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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
PRELIMINARY DESIGN
HIGH FILL AND SWAMP AREAS
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 15
PETERBOROUGH, ONTARIO
G.W.P. 73-99-00**

Submitted to:

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PART A

**FOUNDATION INVESTIGATION AND DESIGN
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by National Capital Engineering Limited (NCE) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation for preliminary design 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 P31-8197, dated August 2003, that forms part of the Consultant's Agreement (P.O. Number 4005-A-000268). Preliminary foundation investigation and design services are required for a total of four structures (i.e. Jackson Creek Bridge, Recreational Trail Culvert and two structural culvert sites) in four separate reports for this project. The scope of foundations work for this project was expanded to include three swamp areas and a high fill area as outlined in Golder's letter titled "Proposal for Addendum Foundation Investigation", dated April 6, 2006 and approved by MTO on April 8, 2006. These areas and their approximate locations are described below.

<i>Area / Station</i>	<i>Approximate Location</i>
High Fill Embankment Area Station 9+750 to 10+000	Lily Lake Road and proposed Highway 7 re-alignment to about 250 m east of existing Highway 7
Swamp Crossing No. 1 Station 23+175 to 23+460	On the proposed Highway 7 realignment about 285 m long at Fowler's Corners
Swamp Crossing No. 2 Station 28+100 to 28+150 and Station 10+000 to 10+050	On Highway 7 Widening at intersection with Parkhill Road West, from about 50 m north to 50 m south of Parkhill Road on east side of existing Highway 7.
Swamp Crossing No. 3 Station 10+400 to 10+600	On Highway 7 Widening between Parkhill Road and Sherbrook Road, about 200 m long on east side of existing Highway 7

This report addresses the preliminary foundation investigation carried out for two swamps (designated as Swamp Nos. 2 and 3) and the high fill embankment area as shown on Drawings 1 to 3. The work was carried out in general accordance with the Quality Control Plan for this project dated April 20, 2006.

Permission to enter private property to access the Swamp No. 1 location was not obtained despite several efforts by Golder and NCE. As a result, no preliminary investigation was carried out at this swamp location. A preliminary investigation should be completed at this location prior to detailed design.

The purpose of this investigation is to establish the subsurface conditions at the high fill and swamp crossing sites by borehole drilling, in situ testing and laboratory testing on selected samples.

2.0 SITE DESCRIPTION

The high fill and swamp crossing sites are located along the existing Highway 7, between Lily Lake Road and Sherbrook Road in Peterborough, Ontario (see key plan on Drawings 1, 2 and 3). The existing highway runs north-south in this area and consists of two lanes, one lane in each direction.

Within the existing MTO right-of-way, the sites generally consist of the raised highway with grass covered embankment side-slopes (sloped at about 2H:1V to 3H:1V). The existing embankments showed no visible signs of significant erosion or distress at the time of our investigation. Flat low-lying areas generally consisting of various vegetation and swamp areas are present on both sides of the existing highway and at the proposed Highway 7 widening and/or re-alignment site. Based on the drawings provided to us titled "Engineering and Title Records", prepared by MTO, dated May 2001, the ground surface elevations along the existing Highway 7 roadway at the proposed High Fill embankment, Swamp No. 2 and Swamp No. 3 crossings are at about 276 m, 248 m and 246 m, respectively. Within the High Fill embankment area, the existing Lily Lake Road is valley-shaped with a high point at Elevation 276 m at the intersection with the existing Highway 7 and a low point at about Elevation 264 m near the middle of the proposed high fill embankment at about Station 9+850 m. Based on the preliminary drawings provided to us, Lily Lake Road is to be raised by up to 8 m above the existing road surface. At the swamp areas, the proposed highway grades are assumed to match the existing Highway 7 profile.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for this investigation was carried out between July 5 and 31, 2006 during which time a total of eight (8) boreholes were advanced; two boreholes at the high fill embankment and shallow cut area, two boreholes at Swamp No. 2 and four boreholes at Swamp No. 3. The boreholes, numbered 06-5 to 06-12 (inclusive), were advanced at the approximate locations shown in plan on Drawings 1 to 3. The boreholes at Swamp Crossing No. 3 were advanced for both the swamp and a culvert (Structure No. 26-186), and have previously been discussed in the separate culvert report.

The field investigation was carried out using a track-mounted CME-55 drill rig supplied and operated by Eastern Soil Investigation Limited of Courtice, Ontario. The boreholes were advanced using 101 mm outside diameter (O.D.) solid stem augers. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. The boreholes were advanced and sampled to depths of about 9.3 m to 12.3 m below ground surface.

Dynamic Cone Penetration Tests (DCPTs) were carried out in boreholes 06-5 to 06-8 (inclusive) from a depth of 9.8 m below ground surface and were terminated on effective refusal (i.e. greater than 100 blows per 0.3 m of penetration) at depths ranging from about 14.3 m to 21.6 m below ground surface.

The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in boreholes 06-5 and 06-9 to permit monitoring of the groundwater level at the swamp locations. The piezometers consisted of 50 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 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 and 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 Mississauga 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. Specialized laboratory consolidation testing was carried out on one sample from borehole 06-5 at the Swamp No. 3 location.

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 a licensed surveyor (i.e. Transenco Limited) and are summarized below and on Drawings 1 to 3.

<i>Site Location</i>	<i>Borehole Number</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
High Fill Embankment Area Station 9+750 to 10+000	06-11	4907547.0	390222.4	265.7
	06-12	4907501.9	390114.3	275.1
Swamp Crossing No. 2 Station 28+100 to 28+150 and Station 10+000 to 10+050	06-9	4906151.5	390580.5	246.5
	06-10	4906201.7	390567.1	246.2
Swamp Crossing No. 3 Station 10+400 to 10+600	06-5	4905696.0	390739.5	243.5
	06-6	4905673.2	390748.7	243.3
	06-7	4905705.4	390726.1	244.7
	06-8	4905746.5	390721.6	243.1

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: High Fill Embankment Area

The following sections discuss the findings of the borehole investigation carried out at the high fill embankment area on Lily Lake Road as shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory testing are given on the Record for Borehole Sheets Nos. 06-11 and 06-12, and on Figures A1 to A3 in Appendix A following the text of this report. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsoils at the high fill embankment area consist of a surficial layer of asphalt (i.e. the existing Lily Lake Road surface), underlain by sand and gravel fill. The asphalt and fill is underlain by a layer of silty sand and gravel containing trace amounts of organics which is underlain by a deposit of silty sand till. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt and Fill Materials

Boreholes 06-11 and 06-12 were advanced through the existing pavement structure on Lily Lake Road which was measured to be approximately 100 mm and 75 mm thick, respectively.

Underlying the asphalt layer, a deposit of sand and gravel fill containing trace to some silt, trace clay, trace organics was encountered. In borehole 06-11, auger refusal on an obstruction (inferred

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.

boulder) was encountered within the granular fill at a depth of about 2.7 m below ground surface. The fill material extended to a depth of 2.8 m (Elevation 272.2 m) in borehole 06-12 and 5.7 m depth (Elevation 259.9 m) in borehole 06-11.

Standard Penetration Test (SPT) 'N' values recorded within the fill materials ranged between 9 blows per 0.3 m of penetration and 50 blows per 0.1 m of penetration indicating a loose to very dense but generally compact to very dense relative density.

The laboratory natural water content measured on three samples of the sand and gravel fill ranged between about 3 per cent and 5 per cent. A grain size distribution performed on a select sample of the sand and gravel fill is shown on Figure A1 and indicates the well graded nature of the fill.

4.2.2 Silty Sand to Silty Sand and Gravel

Underlying the fill materials, a layer of native silty sand to silty sand and gravel containing trace to some clay, trace silt and organics was encountered in both boreholes 06-11 and 06-12. The top of this native granular layer was encountered at a depth of 5.8 m and 2.9 m and was 1.5 m and 1.4 m thick at boreholes 06-11 and 06-12, respectively.

SPT 'N' values measured within the silty sand to silty sand and gravel layer were 13 and 21 blows per 0.3 m of penetration, indicating a compact relative density.

The natural water content measured on two samples of the native granular layer were 10 per cent and 15 per cent. Figure A2 shows the results of a grain size distribution performed on a sample of the of the silty sand and gravel.

4.2.3 Silty Sand Till

Underlying the native silty sand to silty sand and gravel layer, a silty sand till deposit was encountered in the boreholes. The glacial till deposit as recovered in the sampler was principally silty sand and contained trace to some gravel and clay. The samples contained some interlayers of sand and gravel. The drilling encountered some cobbles and boulders. The top of this layer was encountered at depths of 7.3 m and 4.3 m, corresponding to Elevations 258.4 m and 270.8 m for boreholes 06-11 and 06-12, respectively. The till deposit was penetrated about 5 m in both boreholes and drilling was terminated at depths of 12.3 m (Elevation 253.4 m) and 9.3 m (Elevation 265.8 m) within this deposit for boreholes 06-11 and 06-12, respectively.

SPT 'N' values measured within the silty sand till deposit ranged from 13 blows per 0.3 m of penetration near the surface to 100 blows per 0.1 m of penetration at depth. The 'N' values are generally greater than 100 blows per 0.3 m of penetration about 1 m into the till deposit. The 'N'

values indicate that the upper 1 m of the till deposit has a compact to dense relative density and beneath this, the till becomes very dense.

The natural water content measured on samples of the silty sand till ranged between about 6 per cent and 13 per cent. A higher water content of about 22 per cent was measured within the upper metre of the till in borehole 06-11, directly below the saturated sand and gravel layer. Grain size distribution curves for three selected samples of the till deposit are shown on Figure A3.

4.3 Subsoil Conditions: Swamp Crossing No. 2

The following sections discuss the findings of the borehole investigation carried out for the Swamp Crossing No. 2 located at the northeast and southeast corners of the intersection of Highway 7 and Parkhill Road West as shown on Drawing 2. Permission to enter the private property at the southeast corner was not given despite several requests to the owner by NCE; therefore, borehole 06-9 was located as close as possible to the property limits. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory testing are given on the Record of Borehole Logs 06-9 and 06-10, and on Figures A4 to A7 in Appendix A following the text of this report. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsoils at Swamp Crossing No. 2 consist of a surficial layer of topsoil underlain by a clayey silt layer, underlain by a silt layer containing interlayers of clayey silt. The silt layer is underlain by a silty sand to sandy silt till deposit. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.3.1 Topsoil

A surficial layer of topsoil, about 0.3 m thick, was encountered in boreholes 06-9 and 06-10.

4.3.2 Clayey Silt

A predominantly clayey silt layer containing trace to some sand and gravel was encountered below the topsoil in boreholes 06-9 and 06-10. The clayey silt layer contained silt and sandy silt interlayers in borehole 06-9. The clayey silt layer was 2.0 m and 1.7 m thick in boreholes 06-9 and 06-10, respectively.

SPT 'N' values measured within the clayey silt layer ranged from 5 to 18 blows per 0.3 m of penetration suggesting a firm to very stiff consistency.

Two water contents measured on samples of the clayey silt layer were 24 and 22 per cent. The results of an Atterberg limits test carried out on a sample of the clayey silt gave a plastic limit of 18 per cent, a liquid limit of 26 per cent, and a corresponding plasticity index of 8 per cent. The Atterberg limit test results are plotted on a plasticity chart on Figure A4 and indicate a clayey silt of low plasticity. A grain size distribution performed on a sample within the clayey silt layer is shown on Figure A5 and suggests the presence of frequent silt interlayers.

4.3.3 Silt

Underlying the clayey silt layer, a predominantly silt layer was encountered containing some clay, trace sand and interlayers of clayey silt in both boreholes 06-9 and 06-10. In borehole 06-9, interlayers of sandy silt were also encountered. The top of the silt layer was encountered at a depth of 2.3 m (Elevation 244.2) and 2.0 m (Elevation 244.2 m) and was 2.4 m and 3.2 m thick in boreholes 06-9 and 06-10, respectively.

SPT 'N' values measured within the silt layer ranged from 15 to 32 blows per 0.3 m of penetration, indicating a compact to dense relative density.

Natural water contents measured on samples of the silt layer ranged between 8 per cent and 21 per cent; the higher water contents likely indicating a higher silt content and lower water contents likely indicating the presence of sandy silt interlayers. A grain size distribution was performed on a selected sample of the silt and is shown on Figure A6. An Atterberg limits test was performed on a sample of the silt containing clayey silt interlayers and measured a plastic limit of 11 per cent, a liquid limit of 27 per cent, and corresponding plasticity index of 16 per cent. The result of the Atterberg limits test is plotted on the plasticity chart on Figure A4 and indicate clayey silt interlayers of low plasticity are present within the silt layer.

4.3.4 Silty Sand to Sandy Silt Till

A deposit of silty sand to sandy silt till was encountered below the silt layer in both boreholes 06-9 and 06-10. The glacial till deposit of silt and sand material contained some gravel, trace to some clay, and contained sand and gravel interlayers. Cobbles and boulders were encountered during drilling. The top of the till deposit was encountered at depths of about 4.6 m and 5.2 m, corresponding to Elevations 241.9 m to 241.0 m, for boreholes 06-9 and 06-10 respectively. The till deposit had a measured thickness of 4.6 m in borehole 06-10 and borehole 06-9 was terminated within this till deposit after 6.2 m of penetration and at a depth of 10.8 m (Elevation 235.7 m).

SPT 'N' values measured in the cohesionless till deposit ranged from 0 blows (weight of hammer) per 0.3 m of penetration to 117 blows per 0.15 m of penetration; however, the N values are generally greater than 100 blows per 0.3 m of penetration from about 2 m below the till

surface. Therefore, the above 'N' values indicate that the upper 2 m of the till deposit has a very loose to compact relative density and beneath this, the till becomes very dense. The low 'N' values within the upper 2 m of the till in borehole 06-9 may be the result of wet sandy interlayers blowing into the augers.

The natural water content measured on three samples of the till ranged between about 6 per cent and 9 per cent. Grain size distributions were performed on three selected samples of the till deposit and are shown on Figure A7.

4.3.5 Sand to Sand and Gravel

A sand to sand and gravel deposit was encountered beneath the till at borehole 06-10. The top of the granular deposit was encountered at a depth of 8.8 m (Elevation 237.4 m) with a measured thickness of 2 m before the borehole was terminated within this layer at a depth of 10.8 m (Elevation 235.4 m).

Two SPT 'N' values measured within the sand to sand and gravel deposit were 60 and 50 blows per 0.1 m of penetration, indicating a very dense relative density. A single natural water content measured on a selected sample of the sand was 16 per cent.

4.4 Subsoil Conditions: Swamp Crossing No. 3

The following sections discuss the findings of the borehole investigation carried out for Swamp Crossing No. 3 located on the east side of Highway 7 and south of Parkhill Road West as shown on Drawing 3. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory testing are given on the Records of Boreholes 06-5 to 06-8 (inclusive), and on Figures A8 to A13 in Appendix A following the text of this report. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsoils at Swamp Crossing No. 3 consist of a surficial layer of topsoil and/or peat, underlain in places by a layer of silty sand, overlying a deposit of clayey silt to silty clay containing frequent interlayers of silt and sand. The clayey silt/silty clay is underlain by a deposit of sandy silt to silt. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.4.1 Topsoil / Peat

Topsoil and/or peat was encountered at the ground surface in all of the boreholes (i.e. boreholes 06-5 to 06-8). A surficial layer of topsoil (about 0.8 m thick) was encountered in borehole 06-7, underlain by a 1.5 m thick layer of peat containing wood fragments. Fibrous peat was encountered at the ground surface in the remaining boreholes (06-5, 06-6 and 06-8) and ranged in thickness from about 0.8 m to 2.3 m.

Standard Penetration Test (SPT) 'N' values recorded within the topsoil and peat typically ranged between 0 (i.e. weight of hammer) and 3 blows per 0.3 m of penetration indicating a very soft to soft consistency. One 'N' value of 12 was recorded within the peat in borehole 06-7; however, the higher value is likely attributed to wood fragments present within the peat soil.

4.4.2 Silty Sand

Beneath the peat and topsoil, a layer of silty sand containing trace gravel and clay was encountered in boreholes 06-7 and 06-8. The top of the silty sand layer was encountered at a depth of about 2.3 m and 0.8 m, corresponding to Elevation 242.4 m, at boreholes 06-7 and 07-8, respectively. The thickness of the silty sand layer was about 1.4 m and 1.9 m at boreholes 06-7 and 07-8, respectively.

Standard Penetration Test (SPT) 'N' values recorded within the silty sand layer ranged between 0 (i.e. weight of hammer) and 20 blows per 0.3 m of penetration, but typically between about 3 and 10 blows per 0.3 m of penetration, indicating a generally very loose to compact relative density.

The natural water content measured on two samples of the silty sand layer were 16 per cent and 17 per cent.

4.4.3 Clayey Silt to Silty Clay

A clayey silt deposit containing trace to some sand, trace gravel was encountered below the peat and/or silty sand in all of the boreholes. In borehole 06-6, the upper portion of this stratum is described as a clayey silt to a silty clay. The clayey silt contained frequent interlayers of sand and silt throughout the deposit. In borehole 06-5, the interlayers of sand and silt were more predominant within the upper metre of the deposit. The top of the clayey silt/silty clay deposit was encountered at depths ranging between about 1.1 m and 3.7 m, corresponding to between about Elevations 240.4 m and 242.4 m. The thickness of the clayey silt/silty clay deposit varied between about 4.6 m and 7.6 m.

Standard Penetration Testing (SPT) 'N' values recorded within the clayey silt/silty clay deposit ranged between 2 blows and 24 blows per 0.3 m of penetration, indicating a very soft to very stiff

consistency. The large range in 'N' values can be attributed to the presence of frequent silt and sand interlayers.

A field vane test to measure the undrained shear strength of the clayey silt was performed in borehole 06-5 at a depth of 4.1 m (Elevation 239.3 m). The shear strength of the clayey silt was greater than 100 kPa indicating a very stiff consistency at this location. The high measured strength may be attributed to the presence of silt and sand interlayers.

