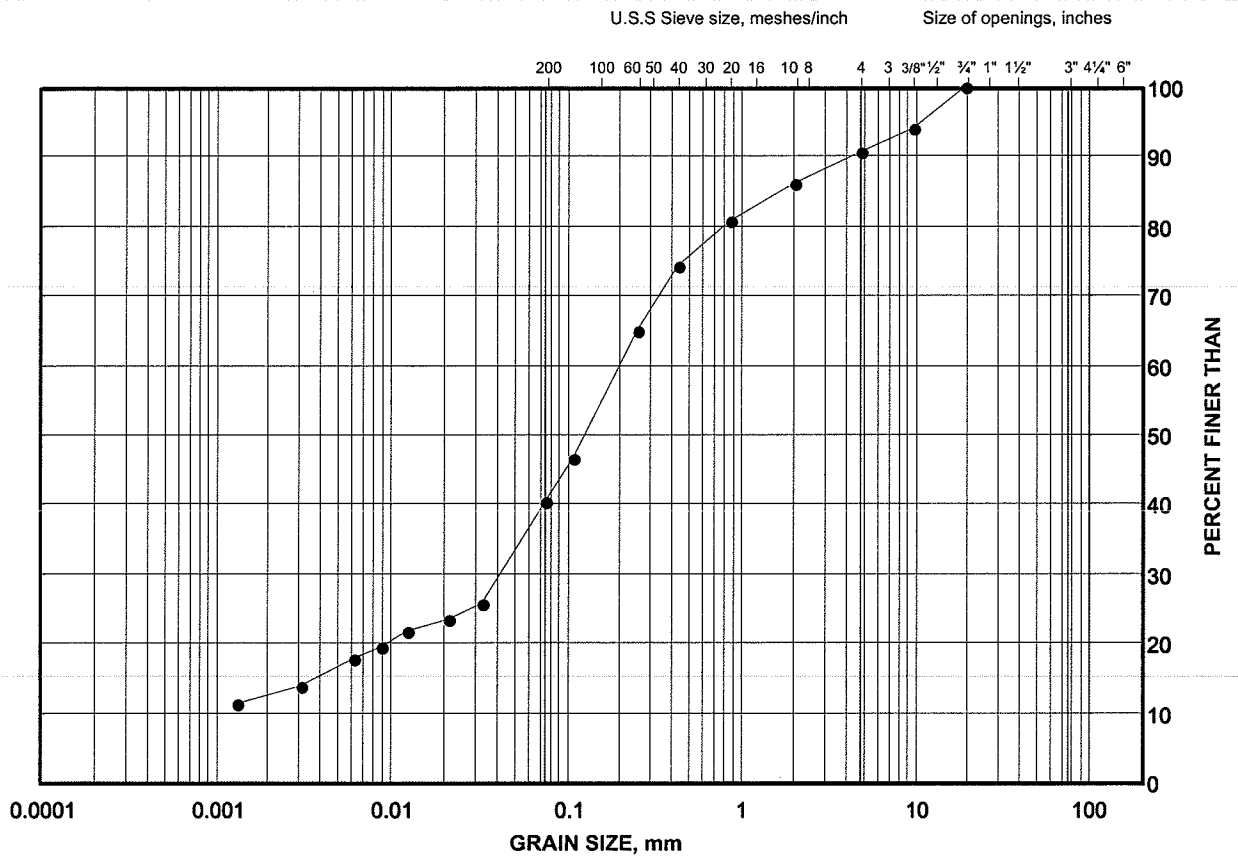
FIGURE A5



GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE A6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-1	5	4.60 - 5.20

Project Number: 04-1111-024C

Checked By: _____

Golder Associates

Date: 02-Feb-07

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

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Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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