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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
PRELIMINARY DESIGN
CULVERT REPLACEMENT AT STATION 16+065.5
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 28
PETERBOROUGH, ONTARIO
G.W.P. 73-99-00, SITE NO. 26-215

Submitted to:

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF DRAWINGS

LIST OF APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
PRELIMINARY DESIGN
CULVERT REPLACEMENT AT STATION 16+065.5
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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 preliminary foundation investigation 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). This report addresses the preliminary foundation investigation carried out for the proposed culvert replacement at Station 16+065.5 (Site No. 26-215) as part of the Highway 7 improvement project. Preliminary foundation investigation and design services are required for a total of four structures (i.e. Jackson Creek Bridge, CNR Overhead Site and two structural culvert sites) in four separate reports for this project. The scope of foundations work for this project was expanded to include several swamp and high fill areas as outlined in Golder's letter titled "Proposal for Addendum Foundation Investigation", dated April 6, 2006 and approved by MTO on April 8, 2006. The work was carried out in accordance with the Quality Control Plan for this project dated April 20, 2006.

The purpose of this investigation is to establish the subsurface conditions at the proposed culvert extension / replacement site by borehole drilling, dynamic cone penetration testing, in situ testing and laboratory testing on selected samples. A plan drawing of the existing culvert location was provided to Golder by NCE in July 2006.

2.0 SITE DESCRIPTION

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

Within the existing MTO right-of-way, the site generally consists of the raised highway embankment and developed commercial properties on the west side of the highway and low-lying grassy area on the east side of the highway. The west side of the highway is generally flat with gently sloping grades that lead to storm drains and pipes that empty into the Trout Creek tributary, just downstream of the existing culvert outlet. The east side of the highway embankment consists of grass-covered side-slopes (sloped at about 1.5H:1V to 2H:1V) that lead into ditches that drain into the upstream side of the culvert. The existing culvert structure consists of an open footing concrete culvert measuring about 3.7 m wide x 1.5 m high x 18.4 m long. The founding level of the footings was not known at the time this report was prepared and should be verified prior to detail design. The existing culvert structure allows passage of the Trout Creek tributary from east to west beneath Highway 7 in this area. It is proposed that the existing structure is to be lengthened or replaced as part of the highway intersection improvements and five-laning (including a turning lane) of Highway 7 at this location. There is an existing retaining wall structure located near the culvert outlet (i.e. west end) on the south bank of the tributary. The proposed embankment widening will require removal of the retaining wall structure and if a new wall is to be constructed, investigation and design will be required during the detailed design.

Based on the MTO drawings provided to us titled "Engineering and Title Records", dated May 2001 and electronic drawing provided to us by NCE titled "2004-002 recommended plan.dwg", received July 31, 2006, the existing Highway 7 road surface at the site is at about Elevation 200.4 m, the existing culvert obvert is at about Elevation 199.5 m, and the tributary creek bed within the existing culvert is at about Elevation 198.3 m.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for this culvert investigation was carried out between July 5 and 7, 2006 during which time two (2) boreholes combined with Dynamic Cone Penetration Tests (DCPTs) were advanced. The combined boreholes and DCPTs, numbered 06-1 and 06-2, were advanced at the approximate locations shown in plan on Drawing 1.

The current 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 107 mm outside diameter (O.D.) solid stem augers. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in depths, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Dynamic Cone Penetration Tests (DCPTs) were carried out in both boreholes and were terminated on effective refusal (greater than 100 blows per 0.3 m of penetration).

The boreholes were sampled to depths of 6.7 m below ground surface. DCPT's were advanced from the bottom of the sampled boreholes and were terminated at depths of 12.2 m and 9.5 m below ground surface for boreholes 06-1 and 06-2, respectively. The groundwater conditions in the open boreholes were observed during the drilling operations and a piezometer was installed in borehole 06-1 to permit monitoring of the groundwater level at this location. The piezometer 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 piezometers will require decommissioning prior to or during construction in accordance with O.Reg 903. The piezometer installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report.

The field work was supervised by a member of our engineering technical staff, who located the boreholes, arranged for the clearance of underground services, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our 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 06-2.

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 Drawing 1.

Borehole Number	*MTM NAD83 Northing (m)*	*MTM NAD83 Easting (m)*	*Ground Surface Elevation (m)*
06-1	4900609.0	392478.2	198.6
06-2	4900606.9	392441.9	200.2

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.

[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.

4.2 Subsoil Conditions

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

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

In general, the subsoils at the culvert replacement / extension site consist of a surficial layer of peat, underlain by a layer of clayey silt to silty clay. The clayey silt to silty clay was underlain by a deposit of silty sand. At borehole 06-2, which was drilled on top of the raised embankment area located on the west side of the highway and near the north crest of the creek valley, a surficial layer of topsoil underlain by sand and gravel fill was generally encountered above the clayey silt to silty clay deposit. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil / Peat

A surficial layer of peat, some 0.5 m thick, was encountered in borehole 06-1 at the toe of the highway embankment. A surficial topsoil layer was encountered, some 0.2 m thick, in borehole 06-2 at the crest of the embankment.

4.2.2 Fill

In borehole 06-2, underlying the topsoil, a layer of sand and gravel fill was encountered. This borehole was drilled at the west side of proposed culvert extension (near the north crest of the creek side-slope), where fill has been placed to match the approximate highway elevation. The top of the fill was encountered at a depth of 0.2 m (Elevation 200.0 m) and was 1.6 m thick.

A Standard Penetration Test (SPT) ‘N’ value recorded within the sand and gravel fill was 2 blows per 0.3 m of penetration, indicating a very loose relative density.

The natural water content measured on one sample of the sand and gravel fill was 8 per cent.

4.2.3 Upper Silty Sand

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

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

4.2.4 Clayey Silt to Silty Clay

A clayey silt to silty clay layer containing trace to some sand and gravel was encountered below the topsoil, peat and silty sand in boreholes 06-1 and 06-2. The cohesive layer contained sand and silt seams throughout. The top of the clayey silt to silty clay deposit was encountered at a depth of about 0.5 m and 2.1 m (Elevation 198.1 m) and the deposit was about 3.8 m and 3.7 m thick in boreholes 06-1 and 06-2, respectively. The colour of the cohesive deposit generally transitioned from brown to grey with depth. The sand and gravel content within the cohesive deposit generally decreased with depth.

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

The natural water content measured on select samples of the clayey silt to silty clay layer ranged between about 18 per cent and 26 per cent.

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

Borehole	***Sample***	***Elevation (m)***	***Liquid Limit (%)***	***Plastic Limit (%)***	***Plasticity Index (%)***
06-1	4	194.9 – 195.6	21	14	7
06-2	4	196.6 - 197.2	40	20	20
06-2	5	195.0 – 195.6	24	15	9

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

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

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

where:
σ_{vo}' is the effective overburden pressure in kPa
σ_p' is the 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 cm^2/s

Based on the test results, the silty clay sample used for the consolidation test (i.e. from borehole 06-2, Sa#4) is considered to be over-consolidated. A bulk unit weight of about 19.6 kN/m^3 and a specific gravity of 2.76 was measured on the consolidation test specimen. The consolidation test results are shown on Figures A2 to A5 (inclusive) in Appendix A.