The natural water content measured on selected samples of the clayey silt/silty clay deposit ranged between about 18 per cent and 32 per cent. A grain size distribution curve for a selected sample of the silty clay is shown on Figure A8 in Appendix A.

The results of Atterberg Limits testing carried out on six samples of the clayey silt/silty clay deposit are illustrated on the plasticity chart on Figure A9 in Appendix A. The test results are summarized below and indicate the clayey silt generally is of low plasticity.

<i>Borehole</i>	<i>Sample</i>	<i>Elevation (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>	<i>Moisture Content (%)</i>
06-5	4	239.8 – 240.4	33	17	16	22
06-5	5	238.3 – 238.9	18	14	4	24
06-6	4	239.6 – 240.2	26	15	11	20
06-6	7	235.1 – 235.7	17	11	6	22
06-7	7	236.5 – 237.1	18	12	6	20
06-8	6	236.4 – 237.0	18	11	7	21

A laboratory consolidation test was carried out on a single specimen of the clayey silt 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^* (cm²/s)
06-5, Sa#4	3.4 m/240.1 m	25	320	295	13	0.20	0.02	0.71	4.5×10^{-2}

Note: * For stress range of $20 \leq \sigma_v' \leq 160$ kPa

where: σ_{vo}' is the effective overburden pressure in kPa
 σ_p' is the 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

The clayey silt sample used for the consolidation test (i.e. from borehole 06-5, Sa#4) is considered to be over-consolidated. A bulk unit weight of about 20 kN/m³ and a specific gravity of 2.76 was measured on the consolidation test specimen. The consolidation test results are shown on Figures A10 to A13 in Appendix A.

4.4.4 Sandy Silt to Silt

A layer of sandy silt to silt, some sand was encountered below the clayey silt deposit in all of the boreholes. The sandy silt to silt contained trace to some gravel and clay. The top of this layer was encountered at depths ranging between about 7.3 m and 8.8 m, corresponding to between about Elevations 234.8 m and 235.8 m. The sandy silt to silt layer was penetrated between about 1.0 m and 2.5 m and all boreholes were terminated within this layer, below which depth Dynamic Cone Penetration Tests (DCPT) were advanced in all boreholes. The DCPTs were terminated upon effective refusal at depths ranging from about 14.3 m (Elevation 228.8 m) to 21.6 m (Elevation 223.1 m). The apparent zig-zag pattern of the DCPTs suggests that interlayers of clayey and granular soils continues with depth.

Standard Penetration Testing (SPT) 'N' values recorded within the sandy silt to silt ranged from 0 (i.e. the weight of hammer) to 18 blows per 0.3 m of penetration, indicating a very loose to compact relative density. The DCPT results suggest a compact to very dense soil at depth.

The natural water content measured on four samples of the sandy silt to silt ranged between about 14 per cent and 20 per cent. Grain size distribution curves for two selected samples from the silt (to sandy silt) deposit are shown on Figure A14 in Appendix A.

4.4.5 Groundwater Conditions

The water levels were noted within the open boreholes at the time of the drilling operations. Piezometers were installed in boreholes 06-5 in Swamp No. 3 and 06-9 in Swamp No. 2. The piezometers were sealed into the lower sandy silt to silty sand deposit, below the clayey silt deposit in borehole 06-5 and into the silty sand till in borehole 06-9. Details of the piezometer installations are shown on the Record of Borehole Sheets following the text of this report. The water levels measured in the piezometers and open boreholes upon completion of drilling are summarized below.

Site	Borehole	Installation	Ground Surface Elevation (m)	Depth to Water Level(m)	Water Level Elevation (m)	Date
High Fill Embankment Area	06-11	Open Borehole	265.7	6.0	259.7	July 10, 2006
	06-12	Open Borehole	275.1	7.9	267.2	July 31, 2006
Swamp Crossing No. 2	06-9	Piezometer	246.5	8.7	237.8	July 7, 2006
				1.9	244.6	July 10, 2006
				0.4	246.1	July 31, 2006
				1.1	245.4	Aug. 18, 2006
	06-10	Open Borehole	246.2	6.1	240.1	July 7, 2006
Swamp Crossing No. 3	06-5	Piezometer	243.5	7.3	236.2	July 5, 2006
				3.8	239.7	July 6, 2006
				-0.9*	244.4	July 10, 2006
				-0.8*	244.3	July 31, 2006
				-1.0*	244.5	Aug. 18, 2006
	06-6	Open Borehole	243.3	0.2	243.1	July 6, 2006
	06-7	Open Borehole	244.7	2.3	242.4	July 6, 2006
	06-8	Open Borehole	243.1	0.6	242.5	July 6, 2006

Note : * Artesian Conditions

It should be noted that the piezometer readings indicate artesian conditions from within the sandy silt up to about 1 m above the ground surface at borehole 06-5 in readings from July 10 to August 18, 2006. The piezometric readings in borehole 06-9 gave readings of 0.4 m and 1.1 m depth on July 31 and August 18, 2006. Groundwater levels at the three sites 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. Suresh Bainey. This report was prepared by Ms. Shannon Palmer, EIT, and reviewed by Mr. Kevin J. Bentley, P.Eng. Fintan J. Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.


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SLP/KJB/FJH/kjb/al

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PART B

**PRELIMINARY FOUNDATION DESIGN
HIGH FILL AND SWAMP AREAS
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 15
PETERBOROUGH, ONTARIO
G.W.P. 73-99-00**

6.0 PRELIMINARY ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the preliminary design of the proposed high fill embankments and swamp crossings as part of the proposed highway operational improvements and future four laning of Highway 7. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the preliminary subsurface investigation at these sites. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out preliminary design of the project. Where comments are made on construction they are provided in order to highlight those aspects which could affect the preliminary design of the project and for which, ultimately, provision will have to be made at the detail design stage of the project as the contract documents are prepared.

Further additional borehole drilling will be required during the future planning once detailed designs of the project are finalized. At that time, investigation of Swamp Crossing No. 1 will need to be carried out since permission to enter private property was not obtained during the recent investigation. Additional investigation is also recommended in some areas of the Swamp Crossing No. 2 and No. 3 locations as permission to enter private property was not given at the southeast portion of Swamp No. 2 and the east limit of Swamp No. 3. Further investigation is required to delineate the thickness of peat/topsoil and determine the extent and consistency of the underlying clayey silt / silty clay soils. The additional work, as discussed in subsequent sections of this report, will refine the predicted magnitude and time rate of settlement under the new embankment loading and to further assess any stability / settlement mitigation options and develop the necessary operational constraints and/or special provisions for the contract.

6.1 General

Existing site plans and profiles were obtained from MTO drawings provided to us titled "Engineering and Title Records", dated May 2001. Preliminary design site plans were provided to us in digital format by NCE on March 31, 2006. There were limited preliminary profile drawings available at the time of this report; however, based on conversations with the designer, it was assumed that the proposed Highway 7 re-alignment / widening road grade would match the existing Highway 7 profile at the swamp locations and at the high fill embankment location, a preliminary grading plan was provided to us. The following table summarizes the approximate existing and proposed pavement elevations and anticipated embankment heights for the swamp crossing and embankment fill areas.

<i>Site</i>	<i>Existing Road Grade</i>	<i>Proposed Road Grade</i>	<i>Approximate Station</i>	<i>Proposed Embankment Height</i>
High Fill Embankment (Lily Lake Road)	264.2 m to 276.4 m	272 m to 275 m	9+750 to 10+000	*4 m to 12 m
Swamp No. 2 (Highway 7)	248.8 m to 248.9 m	Match existing	28+100 to 28+150 10+000 to 10+050	2.5 m to 3 m
Swamp No. 3 (Highway 7)	246.0 m to 246.1 m	Match existing	10+420 to 10+580	2.5 m to 3 m

*Existing high fill embankment is about 4 m to 6 m high.

It is understood that the proposed Highway 7 re-alignment / widening will cross the Swamp No. 2 and No. 3 locations which are currently low lying areas. Future planning also indicates that the existing Highway 7 will be reconstructed to accommodate turning lanes at the Swamp No. 2 location. Where the highway is to be widened and/or re-aligned at the swamp locations, it is anticipated that the proposed grades will be constructed to match the existing grades, typically resulting in grade raises of up to 3 m. At the Lily Lake Road high fill embankment location, it is understood that the new Highway 7 re-alignment will be lower than the existing Highway 7. However, the existing embankment along Lily Lake Road directly east of the proposed Highway 7 re-alignment is proposed to be raised and flattened as part of the construction.

The borehole locations and interpreted soil strata at the three sites are shown on Drawings 1 to 3. The following sections discuss the preliminary design recommendations for each site in further detail and are summarized on Table 1.

6.2 High Fill Embankment

The existing Lily Lake Road embankment (from about Station 9+750 to 10+000) presently measures up to about 6 m high and has grass covered side slope profiles of approximately 2 horizontal to 1 vertical (2H:1V). The existing embankment showed no visible signs of significant erosion or distress at the time of our investigation. Currently, these embankments do not have a mid-height bench.

Based on the borehole results, the existing embankment consists of up to about 6 m of sand and gravel fill containing trace organics. The fills were generally dense to very dense, however, loose to compact zones were generally encountered near the bottom third of the fill deposit. Below the fill, a compact silty sand and gravel layer was present that contained trace organics, underlain by compact to very dense silty sand till.

The existing embankment supporting Lily Lake road will require placement of up to about 8 m of embankment fill above the existing road grade. Assuming the existing ground surface at the toe of the existing embankment is at about Elevation 260 m (based on base of fill in borehole 06-11) and the proposed road grade in this area is about Elevation 272 m, the proposed embankment will be up to 12 m in height. At the east and west limits of the grade raise, the proposed road grade will tie into the existing Lily Lake Road and proposed Highway 7 grade, respectively. The extent of the fill area extends from about Station 9+750 to 10+000; however, the actual extent of the “high fill” area (assuming a minimum 4.5 m high embankment in accordance with MTO typical terms of reference) will need to be determined once a survey of the existing embankment and adjacent topography is provided during the detail design stage. A ground contour plan of this area was not available at the time this report was prepared.

It is important to note that an existing corrugated steel pipe (CSP) runs beneath Lily Lake Road at about Station 9+865; which will need to be either replaced or widened as part of the construction. It is estimated that the existing culvert is currently located below about 6 m of existing embankment fill and after the proposed grade raise, will be located below about 12 m of embankment fill. The design of this culvert should be addressed during the detailed design stage.

6.2.1 Fill Embankment Construction

Prior to the placement of any engineered fill for the new embankment construction, grade raise and widening, all vegetation, topsoil or soils containing significant organics and/or deleterious material should be stripped from below the proposed embankment widening and grade raise footprint areas and wasted/reused for landscaping. The foundation footprint not only includes the area outside the toe of the existing embankment but also includes the areas where fill is to be placed on top of the existing embankment. In areas where fill is to be placed on top of the existing road surface, it is recommended that the existing asphalt pavement structure be pulverized (and recycled if possible) prior to any fill placement. All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with SP206S03.

Benching into the existing embankment side slopes should be carried out as per OPSD 208.010 for construction of the embankment widening and grade raise to ensure adequate keying of the new fill into the existing fill.

Based on the existing subsoil conditions, either earth fill or rock fill embankment options may be considered. The different fill alternatives (i.e. earth fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils / bedrock), construction cost and time, and ease of construction / availability. Earth fill has typically been used in this area in the past; however, if rock fill is being considered, more details

can be provided at the detailed design stage. Construction of an earth fill embankment above the prepared subgrade may be carried out using Select Subgrade Material (SSM) meeting OPSS 1010. All embankment fill should be placed in regular loose lifts not exceeding 300 mm, and compacted to at least 95 percent of the material Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 percent of the SPMDD. 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.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding should be placed as soon as possible in accordance with OPSS 572.

Additional boreholes should be carried out during the detail design stage of the project adjacent to Lily Lake Road at the toe of the existing embankment to confirm the presence/absence of topsoil and/or organic material beyond the footprint of the existing embankment.

6.2.2 Embankment Stability

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W (produced by Geo-Slope International Ltd.), employing the Morgenstern-Price method of analyses, to check that a minimum factor of safety of 1.3 is achieved against deep-seated, global type failures that would impact the operation of the roadway.

Static slope stability analyses that examine the global stability for the high fill embankment was carried out using the following parameters based on field and laboratory test data and accepted correlations:

<i>Soil Layer / Deposit</i>	<i>Bulk Unit Weight (kN/m³)</i>	<i>Effective Friction Angle (ϕ', degrees)</i>
New Embankment fill	21	32
Existing Embankment Fill (Upper dense to very dense sand and gravel fill)	21	35
Existing Embankment Fill (Lower loose to compact sand and gravel fill)	19	30
Silty Sand and Gravel	20	30
Silty Sand Till	22	37

For the purpose of the analyses, it was assumed that the subgrade was properly prepared and proper placement and compaction of the engineered fill embankment was performed. Select Subgrade Material (i.e. granular earth fill) was assumed to be used for the engineered fill for the construction of the proposed high fill embankment. The piezometric conditions used in the

analyses were based on the groundwater levels noted during drilling in the boreholes in this area which resulted in a water level located at the base of the fill.

A maximum embankment height of 12 m and side slopes at 2H:1V were assumed (with no bench). For the proposed 12 m high embankment (i.e. up to 8 m grade raise), a Factor of Safety of 1.3 or greater was calculated against static deep-seated slope instability.

While we consider that the seismic stability of the embankment should also be adequate given that the site is located in a low acceleration-related seismic zone, analyses should be performed at the detailed design stage.

For granular earth fill, the incorporation of a 2 m wide mid-height bench (berm) into the uniform side slope profile is required wherever the embankment will exceed a height of 8 m (as per OPSD 202.010). The incorporation of a bench will further increase the Factor of Safety against instability.

6.2.3 Embankment Settlement

Settlement analyses were carried out for the new high fill embankment along Lily Lake Road using the results from the boreholes, in situ field test data (SPT), and laboratory tests to estimate the deformation parameters of the subsoils. For these analyses, the critical sections are assumed to correspond to the greatest new embankment height of 8 m above existing ground surface located at about Station 9+825. The unit weights and slope profiles for the embankment fill described in the previous section were employed in the analyses. The total net loading on the foundation soils (after stripping, backfilling and embankment fill construction) was estimated to be about 175 kPa.

As noted previously, the subsoils encountered within the limits of the project site generally consist of sand and gravel fill overlying a layer of silty sand and gravel, underlain by a silty sand till deposit. The sand and gravel fills and silty sand and gravel layer contained trace organics within the boreholes. Based on the results of the boreholes, settlement of the cohesionless foundation and embankment soils is expected to occur during or shortly after construction. Settlement of the new embankments will also occur due to compression of the engineered fill itself.

For preliminary design purposes, it is predicted that immediate settlement due to compression of the existing cohesionless embankment and foundation soil layers will be less than 50 mm. Provided that the new embankment fill material consists of granular earth fill, the settlement of the new embankment fill itself is expected to be less than 25 mm. Therefore, the total settlement of the high fill embankment is estimated to be less than 75 mm. These settlements are expected to

occur rapidly (i.e. during or shortly after construction) in response to the filling based on the granular nature of the native and fill soils.

It is noted that these settlements are conditional based on the actual composition and consistency of the existing embankment fill materials. Once the final Lily Lake Road road profile is designed, additional boreholes should be performed at the locations of maximum grade raise (i.e. fill placement) to confirm the consistency of the underlying existing embankment fill and upper native soils containing organics and revise the settlement predictions as appropriate.

Considering the embankment is underlain by predominantly compact to very dense silty sand till soils containing cobbles and boulders, liquefaction is not considered to be a significant concern. However, the compact silty sand and gravel layer present between the existing fill and the till soils should be assessed for liquefaction potential during the detailed design stage of the project.

6.3 Swamp Crossing No. 2

6.3.1 Subgrade Preparation and Embankment Construction

The location of the proposed new Highway 7 embankment crossing over Swamp Crossing No.2 and widening of the existing Highway 7 embankment is shown on Drawing 2. The preliminary design indicates up to about 3 m of embankment fill will be placed above the existing ground surface to match the existing highway grade.

Based on the borehole results, the subgrade soils in the immediate area of Swamp Crossing No. 2 consist of a thin layer of topsoil underlain by firm to very stiff clayey silt layer, underlain by a compact silt layer. The silt layer is underlain by a typically compact to very dense sandy silt to silty sand till deposit. A deposit of very dense sand to sand and gravel was encountered at depth below the till deposit in borehole 06-10.

Prior to the placement of any engineered fill for the new embankment construction and/or widening, all vegetation, topsoil or soils containing significant organics and/or deleterious material should be stripped from below the proposed embankment footprint areas and wasted/reused for landscaping. Based on the results of the preliminary boreholes, up to about 1 m of the surficial topsoil / organics and soft / loose soils will need to be stripped and backfilled in accordance with Section 6.5.1.

The unsuitable topsoil, peat and soft / loose soils containing excessive organics should be completely removed below the anticipated embankment fill footprint in accordance with OPSD 203.010 for proposed new embankments and in accordance with OPSD 203.020 for proposed widening to existing embankments. It is anticipated that conventional excavators will be required

for this work and that the sequence of sub-excavation and backfilling may have to be carried out in a series of small stages.

For the widening component of the existing Highway 7 embankment, the stripping will include portions of the existing embankment side-slopes. All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with SP206S03. Benching into the existing embankment side slopes should be carried out as per OPSD 208.010 for construction of the embankment widening to ensure adequate keying of the new fill into the existing fill.

Based on the existing subsoil conditions, either earth fill or rock fill embankment options may be considered. The different fill alternatives (i.e. earth fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils / bedrock), construction cost and time, and ease of construction / availability. Earth fill has typically been used in this area in the past; however, if rock fill is being considered, details can be provided at the detailed design stage. Construction of an earth fill embankment above the prepared subgrade may be carried out using Select Subgrade Material (SSM) meeting OPSS 1010. All embankment fill should be placed in regular loose lifts not exceeding 300 mm, and compacted to at least 95 percent of the material Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 percent of the SPMDD. 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.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding should be placed as soon as possible in accordance with OPSS 572.

Additional boreholes should be carried near the centre and east side of the proposed new embankment during the detail design stage of the project to confirm the presence/absence of topsoil or peat and to further assess the thickness and consistency of the underlying clayey soils.

6.3.2 Stability

For the embankment widening, assuming there is no grade raise and side-slopes are 2H:1V or shallower (i.e. same properties as the existing embankment), the global stability of the embankment is considered to be adequate.

For the new embankment to be constructed as part of the future four-laning of Highway 7, analyses were performed to assess the stability of the highest anticipated embankment section. Limit equilibrium slope stability analyses were performed using the commercially available

program SLOPE/W (produced by Geo-Slope International Ltd.), employing the Morgenstern-Price method of analyses, to check that a minimum factor of safety of 1.3 is achieved against deep-seated, global type failures that would impact the operation of the roadway.

Static slope stability analyses that examine the global stability for the embankment was carried out using the following parameters based on field and laboratory test data and accepted correlations:

<i>Soil Deposit</i>	<i>Unit Weight (kN/m³)</i>	<i>Effective cohesion (kPa)</i>	<i>Effective Friction Angle (ϕ', degrees)</i>	<i>Undrained Shear Strength (kPa)</i>
Embankment Fill	21	0	32	–
Firm to Very Stiff Clayey Silt	18	0	–	75
		0	28	–
Compact to Dense Silt	19	0	28	–
Very loose to Compact Till (upper 2 m)	21	0	30	–
Compact to Very Dense Till and Sand to Sand and Gravel	21	0	35	–

For the purpose of the analyses, it was assumed that all topsoil / organics were removed, the subgrade properly prepared, and proper placement and compaction of the SSM engineered fill embankment was performed. The piezometric conditions used in the analyses were based on the groundwater levels noted in the piezometer indicated a water level up to 0.4 m below ground surface (Elevation 246.1 m). Both undrained and effective stress analyses were carried out.

For the proposed 3 m high embankment (assuming 2H:1V side slopes), a Factor of Safety of greater than 1.3 was calculated against static deep-seated slope instability.

While we consider that the seismic stability of the embankment should also be adequate given that the site is located in a low acceleration-related seismic zone, analyses should be performed at the detailed design stage.

6.3.3 Settlement

For the proposed new embankment and widening, some settlement is anticipated due to the loading imposed on the subsoils from the new embankment loading. Based on the preliminary plan drawings and conversations with the designer, it is understood that the new and widened embankment will match the existing Highway 7 road grade (about Elevation 249), resulting in a new embankment fill height of up to about 3 m above the existing ground surface.

Settlement analyses were carried out for the proposed new embankment fill over the Swamp Crossing No. 2 location using the boreholes, in situ field test data (SPT), and laboratory tests to estimate the deformation parameters of the subsoils. For the settlement analysis, it is assumed that the new embankment and widening footprint will be stripped of topsoil and surficial loose/soft materials and replaced with engineered fill.

The following sections describe the estimated settlement of the foundation soils and embankment fill itself due to the loading imposed by the new embankment. The total net loading on the foundation soils (after stripping, backfilling and embankment fill construction) was estimated to be about 60 kPa.

6.3.3.1 Settlement of Foundation Soils

The immediate compression of the compact to dense silt, compact to very dense silty sand to sandy silt till, and very dense sand to sand and gravel granular soils were modelled using elastic modulus of deformation based on the SPT 'N' values and laboratory tests to estimate the deformation parameters of the subsoils.

The time-dependant settlement of the clayey silt soils was modeled by estimating consolidation parameters based the results of laboratory index and water content testing as well as the in situ field test data (SPT) and is considered to be over-consolidated.

The simplified stratigraphy, unit weights and deformation parameters (see Chapter 6, "Commentary to the CHBDC, 2001") employed in the settlement analysis are summarized below:

<i>Soil</i>	<i>Thickness (m)</i>	<i>Bulk Unit Weight (kN/m³)</i>	<i>Deformation Properties</i>
Firm to Very Stiff Clayey Silt	2	18	$m_v = 0.17 \text{ m}^2/\text{MN}$
Compact to Dense Silt	3	19	$E' = 10 \text{ MPa}$
Very loose to Compact Till (upper 2 m)	2	21	$E' = 10 \text{ MPa}$
Compact to Very Dense Till and Sand to Sand and Gravel	4	21	$E' = 50 \text{ MPa}$

Based on the results of the settlement analyses, the maximum total settlement (i.e. initial and primary consolidation) of the foundation soils is estimated to be about 50 mm. This total settlement is estimated to be comprised of about 30 mm of immediate settlement due to compression of the cohesionless soil layers and about 20 mm of time dependant settlement of the cohesive soil layer. Settlement of the native granular soils is expected to occur rapidly (i.e. during or shortly after construction). A preliminary estimate of the consolidation settlement of

the cohesive soil layers is anticipated to be completed within about 1 to 2 months given the results of laboratory testing and a consolidation test performed on similar clayey deposits in the vicinity of the area (Swamp No. 3) as part of the overall project.

6.3.3.2 Settlement of Embankment Fill

Assuming the granular earth fill used for the new embankment and widening is placed and compacted in accordance with the recommendations provided in the Section 6.4.1, the settlement of the new embankment fill itself is expected to be less than 25 mm.

For the embankment widening on the existing Highway 7, the embankment fill material should be assessed to determine whether settlements will be a concern. Provided the existing embankment material that is to support the proposed widening area is granular in nature and is free of organic and/or deleterious materials, a preliminary estimate of the total and differential settlement of the new embankment widening is estimated to be less than 50 mm and 25 mm, respectively. The majority of this settlement (about 60%) is expected to occur immediately during construction.

Consideration could be given to subexcavating the clayey silt layer and replacing with engineered fill to reduce settlements; however, this option is not recommended due to the fact that water levels were measured in the piezometer to be as high as 0.4 m below ground surface (Elevation 245.4). Subexcavation to about Elevation 244 m would be required to remove the cohesive deposit resulting in subexcavating up to about 1.5 m below the water level. In addition, the integrity of the underlying silt soils would be difficult to maintain given the potential for basal instability (potentially artesian conditions in this area) and compaction of engineered fill would require substantial dewatering efforts.

6.4 Swamp Crossing No. 3

The location of the proposed new Highway 7 embankment crossing over Swamp Crossing No. 3 as part of the Highway 7 re-alignment is shown on Drawing 3. The preliminary design indicates up to about 3 m of embankment fill will be placed above the existing ground surface to match the existing highway grade.

Based on the borehole results, the subgrade soils in the immediate area of Swamp No. 3 generally consist of surficial topsoil and/or peat to 0.8 m to 2.3 m depth, underlain by a layer of silty sand, underlain by a deposit of clayey silt to silty clay containing interlayers of silt and sand. The clayey silt to silty clay deposit is underlain by layers of sandy silt to silt at depth. Placement of the new embankment directly on the existing peat in the swamp area will result in excessive settlements and side slope instability. In addition, long-term settlements related to decay of the organic materials would also occur.

Prior to the placement of any engineered fill for the new embankment construction and/or widening, all vegetation, topsoil or soils containing significant organics such as the peat and/or deleterious material should be stripped from below the proposed embankment footprint areas and wasted/reused for landscaping. Based on the preliminary boreholes, it is anticipated that excavations up to about 2.3 m below the existing ground surface will be required to remove the topsoil and peat soils. This excavation will be below the observed ground water level. After approval of the subgrade, the subexcavation should be backfilled in accordance with Section 6.5.1. Based on the plan drawing provided to us, the new embankment will merge completely into the existing Highway 7 embankment at about Station 10+600. As a result, there will be some widening of the existing embankment required as part of the construction and this will include stripping of all topsoil, softened/loosened soils or soils containing excessive organics from the side-slopes of the existing embankment. Where practical, all subgrade soils should be proof-rolled prior to backfill placement and embankment fill should be placed in accordance with SP206S03 and compacted in accordance with SP105S10. Benching into the existing embankment side slopes should be carried out as per OPSD 208.010 for construction of the embankment widening to ensure adequate keying of the new fill into the existing fill.

Based on the existing subsoil conditions, either earth fill or rock fill embankment options may be considered in this area. The different fill alternatives (i.e. earth fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils / bedrock), construction cost and time, and ease of construction / availability. Earth fill has typically been used in this area in the past; however, filling below the groundwater level should be by coarse granular fill such as Granular B Type II or rock fill.

The unsuitable topsoil, peat and organic soils should be completely removed below the anticipated embankment fill footprint in accordance with OPSD 203.010 for proposed new embankments and in accordance with OPSD 203.020 for proposed widening to existing embankments. It is anticipated that conventional excavators will be required for this work and that the sequence of sub-excavation and backfilling may have to be carried out in a series of small stages. Referring to Section 6.5.2, dewatering may be required for removal of the unsuitable soils in this swamp area depending on the conditions at the time of construction. Alternatively, removal of the unsuitable soils may require subaqueous excavation and placement of backfill. Placement of backfill under water will prohibit conventional spreading and compaction of materials. As mentioned previously, a coarse granular fill or rock fill should be used as backfill material for subaqueous backfill at this site.

Construction of the fill embankment above the original ground surface may be carried out using Select Subgrade Material (SSM) meeting OPSS 1010. For earth fill to rock fill transitions, the requirements outlined in OPSD 205.040 should be followed to prevent migration/loss of fines. All embankment fill should be placed in regular loose lifts not exceeding 300 mm, and compacted to at least 95 percent of the material Standard Proctor maximum dry density. The final lift prior

to placement of the granular subbase or base course should be compacted to 100 percent of the SPMDD. 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.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding should be placed as soon as possible in accordance with OPSS 572.

Further borehole drilling should be performed during the detail design phase of the project to delineate the extents of the peat and clayey deposits, when the details of the new embankment alignment are finalized. At that time, further investigation of the thickness of peat and composition of the embankment fills below the existing road surface and extent of clayey silt / silty clay soils will be required to confirm temporary stability concerns during construction, dewatering requirements and excavation / fill requirements.

A proposed culvert is to be constructed across Highway 7 near the centre of Swamp No. 3 at about Station 10+500. A separate preliminary foundation investigation and design report was prepared by Golder as part of the overall project titled "*Culvert Replacement at Station 10+502.5, Hwy 7 from Fowlers Corners Southerly to County Road 28, G.W.P. 73-99-00, Site No. 26-186*".

6.4.1 Stability

Given the fact the new embankment and / or widening to the existing embankment will match the existing embankment in both total height and side slope at the swamp crossing location, the global stability of the embankment is considered to be adequate.

A preliminary analysis was performed for the new embankment assuming the new embankment road grade is at the same level as the existing Highway 7 road grade (about Elevation 246 m) and side slopes are maintained at 2H:1V or shallower.

Static slope stability analyses for this embankment configuration have been 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. Both undrained and effective stress analyses were carried out.

<i>Soil Deposit</i>	<i>Unit Weight (kN/m³)</i>	<i>Effective Friction Angle (ϕ', degrees)</i>	<i>Undrained Shear Strength (kPa)</i>
Embankment fill (including backfill)	21	32	–
Loose to Compact Silty Sand	19	28	–
Soft to Very Stiff Clayey Silt to Silty Clay	18	–	50
Very Loose to Compact Sandy Silt	18	28	–
Underlying Interlayered Soil (based on DCPT tests)	20	32	–

For the purpose of the analysis, it was assumed that all topsoil and peat were removed, the subgrade properly prepared, and proper placement and compaction of the SSM engineered fill for the backfill and embankment construction was performed. As artesian groundwater conditions were encountered within the lower sandy silt layer encountered at a depth of about 8.7 m below ground surface (about Elevation 235 m to Elevation 236 m), a water level at the existing ground surface was modeled in the analysis.

For the proposed 3 m high embankment (assuming 2H:1V side slopes), a Factor of Safety of greater than 1.3 was calculated against static deep-seated slope instability.

While we consider that the seismic stability of the embankment should also be adequate given that the site is located in a low acceleration-related seismic zone, analyses should be performed at the detailed design stage.

6.4.2 Base Heave

Subexcavation depths to remove the topsoil and peat soils should be controlled to reduce the risk of basal heave or unstable founding soil conditions with the clayey silt soils containing interlayers of silt and sand. Given the artesian water conditions encountered within the underlying lower sandy silt to silt layer, preliminary calculations indicated that a soil thickness of about 6.5 m must be maintained for a Factor of Safety against basal heave of about 1.5. However, subexcavations to as deep as Elevation 241 m (i.e. the lowest elevation where peat / topsoil was encountered in borehole 06-6 which results in a clayey silt soil thickness of less than 5 m above the artesian water conditions) will be required for subexcavation resulting in a factor of safety less than 1.5 (but above unity) against base heave. The potential for base heave will have to be re-addressed after the detail design investigation is complete. Depending on the detail design, relief wells may be required to lower the artesian water pressure to below the subexcavation level or subaqueous subexcavation and backfill construction practices may be required.

6.4.3 Settlement

For the proposed new embankment as part of the future four-laning of Highway 7, some settlement is anticipated due to the loading imposed on the clayey subsoils from the new embankment loading. As previously discussed, the proposed embankment will be up to 3 m high (i.e. above the existing ground surface). Assuming the new embankment footprint will be stripped of topsoil and peat (which was up to about 2.3 m thick in borehole 06-6 and 06-7) and replaced with engineered fill (i.e. backfill), the total thickness of the new embankment fill is anticipated to be up to about 5 m. The anticipated total settlement will be a combination of the foundations soils and embankment fill itself as described in the next sections.

6.4.3.1 Settlement of the Foundation Soils

A preliminary settlement analysis was performed using the commercially available program UNISETTLE (Version 3.2) produced by Unisoft Limited. The soil parameters used for the analysis were based on the laboratory and in situ test data collected during the current preliminary investigation. The over-consolidation ratio (OCR) required in the settlement analysis was estimated using the results of the boreholes and consolidation test data. The total net loading on the foundation soils (after stripping, backfilling and embankment fill construction) was estimated to be about 70 kPa.

The immediate compression of the existing granular fill above the clayey silt to silty clay deposit and the compression of the lower sandy silt to silt below the cohesive deposit were modelled using elastic moduli. The time dependant consolidation settlement of the clayey silt to silty clay deposit was modelled using the following parameters.

<i>Soil</i>	<i>Initial Void Ratio</i> <i>e_o</i>	<i>Recompression Index</i> <i>C_r</i>	<i>Compression Index</i> <i>C_c</i>	<i>OCR</i>
Clayey Silt to Silty Clay	0.71	0.02	0.2	13

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

<i>Soil</i>	<i>Thickness (m)</i>	<i>Bulk Unit Weight</i> <i>(kN/m³)</i>	<i>Deformation Properties</i>
Backfill and Native Upper Silty Sand	1	20	E' = 10 MPa
Clayey Silt to Silty Clay	6	20	See table above
Lower Sandy Silt to Silty Sand Layer	2	19	E' = 5 MPa
Underlying Interlayered Soil (based on DCPT results)	5	20	E' = 20 MPa

The predicted maximum total settlement (initial and primary consolidation) from the new embankment loads is estimated to be about 75 mm due to the loading imposed by the new embankment fill. This total is estimated to be comprised of about 40 mm of immediate settlement due to compression of the cohesionless soils layers and about 35 mm of time dependant settlement of the cohesive soil layers. Based on an estimated coefficient of consolidation (*c_v*) of $4.5 \times 10^{-2} \text{ cm}^2/\text{s}$ and assuming two-way drainage of the approximately 6 m thick clayey silt to silty clay stratum, it is estimated that about 90% of the consolidation settlement will be complete within about 2 months. Settlement of the granular embankment fill

and shallow underlying native granular interlayers is expected to occur rapidly (i.e. during or shortly after construction). The majority of settlement within the deeper sandy silt to silt soils (below the cohesive deposit) are anticipated to occur gradually over about 2 months.

Based on the anticipated impact to the culvert and roadway due to the settlement of the future embankment, preloading is recommended. The new embankment should be constructed adjacent to the drainage course and allowed to settle at the proposed culvert location for at least 2 months prior to culvert installation. Other options (other than preloading) to mitigate the impacts of consolidation settlement could include surcharging, use of light weight fill, or partial or full subexcavation. Given the high water level, the deep extent of the clayey deposit and construction issues such as the close proximity of the existing roadway, full or partial subexcavation of the clayey deposit is not considered practical, use of light weight fill too expensive and surcharging not necessary.

6.4.3.2 Settlement of Backfill and Embankment Fill

For granular earth fill (i.e. SSM) placed above the water table and compacted in accordance with the recommendations provided in Section 6.4 and 6.5.1, the settlement of the new embankment fill itself (including backfill) is expected to be less than 25 mm.

If subexcavation and backfilling is carried out below the water level, the magnitude of the settlement for the subaqueous backfill is difficult to predict and is dependant on the material and construction techniques. If granular fill (i.e. SSM) is used as backfill below the water table, compaction in the wet will not be achievable and the magnitude of settlement of this portion of the fill may be excessive over time. As a result, to reduce the risk of settlements, it is recommended that backfill placed in subaqueous conditions consist of Granular B Type II material or rock fill and preloaded at least 2 months prior to construction of the pavement structure. If rock fill is used, the surface should be properly chinked and graded before placing the granular embankment fill. As a result, it is recommended that subexcavation and backfill activities for this area be completed as early as possible in the schedule for this project.

6.5 Other Considerations for Embankment Construction

6.5.1 Backfill

As previously mentioned, subexcavation of unsuitable soils of up to about 2.3 m is anticipated at the swamp locations. As a result, the subgrade will require placement of engineered fill (i.e. backfill) to raise grades to the existing surface elevations prior to embankment construction. This can be achieved by placing additional lifts of the properly placed and compacted embankment fill material (i.e. OPSS Select Subgrade Material or Granular 'B' Type II / rock fill in wet ground

conditions). If Granular 'B' Type II or rock fill is used, the placement of a geotextile filter between the bottom of the backfill and native soils may be required depending on the actual in situ ground conditions. The use of geotextiles should be confirmed at the detailed design stage.

Following proof-rolling and/or approval of the subgrade, the backfill should be placed in maximum 300 mm thick loose lifts and be uniformly compacted to 95 percent of the SPMDD. Placement of backfill for the new embankment should be carried out in accordance with Special Provision SP206S03 and compacted in accordance with SP105S10.