4.2.5 Lower Silty Sand

A deposit of silty sand was encountered below the clayey silt to silty clay deposit in both boreholes 06-1 and 06-2. The silty sand contained trace to some gravel and clay. The top of this deposit was encountered at a depth of 4.3 m (Elevation 194.3 m) and 5.8 m (Elevation 194.5 m) at boreholes 06-1 and 06-2, respectively. The silty sand was penetrated about 2.4 m and 0.9 m (in boreholes 06-1 and 06-2, respectively) and both boreholes were terminated within this deposit, below which depth Dynamic Cone Penetration Tests (DCPT) were advanced in both boreholes. The DCPTs were terminated upon effective refusal at depths of about 12.2 m (Elevation 186.4 m) and 9.5 m (Elevation 190.7 m) for boreholes 06-1 and 06-2, respectively.

Standard Penetration Testing (SPT) ‘N’ values recorded within the lower silty sand deposit ranged from 8 blows to 38 blows per 0.3 m of penetration, indicating the silty sand is loose to dense.

The natural water content measured on a sample of the silty sand was 9 per cent. A grain size distribution curve for a selected sample of the silty sand is shown on Figure A6 in Appendix A.

4.2.6 Groundwater Conditions

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

Borehole	*Installation*	*Ground Surface Elevation (m)*	*Depth to Water Level(m)*	*Water Level Elevation (m)*	*Date*
06-1	Piezometer	198.6	4.5	194.1	July 7, 2006
			- 0.1*	198.7	July 10, 2006
			- 0.9*	199.5	July 31, 2006
			- 0.9*	199.5	Aug. 18, 2006
06-2	Open Borehole	200.2	1.6	198.6	July 5, 2006

Note : * Artesian Conditions

It should be noted that the piezometer readings indicate artesian conditions from within the silty sand up to about 0.9 m above the ground surface at borehole 06-1. Groundwater levels at the site of the culvert will depend on rainfall and snowmelt conditions and are expected to fluctuate seasonally.

5.0 CLOSURE

The field technician supervising the drilling program was Mr. Suresh Bainey. This report was prepared by Mr. Kevin J. Bentley, P.Eng., a geotechnical engineer. Mr. Fintan J. Heffernan, Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

GOLDER ASSOCIATES LTD.

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PART B

FOUNDATION DESIGN REPORT
PRELIMINARY DESIGN
CULVERT REPLACEMENT AT STATION 16+065.5
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 28
PETERBOROUGH, ONTARIO
G.W.P. 73-99-00, SITE NO. 26-215

6.0 PRELIMINARY ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the preliminary design of the proposed culvert extension / replacement as part of the proposed highway operational improvement plan. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the preliminary subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out preliminary design of the proposed culvert foundations. Where comments are made on construction they are provided in order to highlight those aspects which could affect the 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 borehole drilling will be required during the detail design phase of the project, when the details of the culvert extension / replacement and embankment widening are finalized. At that time, further investigation of the thickness of topsoil and peat and composition of the embankment fills below the existing road surface will be required to confirm temporary stability concerns during construction, dewatering requirements and excavation / fill requirements. For the proposed culvert extension under the proposed embankment widening as part of the five-laning of Highway 7 (including the turning lane), additional borehole drilling is recommended at the culvert ends (i.e. on the opposite side of the culvert to where the current boreholes are drilled), to confirm the thickness and consistency of the clayey silt to silty clay soils, refine the predicted magnitude and time rate of settlement under the new embankment loading, and to further assess any stability / settlement concerns and provide mitigation options and develop the necessary operational constraints and/or special provisions for the contract.

The type, location and founding depth of the existing open footing culvert should be verified prior to detail design such that the existing footing or any disturbance related to removal of the existing footing will not compromise the integrity of the proposed new culvert foundations.

6.1 General

As mentioned previously, the existing Highway 7 road surface at the site is at about Elevation 200.4 m and the existing open footing culvert obvert is at about Elevation 199.5 m. The existing structure consists of an open footing concrete culvert that measures about 3.7 m wide x 18.4 m long. The elevation of the creek bed at the inlet and outlet of the culvert is about Elevation 198.3 m, resulting in a height of about 1.2 m from the creek bed to culvert obvert. The existing open footing foundation width and depth was not known at the time of this report and should be confirmed prior to detail design. Based on the results of a visual inspection of the existing culvert structure (performed by Harmer Podolak Engineering Consultants), we understand that the culvert is approaching the end of it service life and will need to be replaced as part of the operational improvements. The culvert will also need to be lengthened in order to permit widening the highway embankment to accommodate five-laning and intersection improvements (see Drawing 1).

Based on conversations with the designer, the existing roadway elevation will remain unchanged and the height of the future embankments for the five-laning of Highway 7 will likely match the current road elevation. Based on the existing topography, the new embankment widening is expected to be up to about 2 m high (i.e. above the existing ground surface). The culvert should be designed to withstand the maximum anticipated overburden, lateral pressures and live loads. The effects of frost should also be considered in the structural design if frost susceptible soils are located within the depth of frost penetration.

Several material type alternatives (i.e. steel and concrete) for the culvert replacement have been considered based on the information presented in the NCE memorandum titled " Evaluation of Structural Alternatives, Structural Culvert MTO Site No. 26-215 Station 16+065.5", dated November 29, 2006. The preferred replacement structure was established to be an open footing precast concrete culvert (to maintain a native channel bed) measuring about 3.7 m wide x 1.3 m high x 32.5 m in length.

6.2 Culvert Foundation Options

Apart from the existing topsoil and fill placed on the west side of the highway (i.e. previous grade raise) and existing road embankment fill, the shallow subsoils at the culvert location generally consist of a surficial layer of peat, underlain by a clayey silt to silty clay deposit. A thin layer of silty sand was present between the fill and clayey silt layer in borehole 06-2. Underlying the clayey silt to silty clay deposit, a silty sand deposit was encountered. Groundwater was encountered about 1.6 m (Elevation 198.6 m) below ground surface on the west side of the culvert (i.e. below the fill area) and about 4.5 m (Elevation 194.1 m) below ground surface on the east side of the culvert within the open boreholes. However, the piezometer, which was sealed below the clayey silt to silty clay layer and within the lower silty sand deposit, measured artesian conditions with water levels up to about 0.9 m above ground surface (Elevation 199.5 m) in August 2006. It should be noted that the water levels in this area will fluctuate on a seasonal basis.

Based on the subsurface information obtained, the upper portion of the native firm to very stiff clayey silt to silty clay soils located below the surficial topsoil, peat, fill and very loose silty sand deposits are considered suitable for the support of the proposed culvert foundation. Table 1 provides a comparison of culvert foundation options based on advantages, disadvantages, risks, relative costs and consequences. Both open footing and closed box culverts have been included as options. The options also assume that open footings will be cast-in-place and the culvert structure placed or constructed on top of the footing as either a precast or cast-in-place unit. Closed box culverts can be either precast or cast in place as a single unit. From a foundations perspective, the preferred culvert replacement structure type at this location is an open footing culvert with precast culvert units. Given that the culvert replacement is to be located at the same location as the existing culvert, the preferred alternative allows founding the culvert footings at or below the existing culvert open footing level, which was not known at the time of this report. Some localized stripping of the topsoil, peat and any very loose or highly organic soils that are present at this site will also be required as part of the embankment construction.

6.2.1 Geotechnical Resistance

6.2.1.1 Preferred Alternative - Open Footing Culvert

The approximate invert (creek bed) elevation, recommended footing founding level (i.e. depth of subexcavation), and the founding soil type at the footing level for the proposed open footing culvert replacement / extension are presented below.

Approximate Culvert Station	Culvert	Relevant Boreholes	Approximate Creek Bed Elevation (m)	*Recommended Founding Levels for Proposed Culvert Footing (m)	Founding Soil Type
16+065.5	Culvert Replacement / Extension	06-1, 06-2	198.3	197.0 – 198.0	Clayey Silt to Silty Clay containing silt and sand seams

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

The above recommended founding levels indicate the estimated minimum and maximum target elevations required to reach the appropriate founding subgrade soil(s). The actual founding level will depend on the final location of the culvert, the design invert level and frost protection requirements, the elevation of the existing footings, and results of additional boreholes / investigation during detailed design.

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

Approximate Culvert Station	Culvert	Assumed Open Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa) for 25 mm settlement
16+065.5	Culvert Replacement / Extension	0.5	225	150

The geotechnical resistance values assume the culvert is founded on the undisturbed native silty clay to clayey silt soils which for design may be assumed to extend to the founding (i.e. subexcavation) levels indicated above. It should be noted that lower strength clayey soils were encountered below about Elevation 196 m, (i.e. below the apparent crust) within the cohesive deposit. In addition, artesian groundwater conditions were encountered within the lower silty sand layer (located within about 2.5 m to 3.5 m of the proposed founding elevation). As a result, it is recommended that subexcavation / founding depths be as high as possible to reduce the risk of unstable founding soils and to reduce dewatering efforts. It should be noted that, depending on design founding elevation and the water levels at the time of construction, dewatering may be required to limit the potential for boiling, loosening, and/or disturbance of the founding clayey soils containing silt and sand seams at the founding level. As a result, excavation for and construction of the open footings should be in accordance with Special Provision No. 902S01.

6.2.1.2 Box Culvert

As an alternative to an open footing culvert, a box culvert (i.e. concrete box or steel arch supported on a base slab) could be used. The approximate invert elevation, recommended level of subexcavation, and the founding soil type for a box culvert is presented below.

Approximate Culvert Station	Culvert	Relevant Boreholes	Approximate Creek Bed Elevation (m)	Recommended Subexcavation Level for Proposed Culvert (m)	Founding Soil Type
16+065.5	Culvert Replacement / Extension	06-1, 06-2	198.3	197.0 – 198.0	Clayey Silt to Silty Clay containing silt and sand seams

The above recommended subexcavation level indicates the estimated minimum and maximum target elevation required to reach the appropriate founding subgrade soil. The actual founding level will depend on the final location of the culvert, thickness of the bottom of the culvert, any overlying substrate material, design invert level and frost protection requirements, the depth of the granular bedding and/or engineered fill required under the culvert (see Section 6.7.3), and the results of additional boreholes / investigation during detailed design.

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

Approximate Culvert Station	**Culvert**	**Total Proposed Culvert Width (m)**	**Factored Geotechnical Resistance at ULS (kPa)**	**Geotechnical Resistance at SLS (kPa) for 25 mm settlement**
16+065.5	Culvert Replacement / Extension	3.7	225	150

The geotechnical resistance values assume the culverts are founded on granular bedding soils supported on approved engineered fill and/or undisturbed native clayey silt to silty clay soils which for design may be assumed to extend to the subexcavation levels indicated above. As mentioned previously, lower strength soils were encountered below about Elevation 196 m within the cohesive deposit. In addition, artesian groundwater conditions were encountered within the lower silty sand layer (located within about 2.5 m to 3.5 m of the proposed founding elevation). As a result, it is recommended that subexcavation / founding depths be as high as possible to reduce the risk of unstable founding soils and to reduce dewatering efforts.

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

6.2.2 Resistance to Lateral Loads

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

For the open footing culvert option, assuming the footings are cast-in-place and founded on undisturbed clayey silt to silty clay, a coefficient of friction value (tan δ) of 0.43 can be used for design. In accordance with the *CHBDC*, a factor of 0.8 is to be applied to the coefficient of friction value when calculating the horizontal resistance.

6.2.3 Frost Protection

The frost penetration depth in the area of the proposed culvert is about 1.5 m. All shallow foundations should be provided with a minimum of 1.5 m of soil cover or equivalent thermal insulation for frost protection.

6.3 Lateral Earth Pressures for Preliminary Design

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

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular ‘A’ or Granular ‘B’ Type II, but with less than 5 percent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the culvert / wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the wall stem (Case I in Figure C6.9.1(l)(i) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l)(ii) of the *Commentary to the CHBDC).*

- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade Material (SSM) for the new portions of the approach embankments:

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

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

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

- If the culvert structure allows lateral yielding of the culvert walls, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (such as typically the case for a rigid concrete box culvert), at-rest earth pressures should be assumed for geotechnical design.

- Seismic loading will result in increased lateral earth pressures acting on the culvert wall and any retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.1 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio (A) for Peterborough is 0.05. Based on experience, for the overburden soils at the site and embankment heights of up to about 2.5 m, a 10 to 20 per cent amplification of the ground motion may occur, resulting in an increase in the ground surface acceleration from 0.05g to between 0.055g and 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. k_h = 0.03). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. k_h = 0.09). The seismic active earth pressure coefficients are also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2.3k_h$, $k_v = 0$, and $k_v = -2/3$.

- The following seismic active earth pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE}

obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

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

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

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

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

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

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

6.4 Settlement

6.4.1 Culvert Replacement and Embankment Widening

Referring to Drawing 1, we understand that the existing embankment will be widened by up to about 6 m on each side to match the existing roadway profile. Based on conversations with the designer, it is understood that the new embankment will match the existing Highway 7 road grade (Elevation 200.4 m). Based on the boreholes and surrounding topography, the existing ground surface is typically less than 0.2 m below the proposed road surface on the west side of the culvert replacement (outside of the tributary creek valley) and generally about 2 m below the road surface on the east side of the culvert at the borehole locations.

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

For the proposed culvert replacement and embankment widening on the east side, settlements are anticipated to be greater due to the increased loading imposed on the clayey subsoils from the new embankment loading. Assuming the new embankment footprint will be stripped of topsoil and peat and replaced with engineered fill, the total thickness of the new embankment fill on the east side is anticipated to be up to about 2.5 m. The total net loading on the foundation soils (after stripping, backfilling and embankment fill construction) from the embankment widening on the east side was estimated to be about 50 kPa.

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 borehole and consolidation test data.

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

Soil	***Initial Void Ratio*** ***e_o***	***Recompression Index*** ***C_r***	***Compression Index*** ***C_c***	***OCR***
Clayey Silt to Silty Clay	0.74	0.02	0.19	7

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

Soil	***Thickness (m)***	***Bulk Unit Weight (kN/m^3)***	***Deformation Properties***
Clayey Silt to Silty Clay	4	20	See table above
Lower Silty Sand Deposit	5	20	E′= 15 MPa

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

The predicted maximum total settlement of the foundation soils at the culvert location in the area of the east widening is estimated to be about 40 mm due to the loading imposed by the new embankment fill. The total is estimated to be comprised of about 15 mm of immediate settlement due to compression of the cohesionless soil deposit at depth and about 25 mm of time dependent settlement of the cohesive soil layer. Based on an estimated coefficient of consolidation (c_v) of 4.5×10^{-2} cm^2/s and assuming two-way drainage of the approximately 4 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 deeper confined silty sand deposit is expected to occur gradually over about 2 months.