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.

6.5.2 Groundwater and Surface Water Control

Based on the borehole information, excessive groundwater is not anticipated to be encountered during construction of the high fill embankment or at the Swamp Crossing No. 2 location. Any seepage or surface water should be able to be handled by pumping from filtered sumps and/or filtered drains.

The founding soils at Swamp Crossing No.3 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 Jackson Creek tributary can be diverted around the construction area. Groundwater seepage into the excavation is expected, particularly at the south portion of the site where the water level is high and the peat deposit is the thickest. The severity of the groundwater conditions is dependent upon many factors including the season during which construction occurs and the flow rate of the Jackson 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 in the dry.

6.5.3 Excavations

Based on the preliminary boreholes, it is anticipated that excavations up to about 1 m and 2.3 m below the existing ground surface at the Swamp No. 2 and 3 locations, respectively, will be required to remove the topsoil, peat, and soils containing excessive organics to expose the native clayey silt and silty sand soils. Excavation up to about 3 m will be required in cut areas at the

west limit of the high fill / cut area. The depth of subexcavation at the high fill area will need to be addressed during detailed design.

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

Excavation works 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.

For typical subexcavations through dewatered topsoil and peat soils, these organic soils may be classified as Type 3 and 4 soils under the OHSA, as such, it is recommended that temporary open-cut slopes be maintained no steeper than 2H:1V. At the Swamp No. 3 location, some local flattening of the side-slopes may be required due to the surface water conditions and extent of dewatering effort at the time of construction. At the high fill embankment area, the existing compact to dense sand and gravel embankment soils and underlying native silty sand and gravel containing trace organics would be classified as Type 2 soil under the guidelines, as such, it is recommended that temporary open-cut slopes be maintained no steeper than 1H:1V.

For typical subaqueous subexcavations, and in accordance with OPSD 203.010, the side-slopes of the excavated material (i.e. peat) should be equal to the angle of repose of the excavated material. It is anticipated that submerged side-slopes of about 2H:1V will be required to maintain stability and prevent sloughing of the unsuitable soils onto the embankment foundation footprint.

Depending on construction staging, the final sub-excavation depths and embankment footprint (after detailed design), and property constraints, some areas may require temporary shoring or temporary detour embankments to maintain traffic on Highway 7 during construction. Temporary detour embankments should be constructed in accordance with the recommendations provided above for any subexcavation, backfill and engineered fill placement. In general, temporary support systems, if being considered, should be designed to Performance Level 3 as defined SP 105S19. If the temporary excavation support system is required for existing roadway or utility protection, then the temporary shoring system should meet Performance Level 2 as specified in SP 105S19.

6.6 Closure

This report was prepared by Ms. Shannon Palmer, EIT, and the technical aspects were reviewed by Mr. Kevin J. Bentley, P.Eng., and geotechnical engineer. Fintan J. Heffernan, a Designated MTO Contact with Golder, conducted a quality control review of the report.

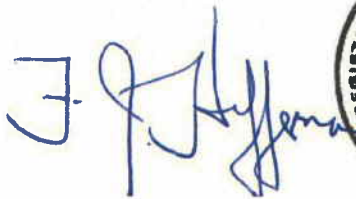
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SLP/KJB/FJH/kjb/al

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REFERENCES

Canadian Geotechnical Society. 1992. "Canadian Foundation Engineering Manual (CFEM) - Third Edition", Technical Committee on Foundations.

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

Ministry of Transportation Ontario. 2004. "Special Provision No. 105S10 – Construction Specification for Compaction", November. Amendment to OPSS 501, Feb. 1996.

Ministry of Transportation Ontario. 2005. "Special Provision No. 105S19 – Construction Specification for Protection Systems", March. Amendment to OPSS 539, Nov. 2003.

Ministry of Transportation Ontario. 2004. "Special Provision No. 206S03 – Earth Excavation, Grading-Item No., Excavation for Pavement Widening-Item No., Rock Excavation, Grading-Item No., Rock Face-Item No., Rock Embankment-Item No.", January.

Ontario Provincial Standards. 2005. "Ontario Provincial Standard Drawing OPSD 202.010 – Slope Flattening Using Excess Material on Earth or Rock Embankment", Revision 1, November.

Ontario Provincial Standards. 2003. "Ontario Provincial Standard Drawing OPSD 208.010 – Benching of Earth Slopes", Revision 1, November.

Ontario Provincial Standards. 2004. "Ontario Provincial Standard Specification OPSS 422 – Construction Specification For Precast Reinforced Concrete Box Culverts And Box Sewers in Open Cut", April.

Ontario Provincial Standards. 2003. "Ontario Provincial Standard Specification OPSS 572 – Construction Specification for Seed and Cover", November.

Ontario Provincial Standards. 2002. "Ontario Provincial Standard Specification OPSS 902 – Construction Specification For Excavation and Backfilling – Structures", November.

Ontario Provincial Standards. 2004. "Ontario Provincial Standard Specification OPSS 1002 – Material Specification for Aggregates - Concrete", April.

Ontario Provincial Standards. 2004. "Ontario Provincial Standard Specification OPSS 1010 – Material Specification For Aggregates – Base, Subbase, Select Subgrade, and Backfill Material", April.

Ontario Regulation 213/91. "Occupational Health and Safety Act - Construction Projects", Amended to O. Reg. 527/00.

TABLE 1
SUMMARY OF RECOMMENDATIONS AT HIGH FILL AND SWAMP CROSSING LOCATIONS
HIGHWAY 7, PETERBOROUGH
G.W.P. 73-99-00

<i>Location</i>	<i>Approx. Station</i>	<i>Boreholes</i>	<i>Proposed Works</i>	<i>Surface Conditions</i>	<i>Recommended Embankment Fill Type</i>	<i>Organics Encountered along alignment</i>	<i>Recommended Embankment Side Slope</i>	<i>*Estimated Post-Construction Settlements (δ) and Mitigation Measures</i>
High Fill Area (Hwy 7 and Lily Lake Road)	9+750 to 10+000	06-11, 06-12	High Fill (up to 12 m high)	Asphalt Road Surface and existing embankment up to 6 m high	OPSS Select Subgrade Material	No, not on existing road; however, visually present on side-slopes and near toes of existing embankment	2H : 1V (bench required)	$\delta_{\max} < 25$ mm No mitigation measures required 2 m wide bench required where embankment height exceeds 8 m.
Swamp Crossing No. 2 (Parkhill Road and Hwy. 7)	28+100 to 28+150 and 10+000 to 10+050	06-9, 06-10	Fill up to 3 m high	Swamp	OPSS Select Subgrade Material	Yes. About 0.3 m below ground surface.	2H: 1V	$\delta_{\max} < 25$ mm No mitigation measures required
Swamp Crossing No. 3 (Highway 7)	10+400 to 10+600	06-5,06-6, 06-7,06-8	Fill up to 3 m high	Swamp	OPSS Select Subgrade Material over Granular B Type II	Yes. Up to 2.3 m below ground surface.	2H:1V	$\delta_{\max} < 35$ mm Preload for 2 months.

***Refers to settlement expected from cohesive foundation soils only. Short-term settlements (including shortly after construction) are not included.**

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Consistency

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

(b) Cohesive Soils

c_u, s_u

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

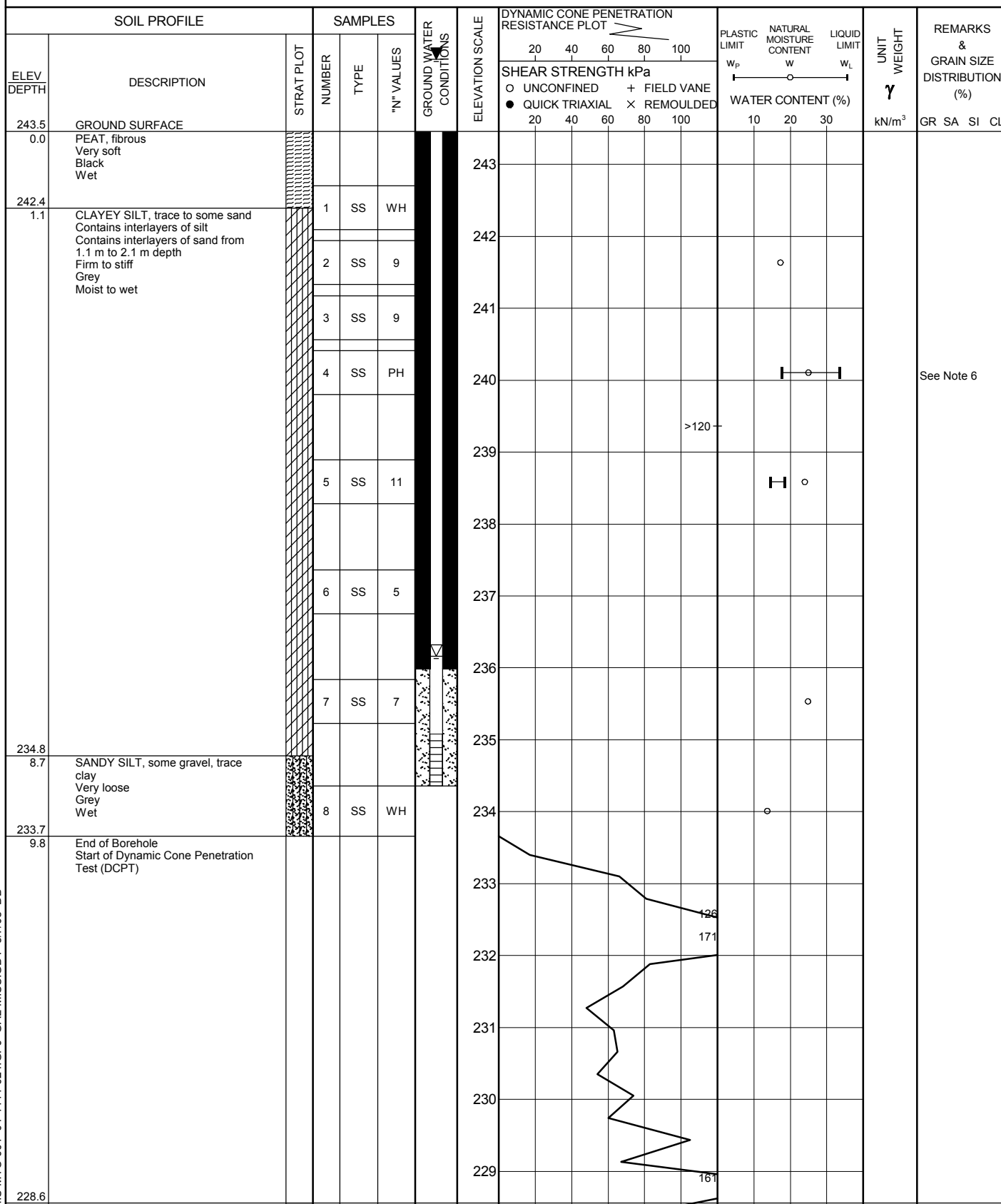
C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation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

(d) Shear Strength

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

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

PROJECT <u>04-1111-024D</u>		RECORD OF BOREHOLE No 06-5		1 OF 2 METRIC	
W.P. <u>73-99-00</u>		LOCATION <u>N 4905696.0 ; E 390739.5</u>		ORIGINATED BY <u>SB</u>	
DIST <u> </u> HWY <u>7</u>		BOREHOLE TYPE <u>Power Auger, 101 mm O.D. Solid Stem Augers</u>		COMPILED BY <u>DD</u>	
DATUM <u>Geodetic</u>		DATE <u>July 5, 2006</u>		CHECKED BY <u>SLP</u>	



Continued Next Page

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

PROJECT 04-1111-024D		RECORD OF BOREHOLE No 06-5				2 OF 2 METRIC																
W.P. 73-99-00		LOCATION N 4905696.0 ; E 390739.5				ORIGINATED BY SB																
DIST _____ HWY 7		BOREHOLE TYPE Power Auger, 101 mm O.D. Solid Stem Augers				COMPILED BY DD																
DATUM Geodetic		DATE July 5, 2006				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 ---							<div style="display: flex; justify-content: space-between; font-size: small;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between; font-size: x-small;"> ○ UNCONFINED ○ FIELD VANE </div> <div style="display: flex; justify-content: space-between; font-size: x-small;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between; font-size: small;"> 20 40 60 80 100 10 20 30 </div>										
14.9	End of DCPT Notes: 1. Water level measured in piezometer at 7.3 m depth (Elevation 236.2 m) upon completion of installation on July 5, 2006. 2. Water level measured in piezometer at 3.8 m depth (Elevation 239.7 m) on July 6, 2006. 3. Water level measured in piezometer at 0.9 m above ground surface (Elevation 244.4 m) on July 10, 2006. 4. Water level measured in piezometer at 0.8 m above ground surface (Elevation 244.3 m) on July 31, 2006. 5. Water level measured in piezometer at 1.0 m above ground surface (Elevation 244.5 m) on August 18, 2006. 6. Laboratory consolidation (oedometer) test performed on Sample No. 4.						Effective Refusal - 100 blows/0.3m															

PROJECT 04-1111-024D			RECORD OF BOREHOLE No 06-6			1 OF 2 METRIC		
W.P. 73-99-00			LOCATION N 4905673.2; E 390748.7			ORIGINATED BY SB		
DIST HWY 7			BOREHOLE TYPE Power Auger, 101 mm O.D. Solid Stem Augers			COMPILED BY DD		
DATUM Geodetic			DATE July 6, 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 (%) 10 20 30
243.3	GROUND SURFACE						243	
0.0	PEAT, fibrous Very soft Black Moist to wet		1	SS	WH		242	
			2	SS	WH		241	
241.0							240	
2.3	CLAYEY SILT to SILTY CLAY, trace sand and gravel Contains interlayers of silt and sand in upper zone Very soft to stiff Grey Moist		3	SS	2		239	
			4	SS	12		238	
			5	SS	9		237	
237.2							236	
6.1	CLAYEY SILT, trace to some sand and gravel Very soft to firm Grey Moist		6	SS	2		235	
			7	SS	5		234	
234.8							233	
8.5	SANDY SILT, some clay Loose Grey Wet		8	SS	6		232	
233.5							231	
9.8	End of Borehole Start of Dynamic Cone Penetration Test (DCPT)						230	
							229	
228.3								

Continued Next Page

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

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD



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

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD

PROJECT <u>04-1111-024D</u>		RECORD OF BOREHOLE No 06-7		2 OF 2 METRIC	
W.P. <u>73-99-00</u>		LOCATION <u>N 4905705.4 ;E 390726.1</u>		ORIGINATED BY <u>SB</u>	
DIST <u> </u> HWY <u>7</u>		BOREHOLE TYPE <u>Power Auger, 101 mm O.D. Solid Stem Augers</u>		COMPILED BY <u>DD</u>	
DATUM <u>Geodetic</u>		DATE <u>July 6, 2006</u>		CHECKED BY <u>SLP</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 (%)							
						20 40 60 80 100			10 20 30							
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										
15.0	— CONTINUED FROM PREVIOUS PAGE —															
223.1																
21.6	End of DCPT Note: 1. Water level measured in open borehole at 2.3 m depth (Elevation 242.4 m) upon completion of drilling. 2. SPT N-Value for Sample # 1 affected by pressence of wood fragments.															

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD

Continued Next Page

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

PROJECT <u>04-1111-024D</u>		RECORD OF BOREHOLE No 06-8				2 OF 2 METRIC											
W.P. <u>73-99-00</u>		LOCATION <u>N 4905746.5 ; E 390721.6</u>				ORIGINATED BY <u>SB</u>											
DIST <u> </u> HWY <u>7</u>		BOREHOLE TYPE <u>Power Auger, 101 mm O.D. Solid Stem Augers</u>				COMPILED BY <u>DD</u>											
DATUM <u>Geodetic</u>		DATE <u>July 6, 2006</u>				CHECKED BY <u>SLP</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---																
	End of DCPT Note: 1. Water level measured in open borehole at 0.6 m depth (Elevation 242.5 m) upon completion of drilling.																

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD

PROJECT 04-1111-024E			RECORD OF BOREHOLE No 06-9			1 OF 1 METRIC																								
W.P. 73-99-00			LOCATION N 4906151.5 ; E 390580.5			ORIGINATED BY SB																								
DIST HWY 7			BOREHOLE TYPE Power Auger, 101 mm O.D. Solid Stem Augers			COMPILED BY DD																								
DATUM Geodetic			DATE July 7, 2006			CHECKED BY SLP																								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																									
246.5	GROUND SURFACE																													
246.2	TOPSOIL																													
0.3	CLAYEY SILT, some sand, trace gravel, contains silt and sandy silt interlayers Firm to very stiff Brown to brown and grey Moist to wet		1	SS	5																									
			2	SS	18																									
244.2	SILT, trace sand, trace to some clay Contains clayey silt and sandy silt interlayers Compact to dense Brown to grey Moist to wet Becoming grey at 3.1 m depth		3	SS	32																									
2.3			4	SS	23																									
241.9	SILTY SAND, some gravel, trace to some clay, contains sand and gravel interlayers, cobbles and boulders (TILL) Very loose to very dense Grey Moist		5	SS	3																									
4.6			6	SS	WH																									
			7	SS	14/15																									
			8	SS	100/15																									
			9	SS	117/15																									
235.7	End of Borehole																													
10.8	Notes: 1. Water level measured in piezometer at 8.7 m depth (Elevation 237.8 m) upon completion of drilling. 2. Water level measured in piezometer at 1.9 m depth (Elevation 244.6 m) on July 10, 2006. 3. Water level measured in piezometer at 0.4 m depth (Elevation 246.1 m) on July 31, 2006. 3. Water level measured in piezometer at 1.1 m depth (Elevation 245.4 m) on August 18, 2006.																													

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD

PROJECT 04-1111-024E			RECORD OF BOREHOLE No 06-10			1 OF 1 METRIC		
W.P. 73-99-00			LOCATION N 4906201.7 ; E 390567.1			ORIGINATED BY SB		
DIST HWY 7			BOREHOLE TYPE Power Auger, 101 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
246.2	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
0.0	TOPSOIL						246	
0.3	CLAYEY SILT, trace sand and gravel Stiff Brown and grey Moist to wet		1	SS	12		245	
244.2			2	SS	12		244	
2.0	SILT, trace to some clay, trace sand, contains clayey silt interlayers Compact Grey Wet		3	SS	15		243	
			4	SS	15		242	
241.0			5	SS	16		241	
5.2	SANDY SILT to SILTY SAND, trace to some clay, some gravel, contains sand and gravel interlayers, cobbles and boulders (TILL) Compact to very dense Grey Moist to wet		6	SS	20		240	
			7	SS	76/15		239	
							238	
237.4			8	SS	60/10		237	
8.8	SAND, trace silt Very dense Moist						236	
235.8								
10.4	SAND and GRAVEL Very dense Grey Moist		9	SS	50/08			
235.4								
10.8	End of Borehole							
Note: 1. Water level measured in open borehole at 6.1 m depth (Elevation 240.1 m) upon completion of drilling.								
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL								

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD

PROJECT 04-1111-024E			RECORD OF BOREHOLE No 06-11			1 OF 2 METRIC											
W.P. 73-99-00			LOCATION N 4907547.0 ; E 390222.4			ORIGINATED BY SB											
DIST _____ HWY 7			BOREHOLE TYPE Power Auger, 101 mm O.D. Solid Stem Augers			COMPILED BY DD											
DATUM Geodetic			DATE July 10, 2006			CHECKED BY SLP											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30					
265.7	GROUND SURFACE																
0.1	ASPHALT																
	Sand and gravel, trace to some silt, trace clay (FILL)																
	Compact to very dense																
	Brown																
	Moist																
	Obstruction (inferred boulder) at 2.7m depth																
			1	SS	63		265										
			2	SS	40		264										
			3	SS	77/25		263										
			4	SS	32		262										
			5	SS	16		261										
259.9							260										
5.8	SAND and GRAVEL, trace silt, trace organics																
	Compact																
	Grey to black																
	Wet																
			6	SS	13		259										
258.4																	
7.3	SILTY SAND, some gravel, trace to some clay, contains cobbles and boulders (TILL)																
	Compact to very dense																
	Grey																
	Moist to wet																
			7	SS	13		258										
			8	SS	64/15		257										
			9	SS	100/10		256										
			10	SS	92/15		255										
253.4							254										
12.3																	

Continued Next Page

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

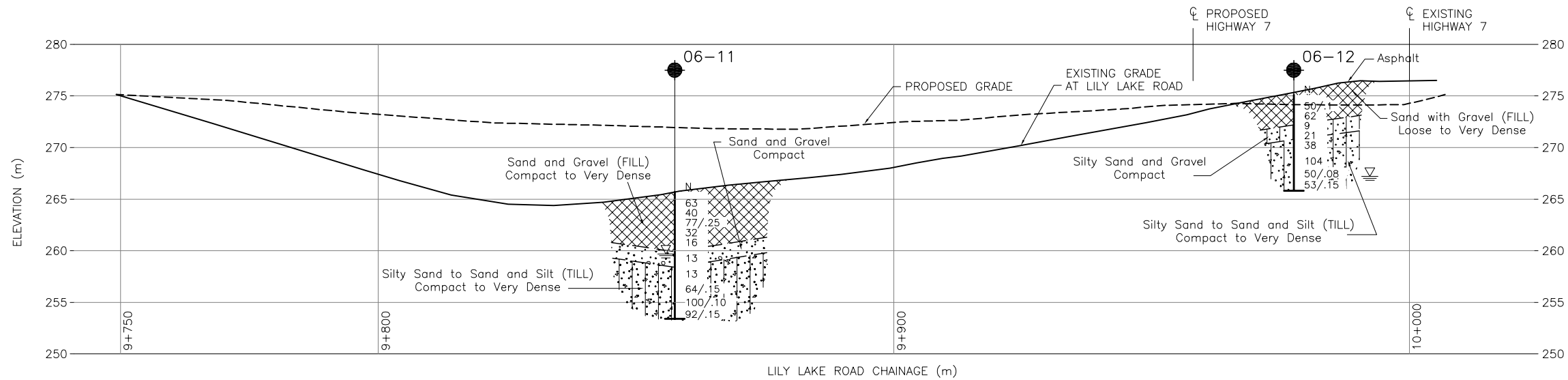
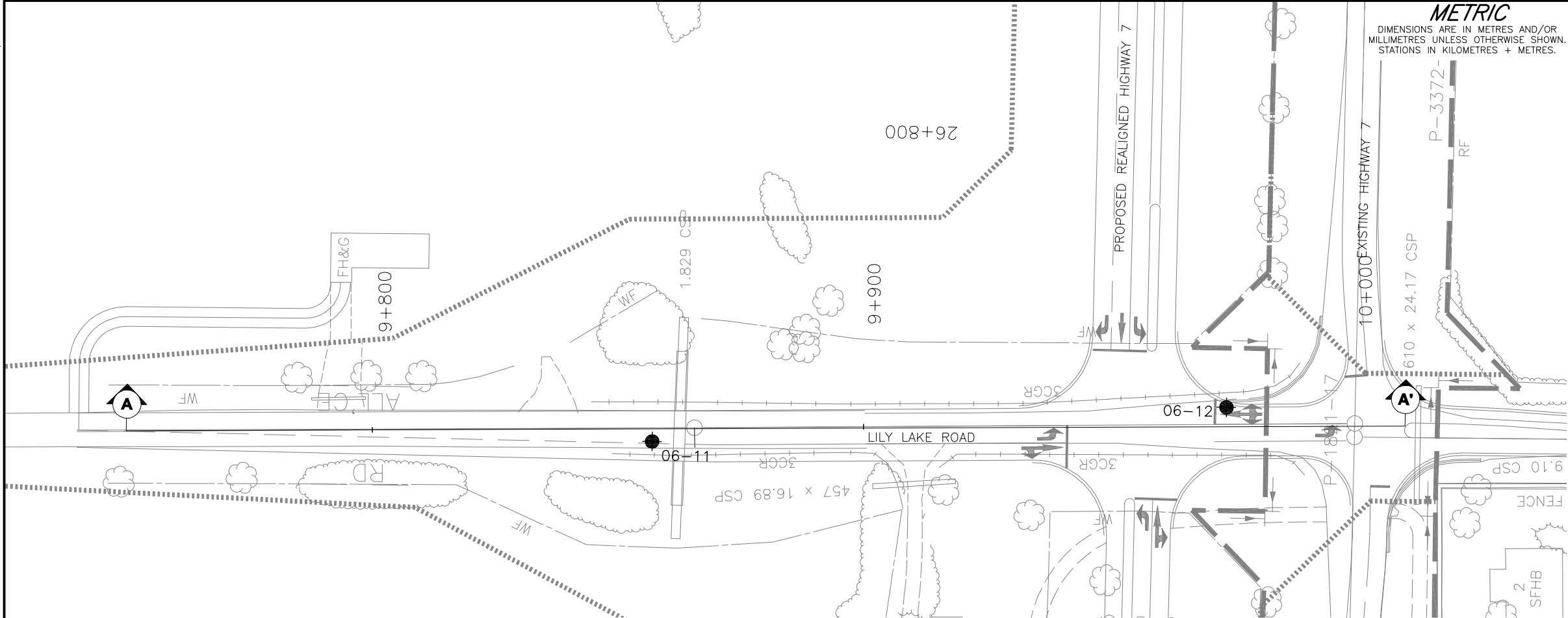
MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD



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

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 3/7/08 DD

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



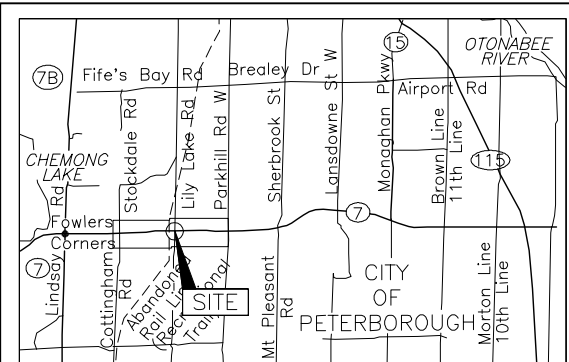
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 73-99-00

HIGHWAY 7
HIGH FILL EMBANKMENT
BOREHOLE LOCATIONS AND
SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE 2 0 2 km

LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
06-11	265.7	4907547.0	390222.4
06-12	275.1	4907501.9	390114.3

NOTES

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

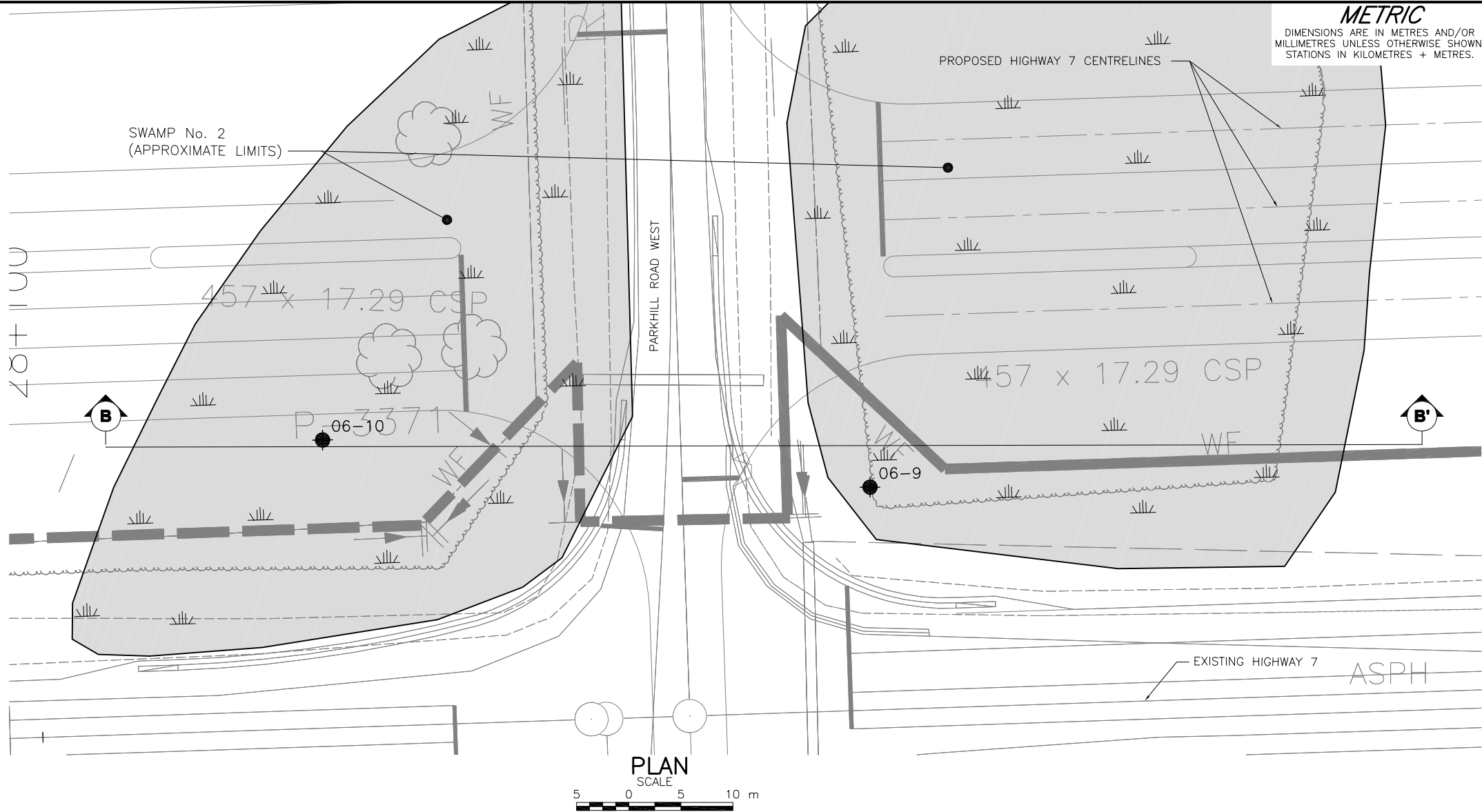
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

REFERENCE

Base plans provided in digital format by NCE, drawing file no. 2004-002 recommended plan.dwg, received March 31, 2006.

NO.	DATE	BY	REVISION
Geocres No.31D-437			
HWY. 7		PROJECT NO. 04-1111-024E	DIST.
SUBM'D. SLP	CHKD. KJB	DATE: Mar. 7, 2008	SITE:
DRAWN: MSM	CHKD. KJB	APPD.	DWG. 1

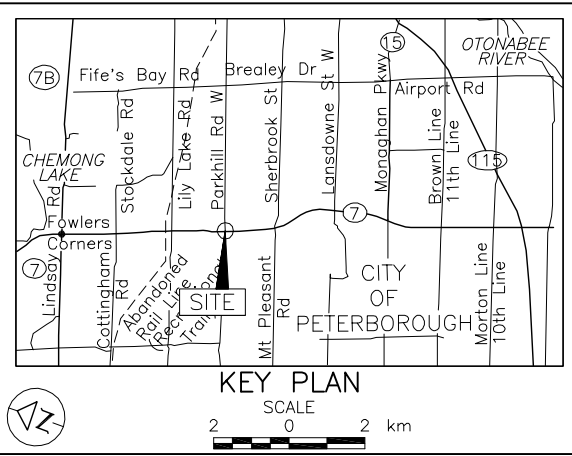


CONT No.
WP No. 73-99-00

HIGHWAY 7
SWAMP CROSSING NO. 2
BOREHOLE LOCATIONS AND
SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on August 18, 2006
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
06-9	246.5	4906151.5	390580.5
06-10	246.2	4906201.7	390567.1