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

Considering the central portion of the proposed culvert replacement has been preloaded by the existing roadway embankment, the estimated maximum total settlement of 40 mm at the east widening area is anticipated to be differential to the central portion of the culvert. As a result, preloading is recommended at the embankment widening areas, specifically on the east side. The new embankment widening adjacent to the culvert extension should be constructed and allowed to settle at the proposed culvert location for at least 2 months prior to culvert installation. A camber could be incorporated into the design of the culvert to manage the expected settlements after placement of fill above and adjacent to the culvert where preloading may not be practical given the close proximity to the creek.

Other options (other than preloading) to mitigate the impacts of consolidation settlement could include surcharging, use of light weight fill, partial or full subexcavation, or cambering the culvert with articulating joints. 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. Cambering of the culvert using articulated joints will require specialized construction and may not be cost effective.

6.5 Stability

Based on the results of the boreholes and assuming the height of the proposed embankment widening will match the existing road elevation and consist of granular earth fill sloped at 2H:1V or shallower, the global stability of the embankment is not considered to be a concern. The factor of safety of the proposed embankment is estimated to be greater than 1.3, however, the global stability should be checked after the detailed design investigation is complete.

6.6 Base Heave

As previously mentioned, artesian groundwater conditions were encountered within the lower silty sand layer at about 4 m to 6 m depth (about Elevation 194.5 m). Temporary subexcavations during culvert installation should be maintained as shallow as possible to reduce the risk of basal heave or unstable founding soil conditions within the clayey silt to silty clay soils containing seams of silt and sand. Given the artesian water conditions encountered within the underlying lower silty sand, preliminary calculations indicate that a soil thickness of at least 3 m must be maintained for a Factor of Safety against basal heave of about 1.3. As such, subexcavation to about Elevation 197.5 m, the average level recommended for the open footing or box culvert replacement options (see Section 6.2.1), can be carried out with an adequate factor of safety against base heave. However, subexcavations to lower elevations, as may be required for detail design to remove the existing culvert footings or achieve adequate frost protection, etc. could have a factor of safety less than 1.3 against base heave. The potential for base heave will have to be re-addressed after the detail design investigation is complete and the founding level of the existing footings are known. The level of artesian pressure should also be determined prior to construction. Depending on the detail design, relief wells may be required to lower the artesian water pressures to below the founding level.

6.7 Preliminary Considerations for Culvert Construction

6.7.1 Subgrade Preparation and Excavation

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

Based on the preliminary boreholes and depending on final design founding elevations, it is anticipated that excavations up to about 3.4 m below the existing road and ground surface at the west widening area and up to about 1.6 m at the east widening area will be required to remove the topsoil, peat, and existing fill and silty sand soils, and expose the native clayey silt and silty clay soils. It is not known whether the peat / organic soils were removed as part of the original culvert / embankment construction, as such, detail design investigation should investigate the subsoil conditions below the existing roadway at the proposed culvert replacement site, in order to more accurately estimate subexcavation depths. Depending on construction staging and sub-excavation depths (after detailed design), the culvert replacement and extension may require temporary shoring to maintain at least two lanes of traffic on Highway 7 during construction.

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.

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

6.7.2 Groundwater and Surface Water Control

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

More extensive groundwater control (e.g. relief wells) may be required for excavations that extend deeper into the native clayey silt to silty clay layer. In such cases, the piezometric water level within the lower silty sand soils should be lowered to a minimum of 0.5 m below the base of the proposed excavation level prior to initiation of the excavation in order to limit the potential for disturbance of the native soils and allow placement of concrete in the dry. Consideration should be given to carrying out additional investigation in the area to better define the thickness, consistency and frequency of water-bearing seams within the clayey silt to silty clay layer and assess dewatering requirements.

6.7.3 Bedding and Backfill

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

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

Frost treatment for the culvert structure should follow the guidelines provided in OPSD 803.010 for box and open footing culverts. In order to reduce the potential for frost damage to the culvert (i.e. differential heave and settlement), the combined thickness of backfill and bedding should be at least 1.5 m (i.e. equal to the depth of frost penetration).

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

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

6.7.4 Erosion Protection

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

In order to prevent water from flowing either beneath the culvert (potentially causing undermining and scouring for box culverts) or around the culvert (creating seepage through the embankment fill and potentially causing erosion and loss of fine particles), a clay seal or cut-off headwall should be provided at the upstream end of the culvert.

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

7.0 CLOSURE

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

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TABLE 1
EVALUATION OF CULVERT FOUNDATION ALTERNATIVES
HIGHWAY 7 CULVERT REPLACEMENT (SITE NO. 26-215)
G.W.P. 73-99-00

Option	*Option*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Open Footing Culvert	1	• Routine excavation and construction procedure; • Shallow sub-excavation depth; • Proposed founding elevation is likely at or below existing open footing founding level (however, this in not known at this time); • Reduced construction time for this option if precast culvert units are used as opposed to cast-in-place.	• Dewatering required in order to pour concrete footings "in the dry"; • Limited soil cover above spread footings for frost protection. • Longer construction time for cast-in-place versus precast culvert units	• Comparable costs to Option No. 2 due to dewatering and requirement to place concrete footings "in the dry", especially if precast culvert unit is used; • Higher costs if footing and 3-sided box are cast-in-place.	• Risk of base heave with deeper excavations; • If steel arch used, extra care and time required to compact backfill in close proximity to steel structure and in haunch areas. • Reduced risk of environmental disturbance to stream if existing culvert can be used during construction of a new, wider culvert open footing.
Closed Concrete Box	2	• Routine excavation and construction procedure; • Shallow sub-excavation depth.	• Dewatering required to place and compact bedding layer and/or engineered fill;	• Comparable cost to Option No. 1 due to costs associated with placing/compacting bedding layer and dewatering.	• Risk of base heave with deeper excavations; • If steel arch used, extra care and time required to compact backfill in close proximity to steel structure and in haunch areas; • Temporary diversion to natural stream required to place box units.
Deep Foundations (i.e. Piles or Caissons)	NP		• Pile/caisson tip elevations anticipated to be in excess of 10 m below ground surface.	• Much higher costs than Option No. 1 and No. 2.	

NP = not feasible or not practical

Additional Notes:

1. Preferred option as determined from structural, drainage, and fisheries issues is a Precast Concrete Open Footing culvert on cast-in-place footings.
2. If existing culvert footings are founded at depth, there is increased risk for basal heave during removal, sub-excavation, and placement of new foundations.