NOTES

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

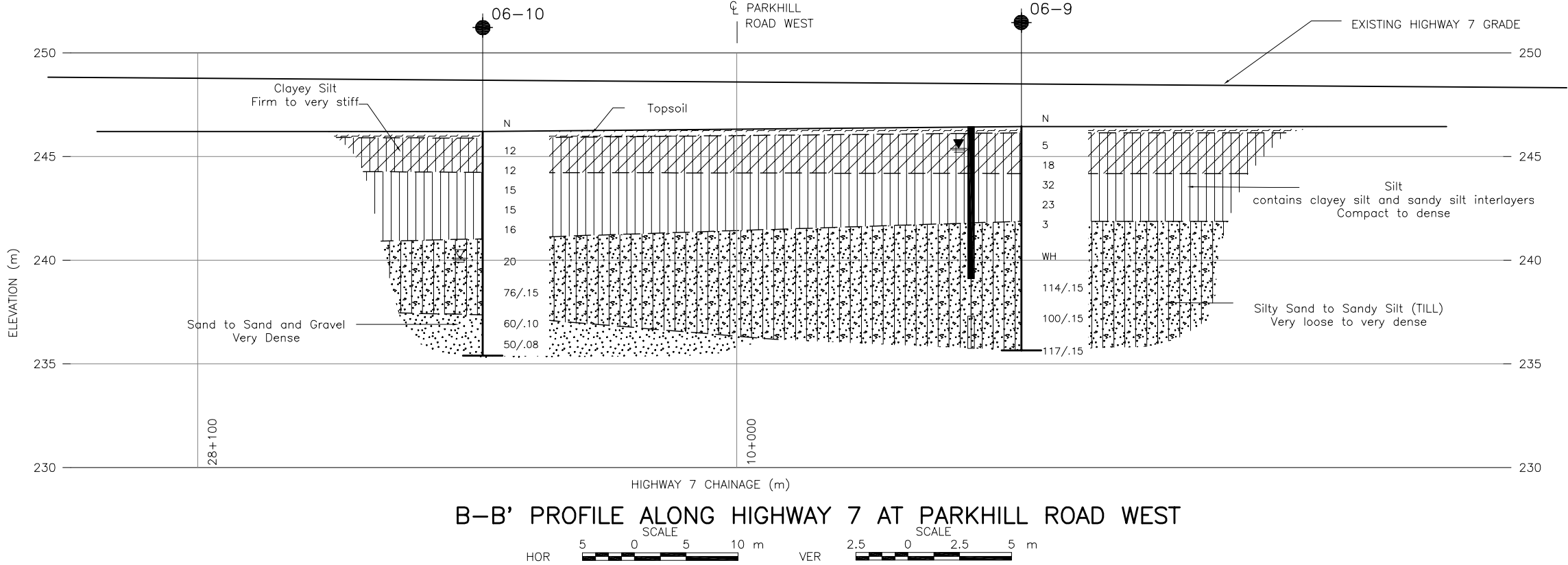
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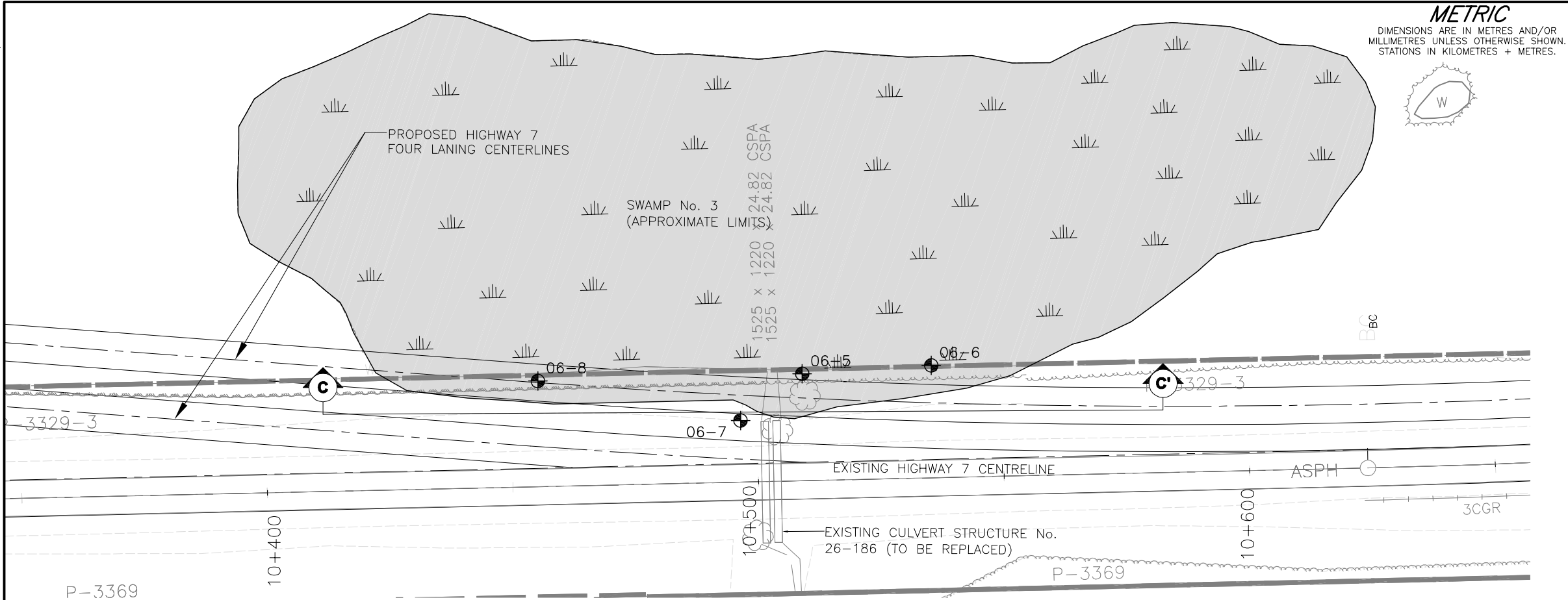
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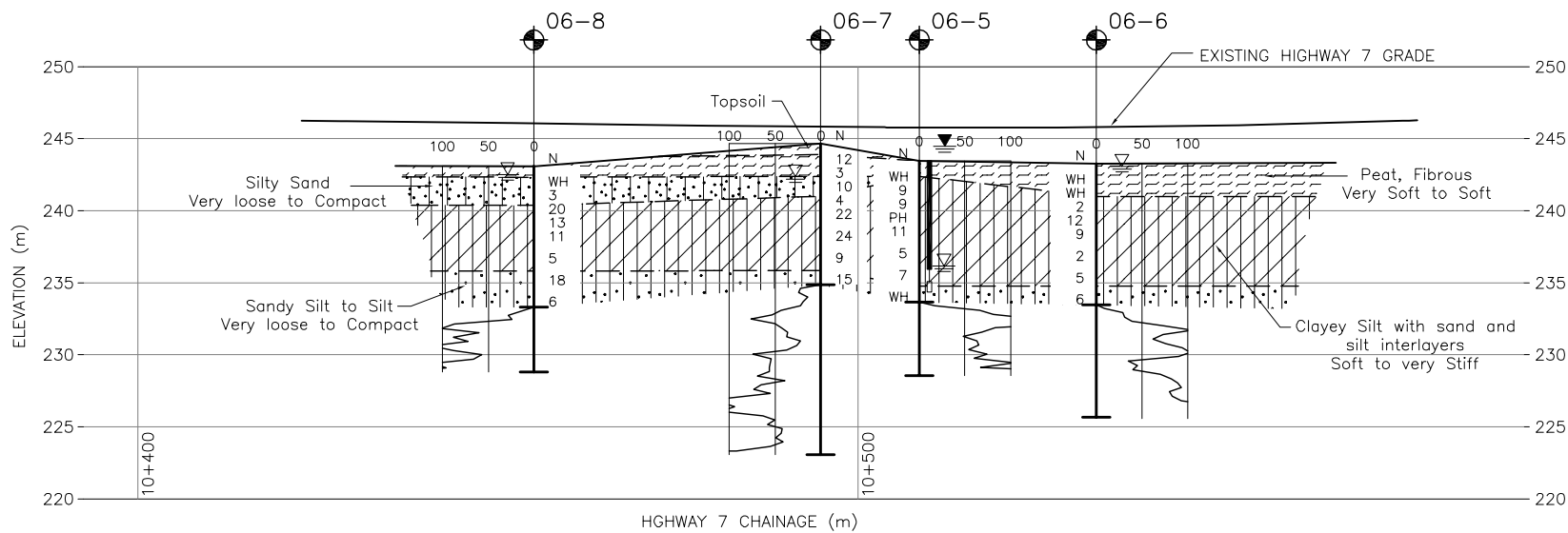
NO.	DATE	BY	REVISION
Geocres No.31D-437			
HWY. 7	PROJECT NO. 04-1111-024E		DIST.
SUBM'D. SLP	CHKD. KJB	DATE: Mar. 7, 2008	SITE:
DRAWN: MSM	CHKD. KJB	APPD.	DWG. 2



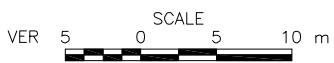
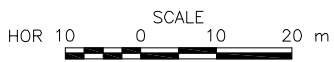
B-B' PROFILE ALONG HIGHWAY 7 AT PARKHILL ROAD WEST



PLAN



C-C' PROFILE ALONG HWY 7 AT SWAMP CROSSING



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



CONT No.
WP No. 73-99-00

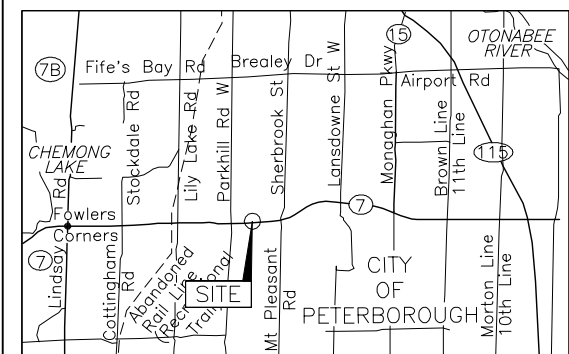
HIGHWAY 7
SWAMP CROSSING NO. 3
BOREHOLE LOCATIONS AND
SOIL STRATA



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
0 2 km

LEGEND

- Borehole & Dynamic Cone Penetration
- Seal
- Piezometer
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on August 18, 2006
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
06-5	243.5	4905696.0	390739.5
06-6	243.3	4905673.2	390748.7
06-7	244.7	4905705.4	390726.1
06-8	243.1	4905746.5	390721.6

NOTES

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

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

REFERENCE

Base plans provided in digital format by NCE, drawing file no. 2004-002 recommended plan.dwg, received March 31, 2006.

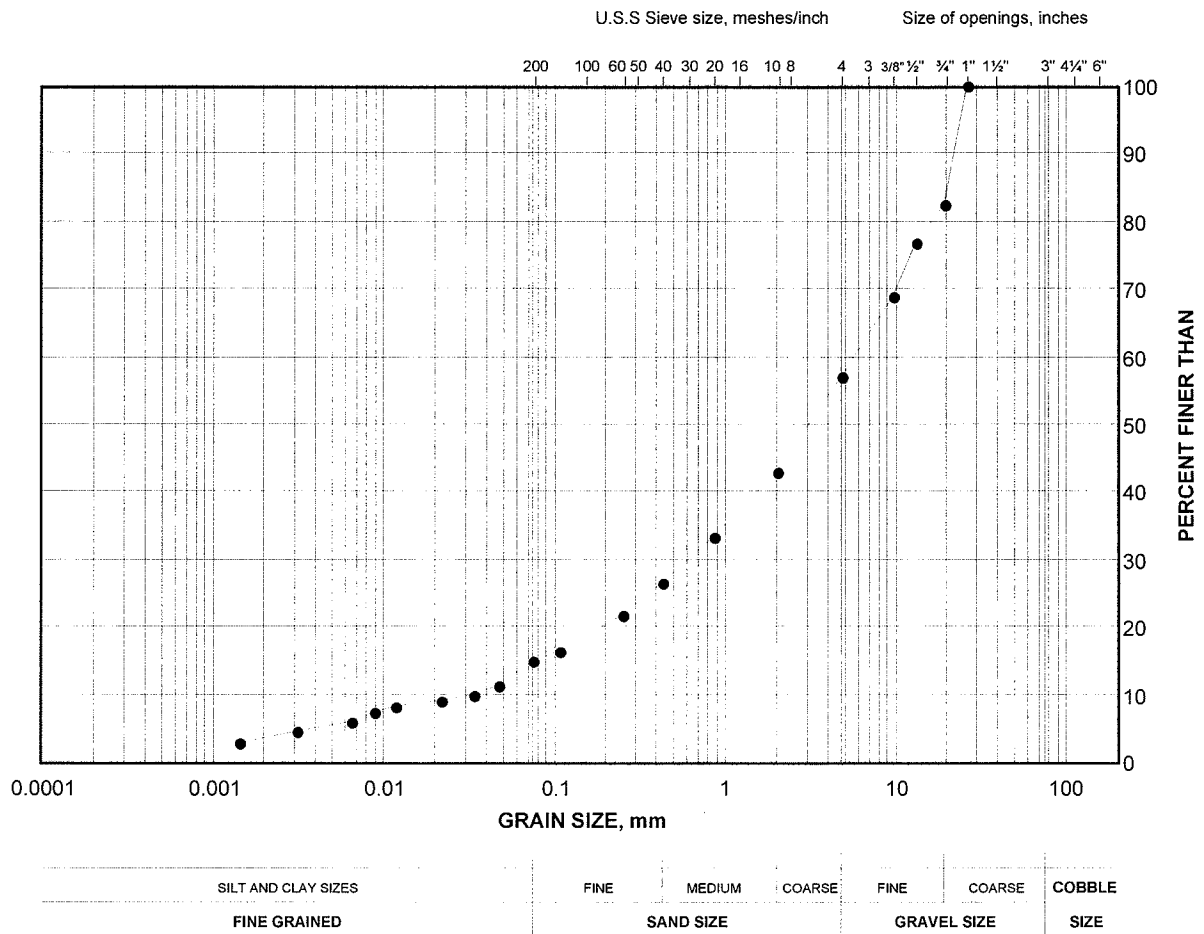
NO.	DATE	BY	REVISION
Geocres No. 31D-437			
HWY. 7		PROJECT NO. 04-1111-024E	
SUBM'D. SLP		CHKD. KJB	DATE: Mar. 7, 2008
DRAWN: MSM		CHKD. KJB	APPD. DWG. 3

APPENDIX A
LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Fill)

FIGURE A1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-11	4	262.3

Project Number: 04-1111-024E

Checked By: KJP

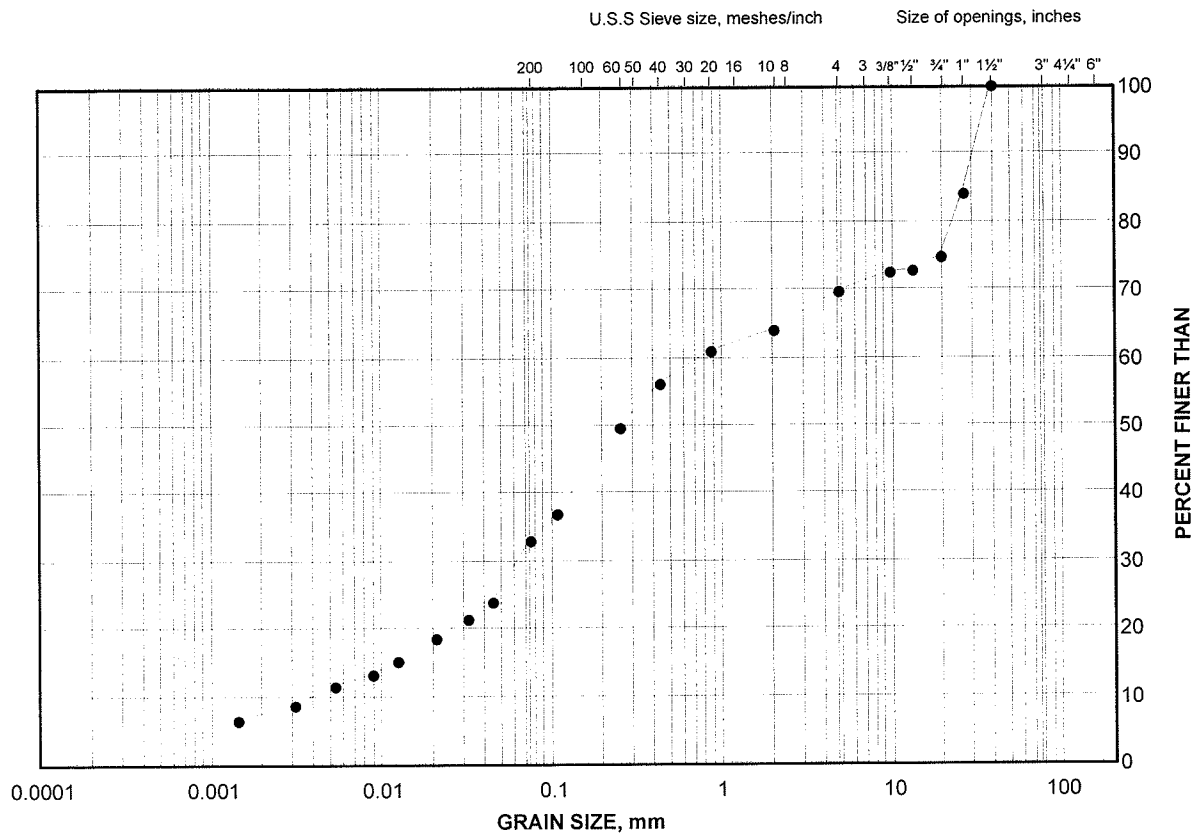
Golder Associates

Date: 01-Jun-07

GRAIN SIZE DISTRIBUTION

Silty Sand and Gravel

FIGURE A2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-12	4	268.7

Project Number: 04-1111-024E

Checked By: KB

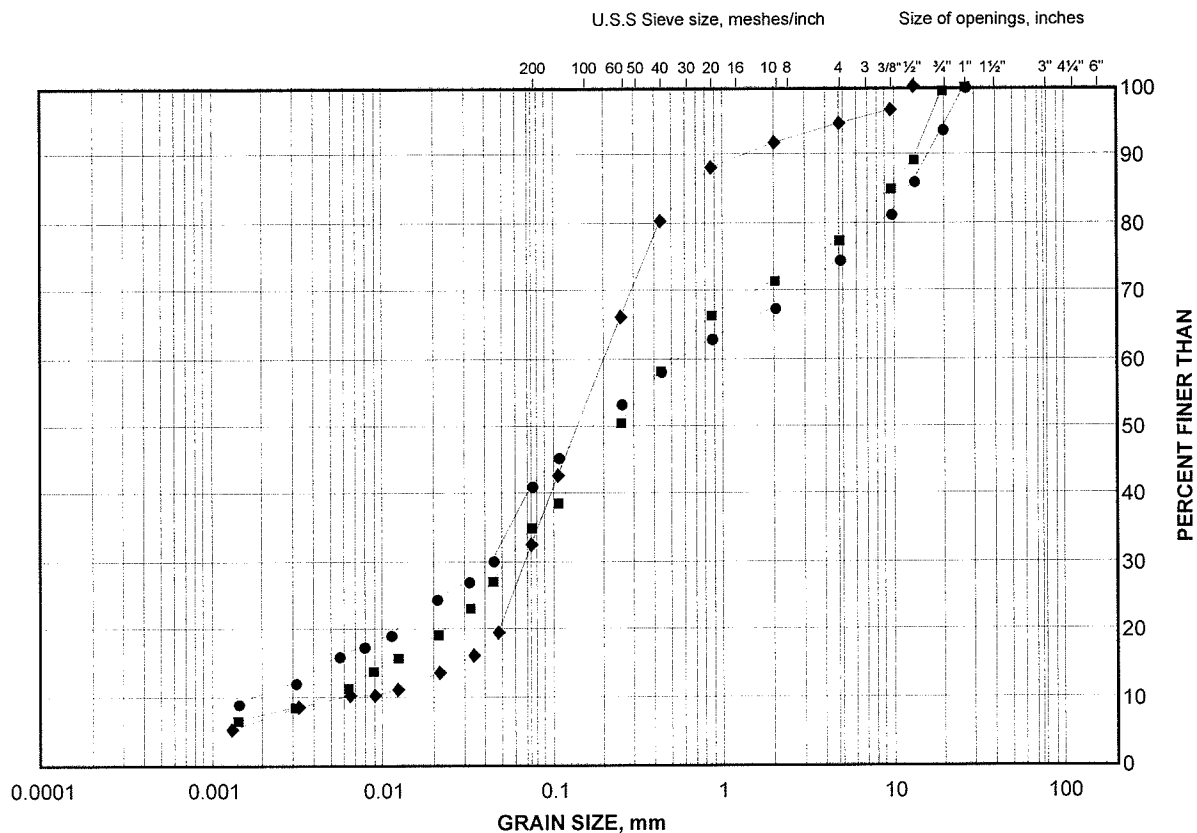
Golder Associates

Date: 01-Jun-07

GRAIN SIZE DISTRIBUTION

Silty Sand (Till)

FIGURE A3



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

LEGEND

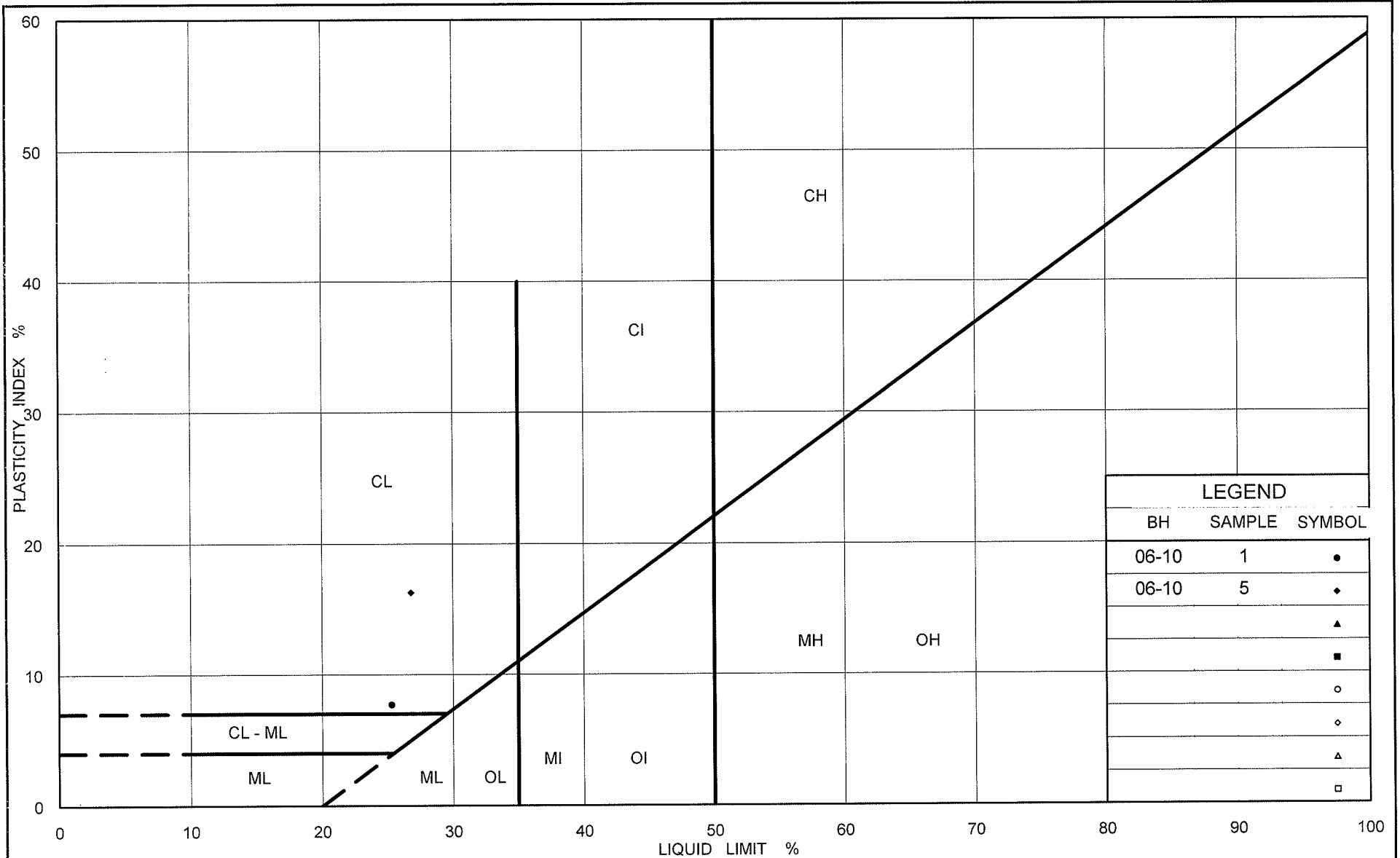
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-12	6	265.9
■	06-11	7	257.8
◆	06-12	8	263.1

Project Number: 04-1111-024E

Checked By: KJB

Golder Associates

Date: 01-Jun-07



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt

FIG No. A4

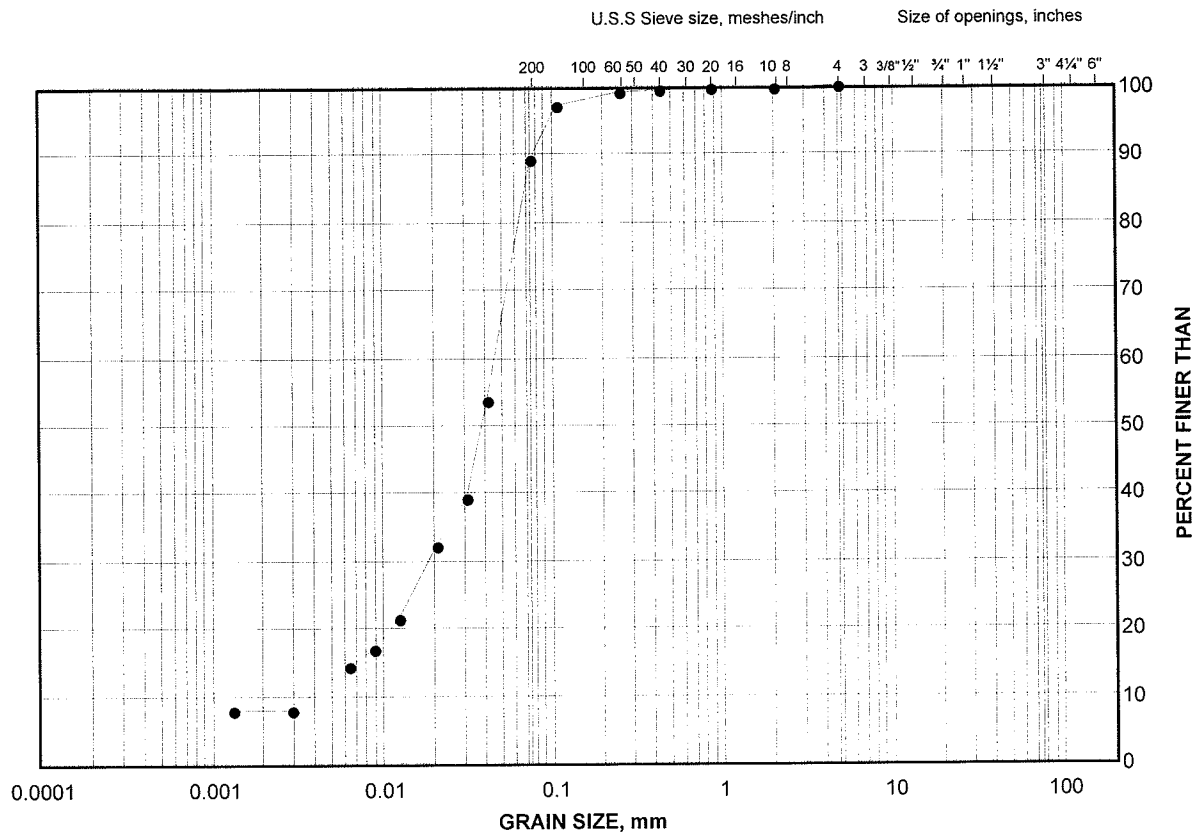
Project No. 04-1111-024E

Checked By: KSB

GRAIN SIZE DISTRIBUTION

Clayey Silt with silt Interlayers

FIGURE A5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-9	2	244.7

Project Number: 04-1111-024E

Checked By: KAB

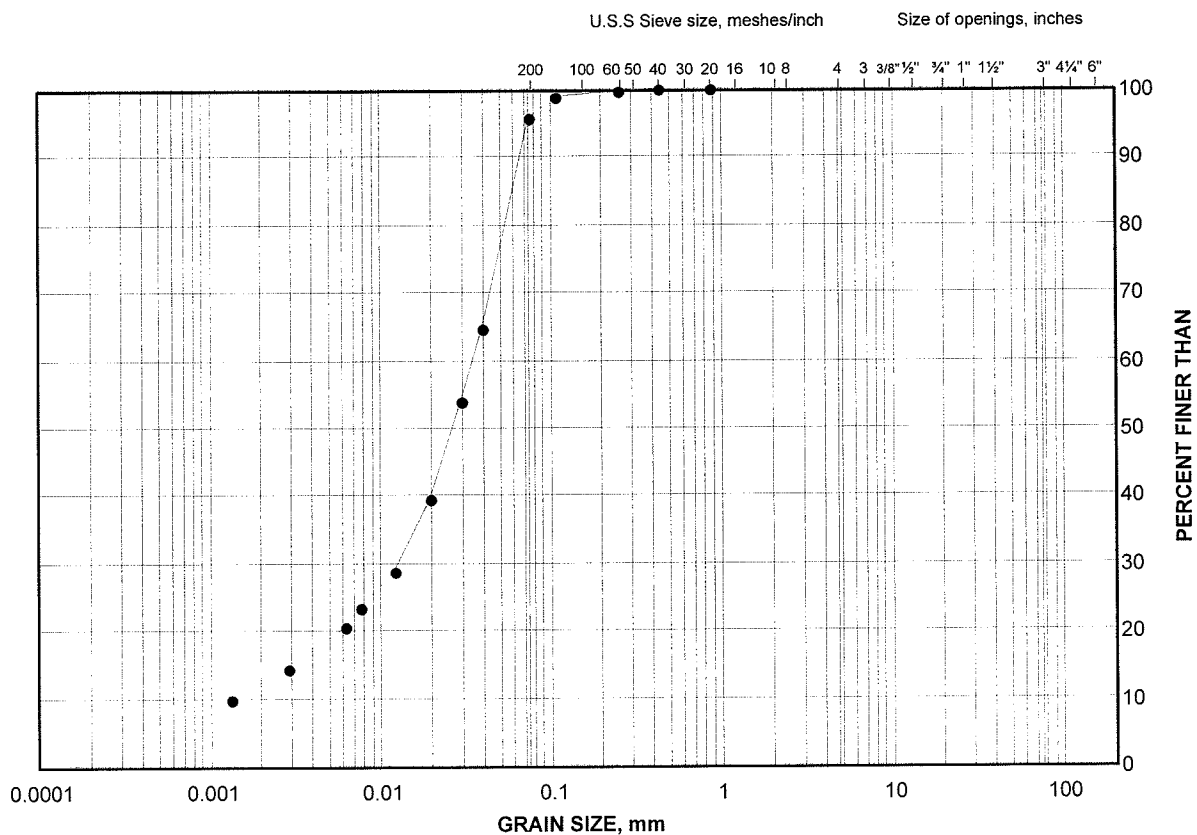
Golder Associates

Date: 07-Jun-07

GRAIN SIZE DISTRIBUTION

Silt

FIGURE A6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-10	3	243.6

Project Number: 04-1111-024E

Checked By: KSB

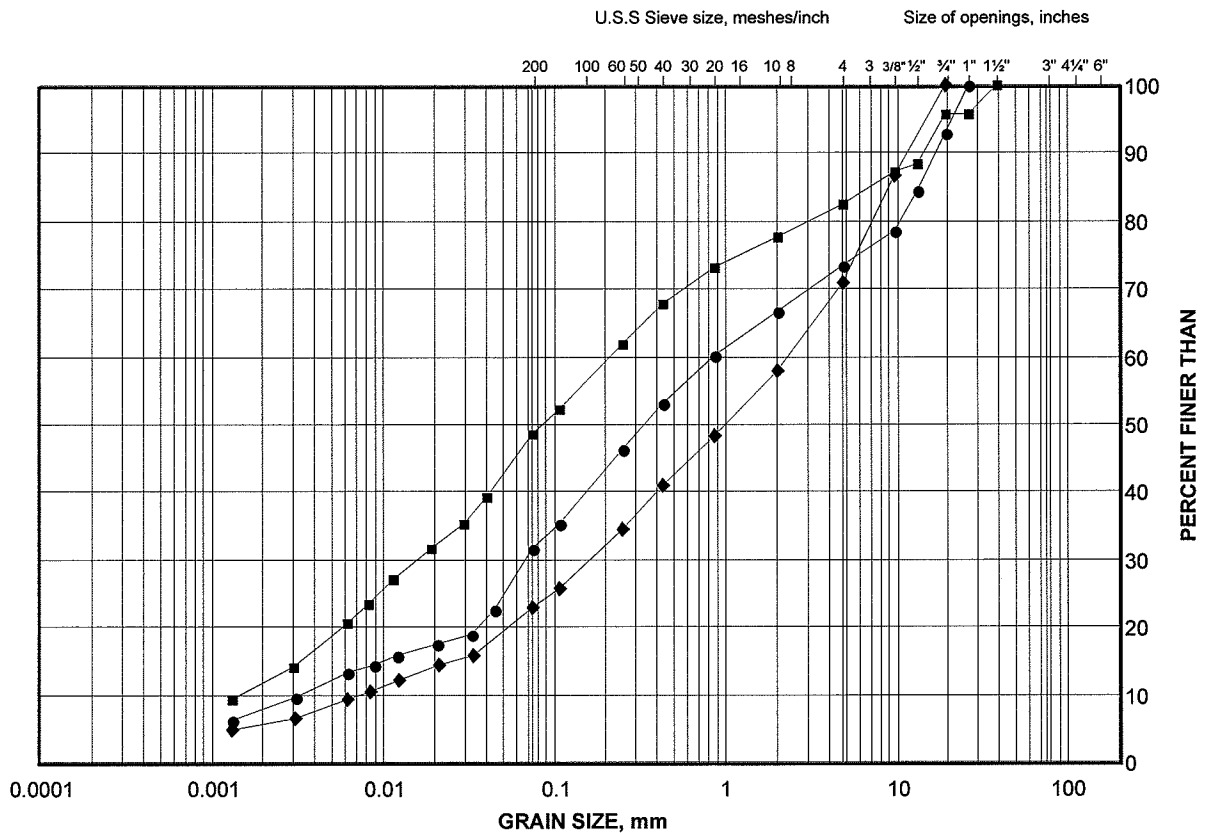
Golder Associates

Date: 01-Jun-07

GRAIN SIZE DISTRIBUTION

Sandy Silt to Silty Sand (Till)

FIGURE A7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-9	5	241.6
■	06-10	6	239.8
◆	06-9	8	237.2

Project Number: 04-1111-024E

Checked By: KJB

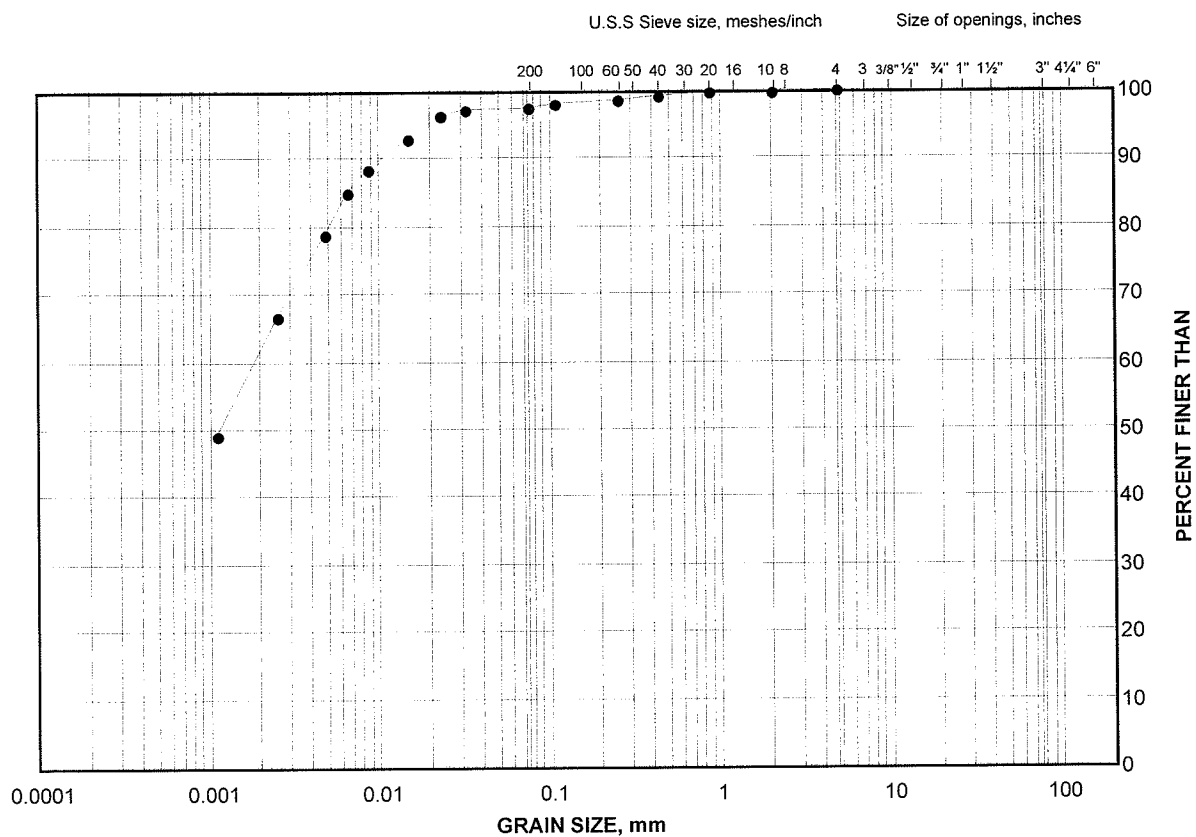
Golder Associates

Date: 07-Mar-08

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE A8



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

LEGEND

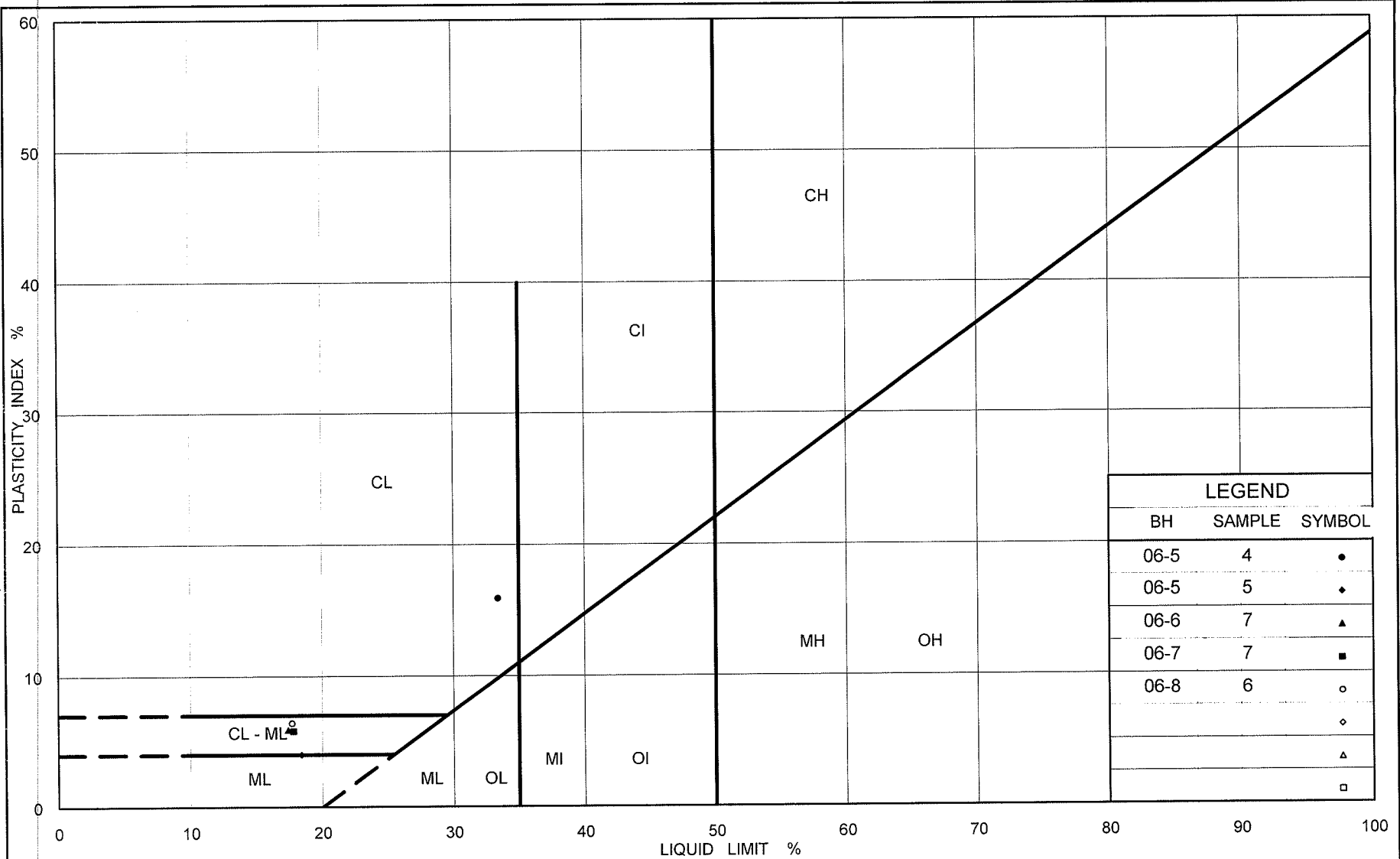
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-6	5	238.4