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_1	liquid limit
w_p	plastic limit
l_p	plasticity index $= (w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p)/I_p$
I_C	consistency index $= (w_1 - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

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

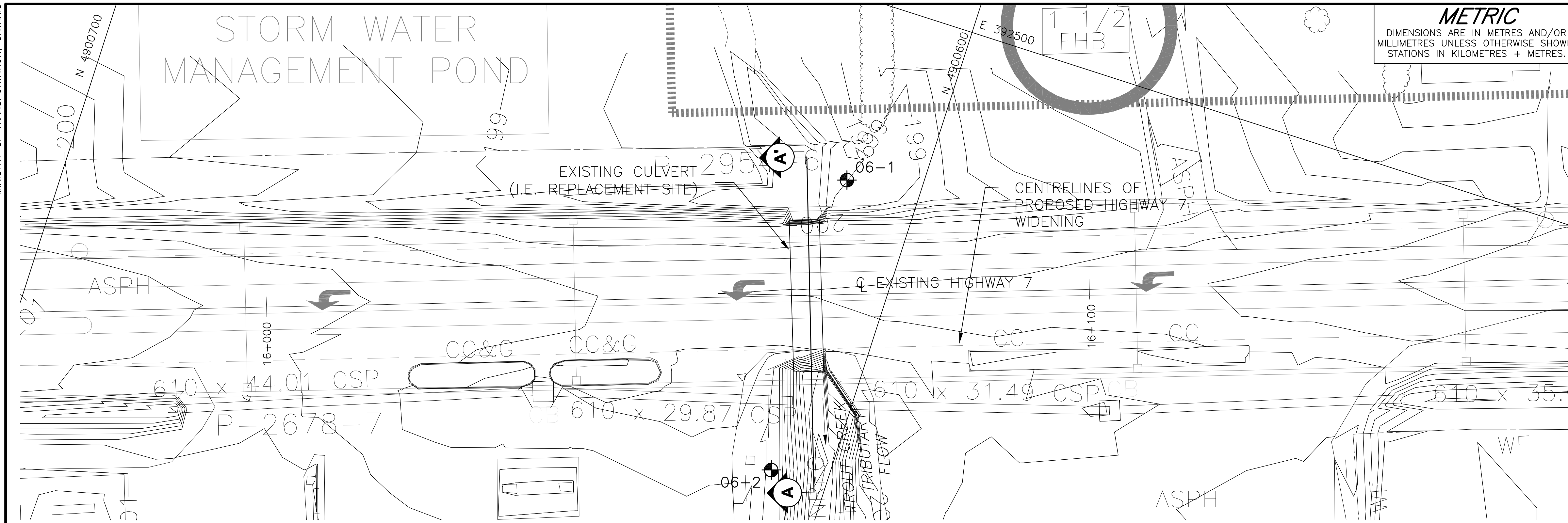
CONT No. 31D428
WP No. 73-99-00

HIGHWAY 7
CULVERT AT STATION 16+065.5
BOREHOLE LOCATIONS AND SOIL STRATA

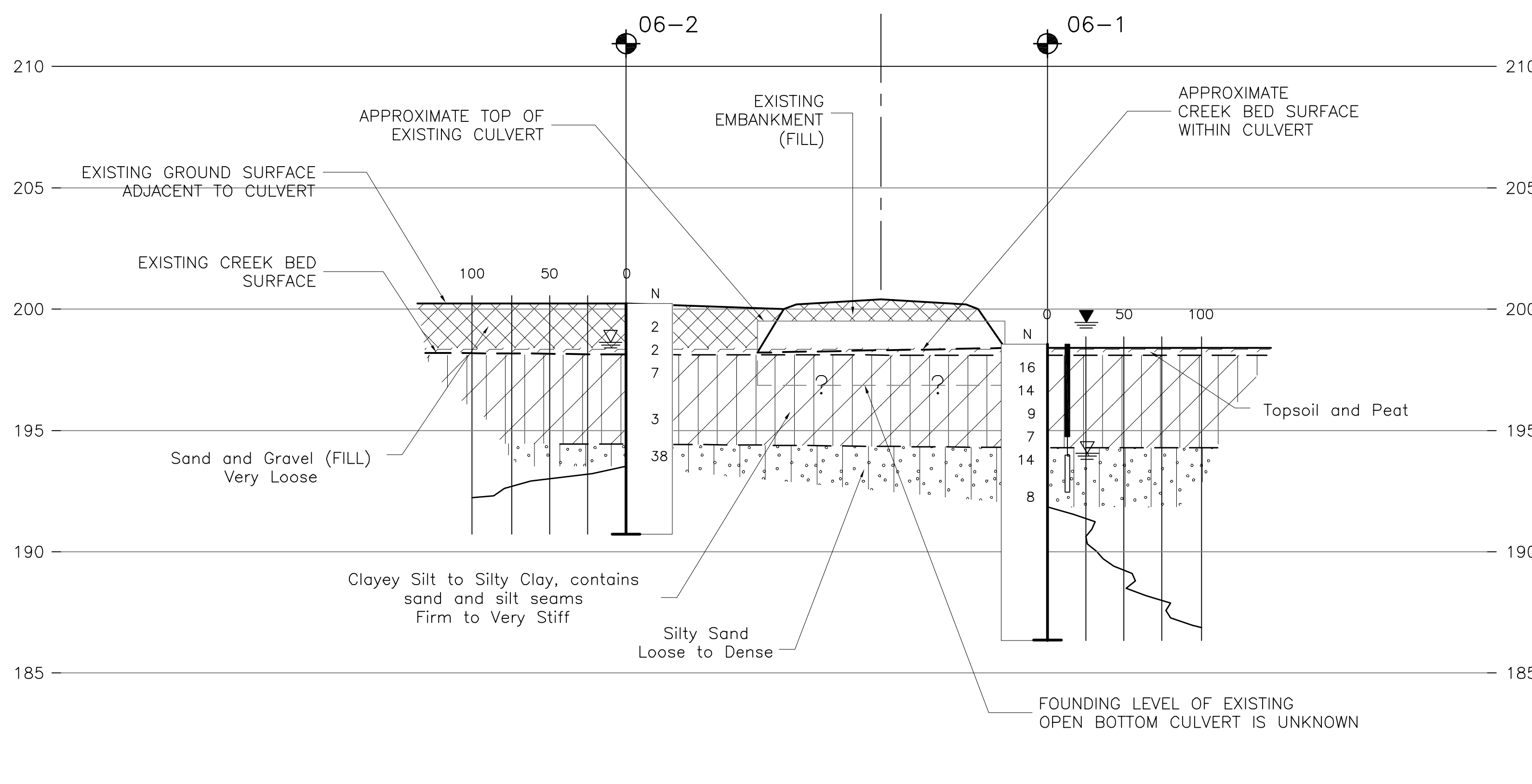
SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE 6 0 6 12 m

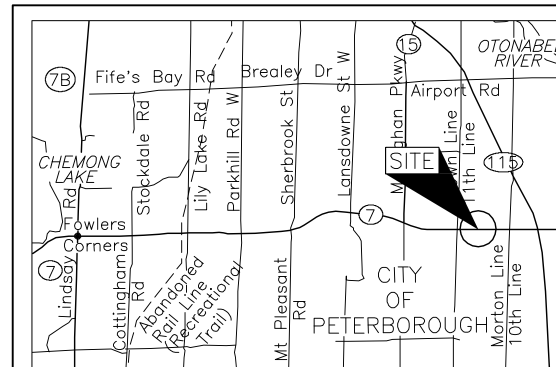


A-A'
1

℄ ALONG EXISTING HIGHWAY 7 CULVERT (STATION 16+065.5)

SCALE HOR 6 0 6 12 m

SCALE VER 3 0 3 6 m



KEY PLAN
SCALE 2 0 2 km

LEGEND

Borehole & Dynamic Cone Penetration Test

Seal

Piezometer

N Standard Penetration Test Value

16 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
06-1	198.6	4900609.0	392478.2
06-2	200.2	4900606.9	392441.9

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the 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 July 31, 2006.