Project Number: 04-1111-024E

Checked By: *KTB*

Golder Associates

Date: 01-Jun-07



Ministry of Transportation

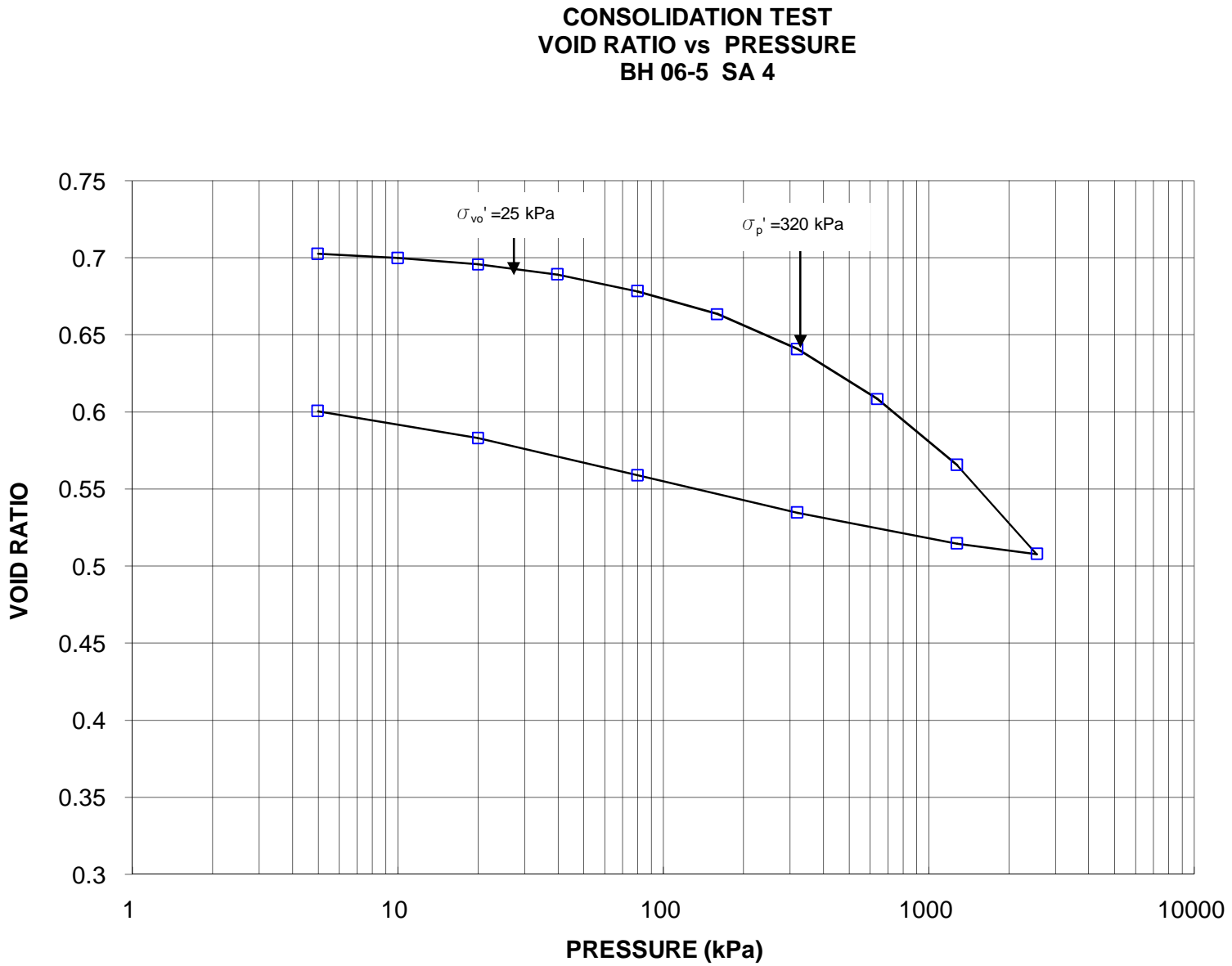
Ontario

PLASTICITY CHART Clayey Silt

FIG No. A9

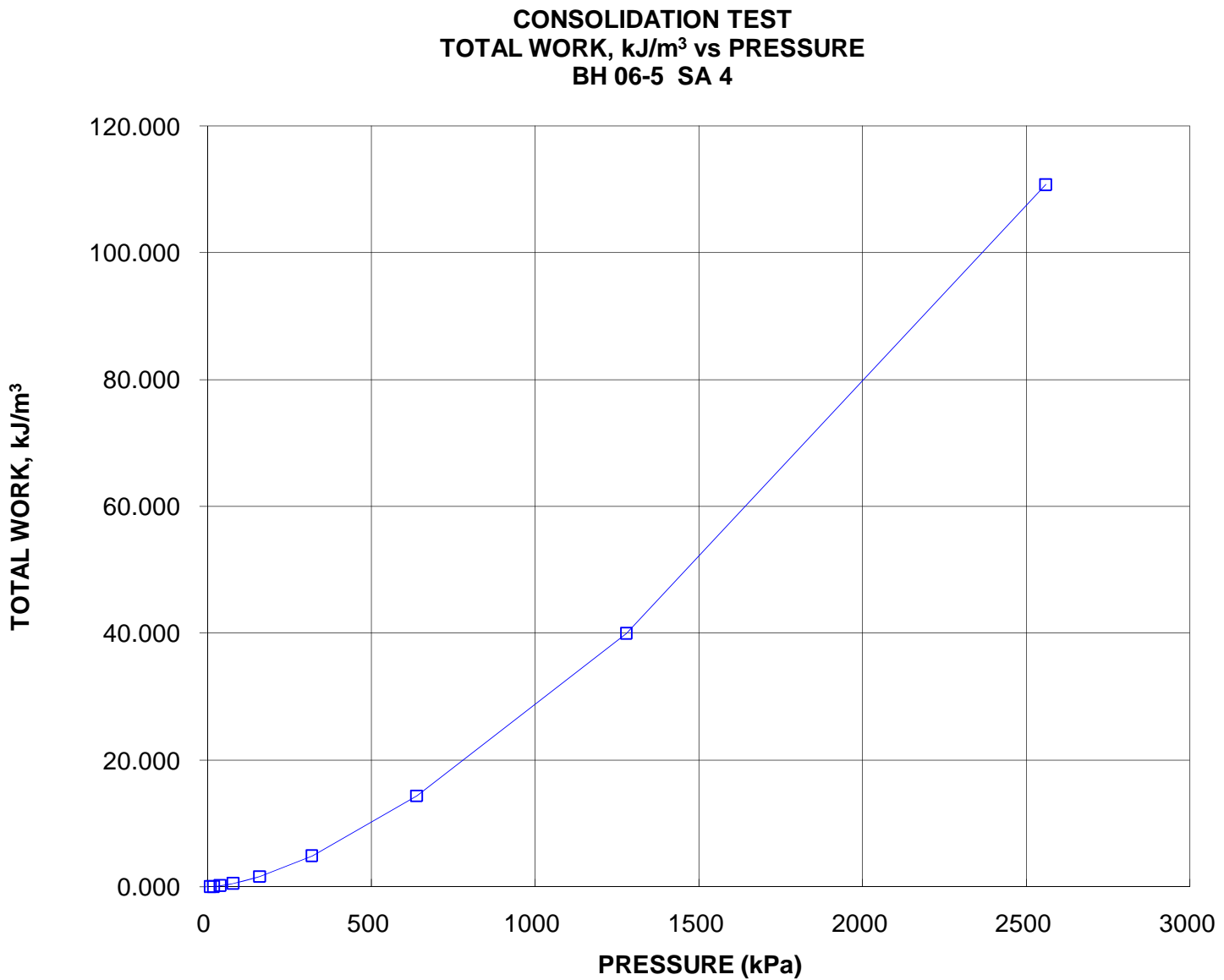
Project No. 04-1111-024E

Checked By: KSB



**CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE**

FIGURE A11



OEDOMETER CONSOLIDATION SUMMARY

FIGURE A12

SAMPLE IDENTIFICATION

Project Number	04-1111-024E	Sample Number	4
Borehole Number	06-5	Sample Depth, m	3.0-3.7

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	08/29/2006		
Date Completed	09/11/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.27	Unit Weight, kN/m ³	19.84
Sample Diameter, cm	4.95	Dry Unit Weight, kN/m ³	15.87
Area, cm ²	19.21	Specific Gravity, measured	2.76
Volume, cm ³	24.39	Solids Height, cm	0.745
Water Content, %	25.00	Volume of Solids, cm ³	14.30
Wet Mass, g	49.35	Volume of Voids, cm ³	10.09
Dry Mass, g	39.48	Degree of Saturation, %	97.9

TEST COMPUTATIONS

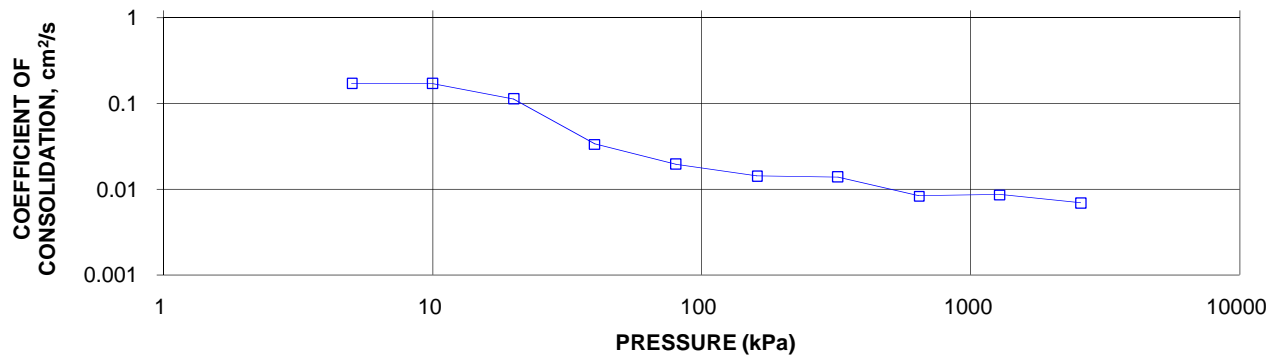
Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.270	0.705	1.270				
5.00	1.268	0.702	1.269	2	1.71E-01	3.15E-04	5.27E-06
10.00	1.266	0.700	1.267	2	1.70E-01	3.15E-04	5.25E-06
20.00	1.263	0.696	1.265	3	1.13E-01	2.36E-04	2.62E-06
40.00	1.258	0.689	1.261	10	3.37E-02	1.97E-04	6.50E-07
80.00	1.250	0.678	1.254	17	1.96E-02	1.57E-04	3.03E-07
160.00	1.239	0.664	1.245	23	1.43E-02	1.08E-04	1.51E-07
320.00	1.222	0.641	1.231	23	1.40E-02	8.37E-05	1.14E-07
640.00	1.198	0.608	1.210	37	8.39E-03	5.91E-05	4.85E-08
1280.00	1.166	0.565	1.182	34	8.71E-03	3.94E-05	3.36E-08
2560.00	1.123	0.508	1.145	40	6.94E-03	2.65E-05	1.80E-08
1280.00	1.128	0.514	1.126				
320.00	1.143	0.535	1.136				
80.00	1.161	0.559	1.152				
20.00	1.179	0.583	1.170				
5.00	1.192	0.600	1.186				

Note:
k calculated using cv based on t_{90} values.

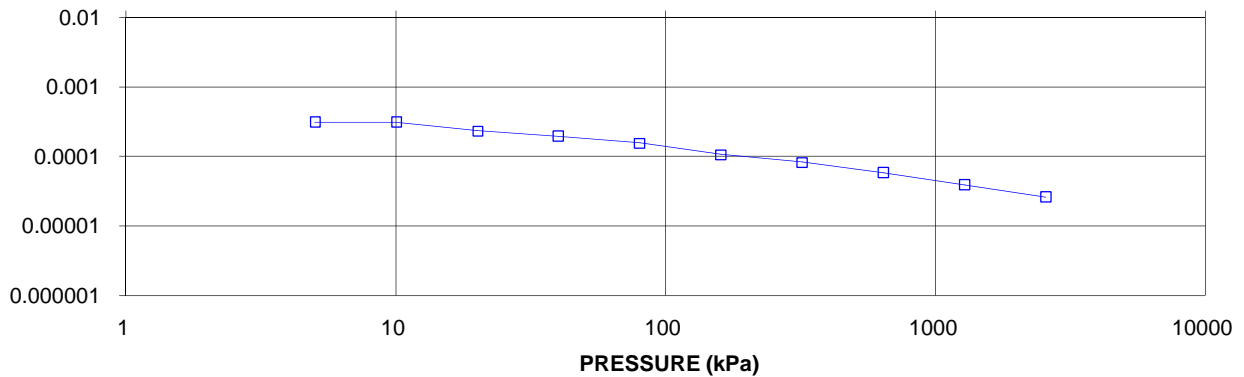
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.19	Unit Weight, kN/m ³	20.70
Sample Diameter, cm	4.95	Dry Unit Weight, kN/m ³	16.91
Area, cm ²	19.21	Specific Gravity, measured	2.76
Volume, cm ³	22.89	Solids Height, cm	0.745
Water Content, %	22.39	Volume of Solids, cm ³	14.30
Wet Mass, g	48.32	Volume of Voids, cm ³	8.59
Dry Mass, g	39.48		

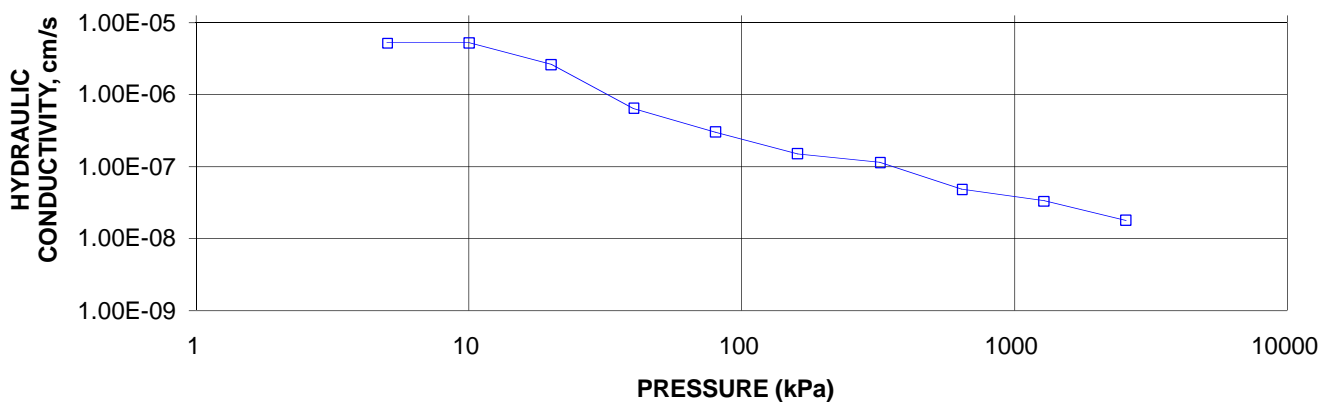
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH 06-5 SA 4



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 06-5 SA 4



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 06-5 SA 4



Silt

Grain size distribution plot showing Percent Finer Than versus Grain Size (mm) and U.S.S. Sieve size (meshes/inch). The plot displays two data series: one represented by solid circles and another by solid squares. Both series show a similar trend, indicating a well-sorted sand with a median grain size (d₅₀) of approximately 0.075 mm.

Grain Size (mm)	U.S.S. Sieve size (meshes/inch)	Percent Finer Than (%) - Circles	Percent Finer Than (%) - Squares
0.0075	20	15	10
0.015	10	25	15
0.03	60	40	25
0.06	30	55	40
0.12	15	70	55
0.25	60	85	70
0.5	30	95	85
1.0	15	100	100

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

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	06-8	8	233.7
■	06-7	8	235.3

Checked By: **KTB**

Date: 01-Jun-07