NO.	DATE	BY	REVISION

Geocres No. 31D-428

HWY. 7		PROJECT NO. 04-1111-024C	DIST.
SUBM'D. SLP	CHKD. KJB	DATE: JAN 2007	SITE: 26-215
DRAWN: MSM	CHKD. KJB	APPD. FJH	DWG. 1

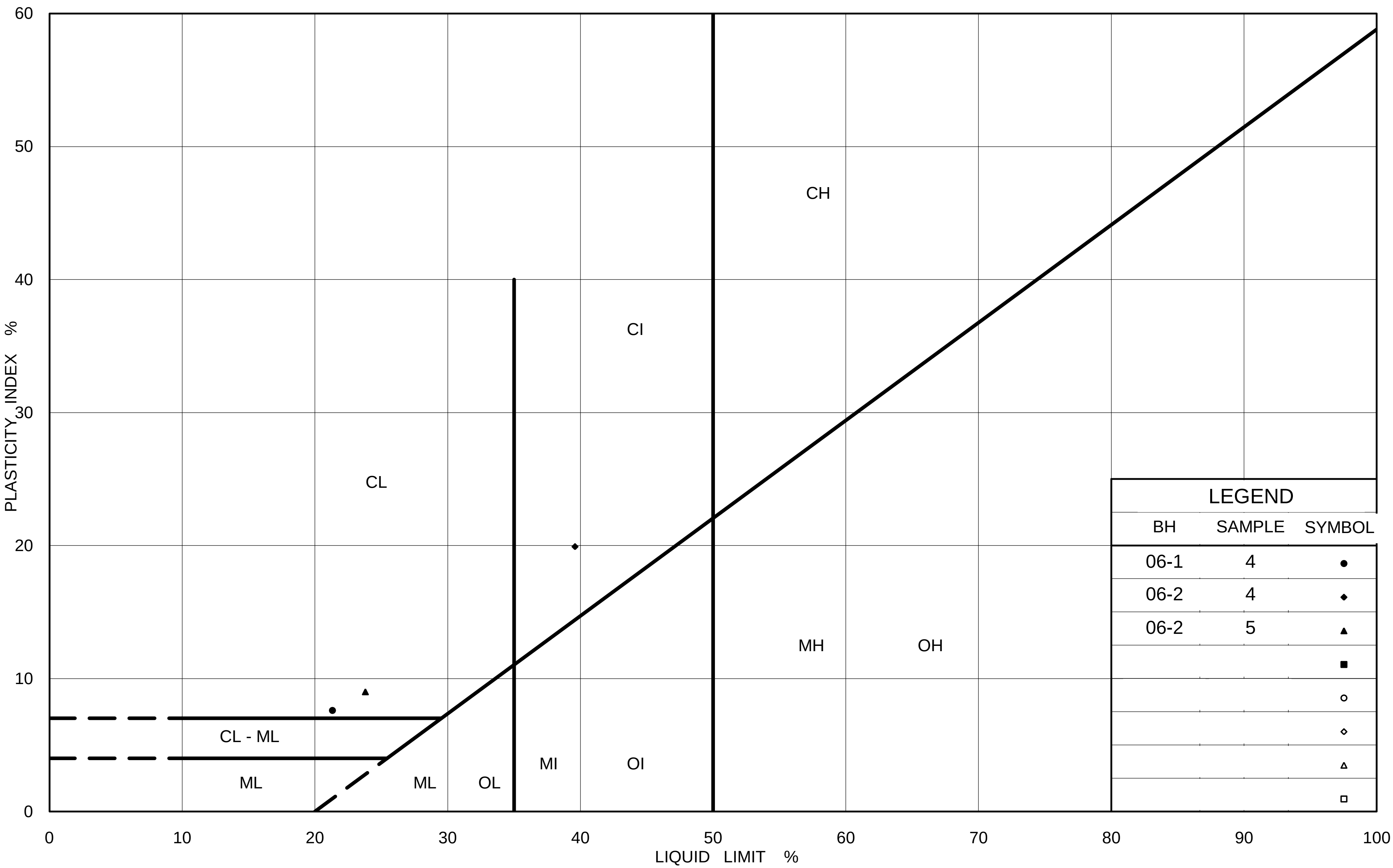
MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: July 27, 2007
FILENAME: T:\Projects\2004\04-1111-024 (NCE, Peterborough)\~CB\~Rpt C (Culvert Str 26-215)\04111024C8001.dwg

APPENDIX A

LABORATORY TEST DATA

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
06-1	4	●
06-2	4	◆
06-2	5	▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Silty Clay

FIG No. A1

Project No. 04-1111-024C

Checked By:

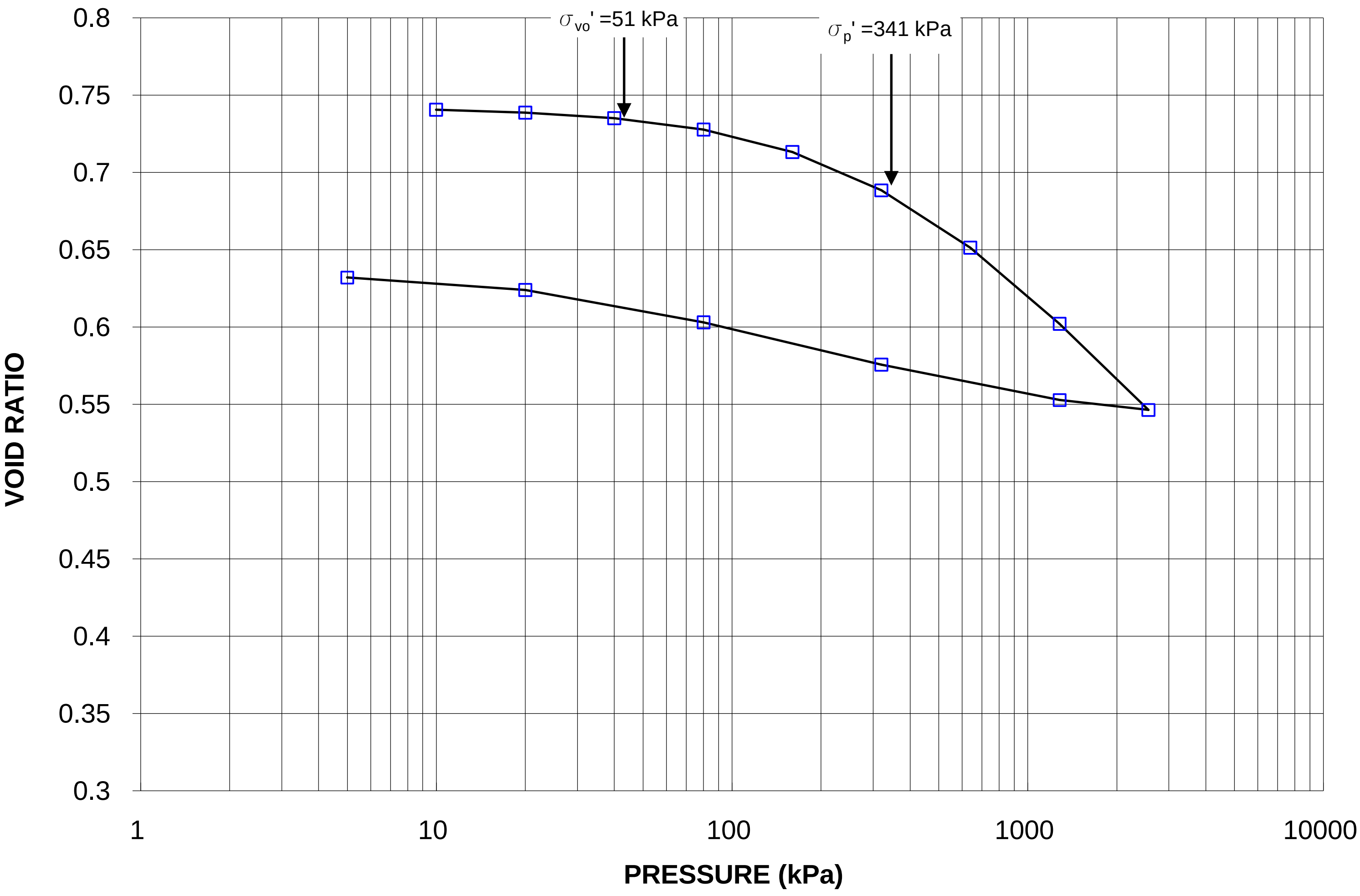
Project No. 04-1111-024

Prepared By: LFG

Checked By: MM

Golder Associates

CONSOLIDATION TEST
VOID RATIO vs LOG PRESSURE
BH 06-2, SA 4



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A2

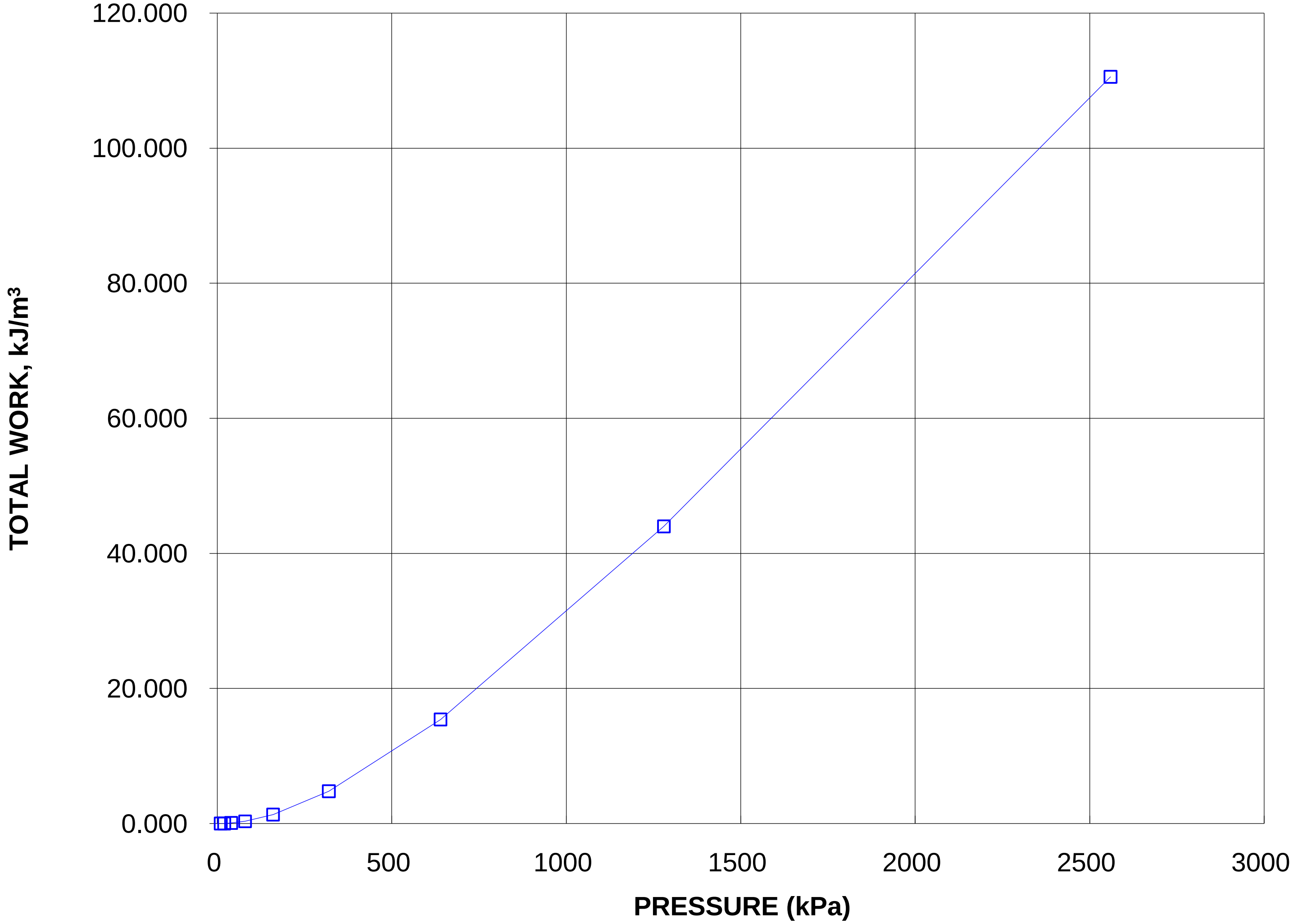
Project No. 04-1111-024

Prepared By: LFG

Golder Associates

Checked By: MM

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



CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE A3

OEDOMETER CONSOLIDATION SUMMARY	FIGURE A4

SAMPLE IDENTIFICATION

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

TEST CONDITIONS

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

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

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

TEST COMPUTATIONS

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

Note:
Specimen swelled under 4.99kPa
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

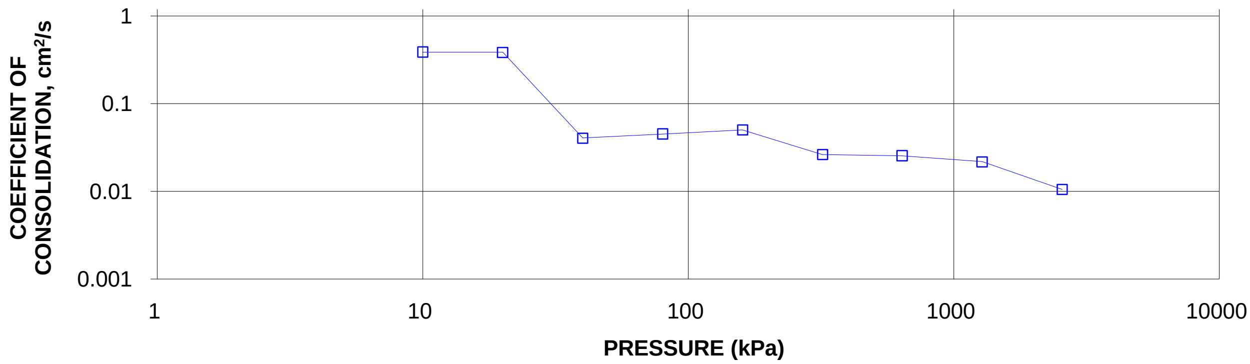
Sample Height, cm	1.79	Unit Weight, kN/m^3	20.57
Sample Diameter, cm	4.42	Dry Unit Weight, kN/m^3	16.58
Area, cm^2	15.35	Specific Gravity, measured	2.76
Volume, cm^3	27.51	Solids Height, cm	1.098
Water Content, %	24.01	Volume of Solids, cm^3	16.86
Wet Mass, g	57.69	Volume of Voids, cm^3	10.65
Dry Mass,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

Prepared By: LFG **Golder Associates** Checked By: MM

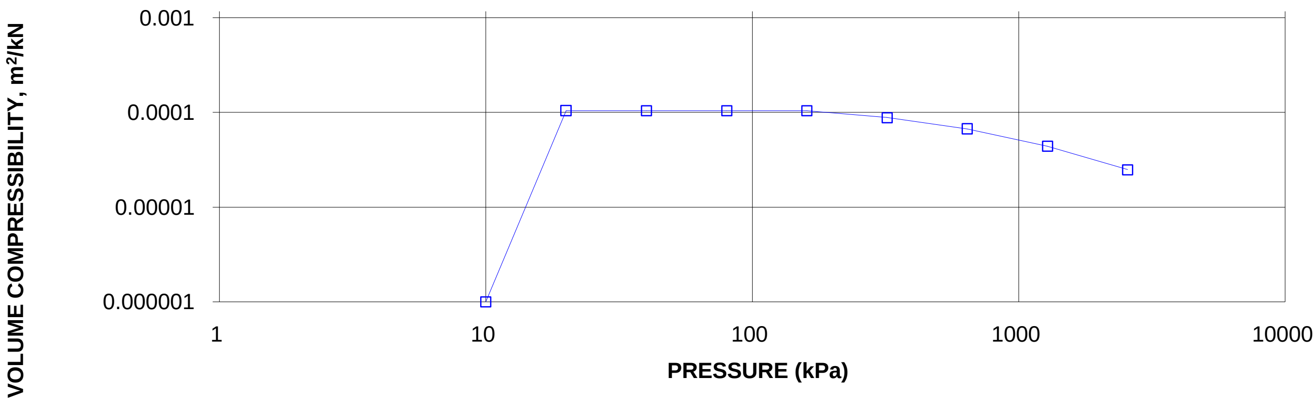
PLOTS OF CONSOLIDATION TEST RESULTS
Cv, Mv, and K VS. PRESSURE

FIGURE A5

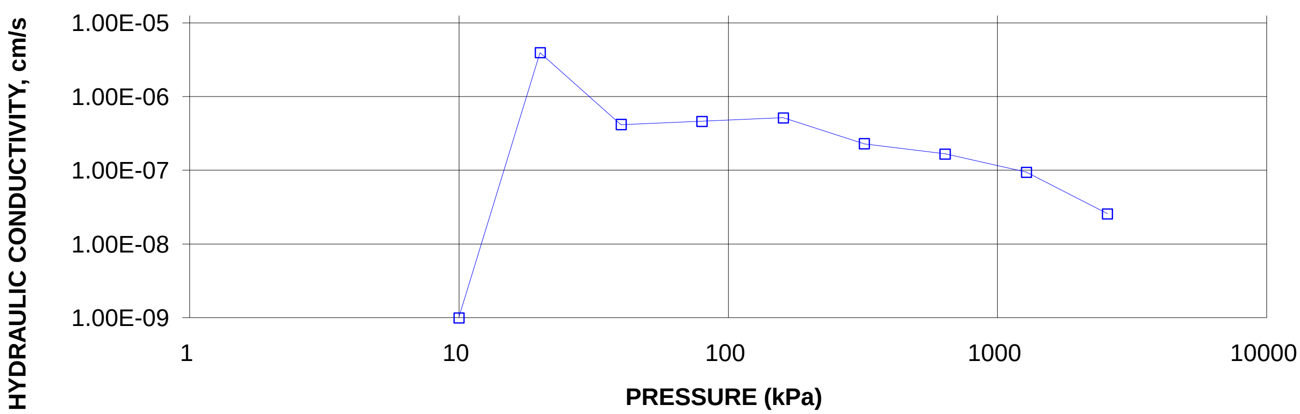
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH 06-2 SA 4



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 06-2 SA 4



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 06-2 SA 4



Project No. 04-1111-024
Prepared By: LFG

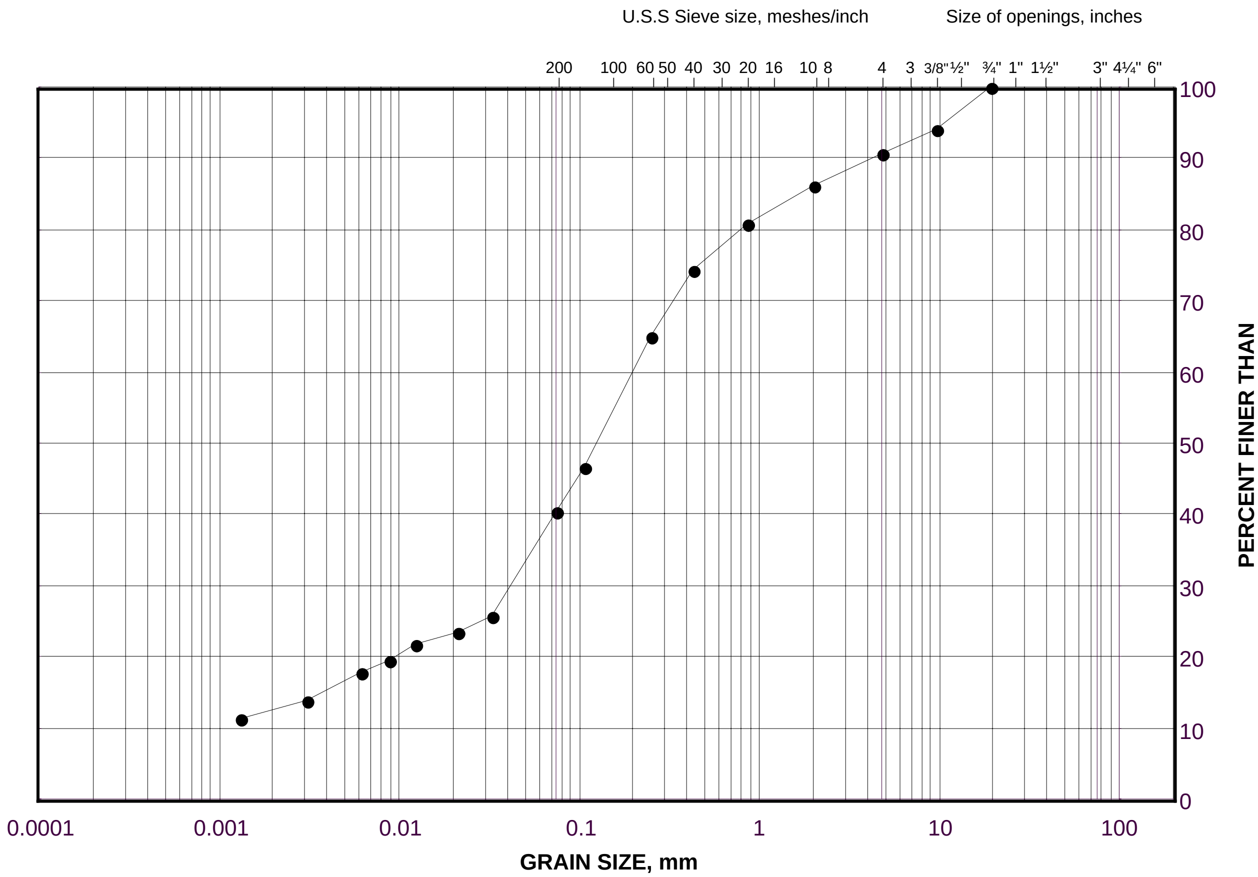
Golder Associates

Checkey By: MM

GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE A6



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

LEGEND

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

Project Number: 04-1111-024C

Checked By: ____________________

Golder Associates

Date: 02-Feb-07