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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN PRELIMINARY DESIGN JACKSON CREEK BRIDGE REPLACEMENT HIGHWAY 7 FROM FOWLERS CORNERS SOUTHERLY TO COUNTY ROAD 28 PETERBOROUGH, ONTARIO G.W.P. 73-99-00, SITE NO. 26-055

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June 2007

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PART A

**FOUNDATION INVESTIGATION REPORT
PRELIMINARY DESIGN
JACKSON CREEK BRIDGE REPLACEMENT
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 28
PETERBOROUGH, ONTARIO
G.W.P. 73-99-00, SITE NO. 26-055**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by National Capital Engineering (NCE) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out preliminary foundation investigations associated with the four-laning of Highway 7 from Fowlers Corner to County Road 28 in Peterborough, Ontario.

Foundation investigation services are required for all foundation engineering elements for the Preliminary Design Study for Highway 7, specifically Jackson Creek Bridge replacement and widening, CNR Overhead, two structural culverts, high fills and swamp areas.

This report addresses the preliminary foundation investigation carried out for the replacement of the Jackson Creek Bridge structure (MTO Structure Site No. 26-055).

The terms of reference for the foundation investigation are outlined in MTO's Request for Proposal for Agreement No. 4005-A-000268, issued in July 2002, and in Golder's proposal (dated August 2003) which is documented under Section 4.8 in NCE's *Technical and Management Proposal for the Highway 7 widening*.

The preliminary investigation was supplemented with information from the following previous report, drawings, and/or investigations:

- Report titled "Foundation Investigation for a Bridge over the Jackson Creek, Highway 133, Near Peterborough, Ontario", Racey, MacCallum and Associates Limited, dated April 12, 1957. Geocres No. 31D-51.
- Foundation Plan (Dwg. No. 458-55-3-A), Site Plan and General Arrangement (Dwg. No. 458-55-1-A) Drawings entitled "Bridge over Jackson Creek", W.P. No. 99-55, dated March 1957.
- Conceptual General Arrangement 4-Lane Replacement Drawing entitled "Bridge over Jackson Creek, Site No. 26-55", Revised date February 22, 2006.
- Drawings titled "Plate 3 and 4, Sta 11+000 to Sta 11+900, Plan and Profile – Contract 3" and preliminary cross-sections provided by NCE on April 5, 2007.

2.0 SITE DESCRIPTION

The Jackson Creek Bridge structure is located approximately 300 m south of the existing intersection of Highway 7 and Sherbrook Road (County Road 9), in Peterborough, Ontario. The existing structure extends from about station 11+655 to 11+685 and consists of a single rigid frame presumed to be supported on timber piles as described on the 1957 General Arrangement drawings.

In general, the site consists of flat terrain transected by the Jackson Creek whose valley is about 13 m wide with a creek bed located about 1 m below the adjacent terrain. The existing Highway 7 roadway embankment is about 2.5 m to 3m high in the area of the approaches to the existing bridge. The natural ground surface in the immediate vicinity of the structure site varies from about Elevation 245 m to 246 m. It is understood from the available General Arrangement drawings that the existing bridge deck is at about Elevation 248.6 m and the approach embankments have 2 horizontal to 1 vertical sideslopes.

The existing embankments are generally grass covered and showed no visible signs of excessive erosion or distress at the time of our investigation. Jackson Creek flows from west to east and the water level is generally at about Elevation 245 m, based on current and previous investigations.

It is our understanding that the existing structure is to be replaced with a wider structure to the west to accommodate four lanes of traffic and associated service shoulders. Based on discussions with the designer, the construction will be staged in order to maintain continuous traffic flow along Highway 7. The first stage will involve constructing the west half of the new structure while maintaining traffic flow on the existing Highway 7 structure. Following completion of the western half of the new structure and embankment widening, traffic will be diverted to the new structure and embankment removal and replacement of the existing structure will be performed.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the proposed Jackson Creek Bridge replacement structure in July 2006, at which time two boreholes (06-3 and 06-4) were advanced at the site using portable manual drilling equipment due to access restrictions, supplied and operated by Eastern Soil Investigation Limited of Courtice, Ontario.

The boreholes were advanced to depths of 5.2 m and 7.1 m below existing ground surface. Continuous samples of the overburden were obtained using 50 mm outside diameter split-spoon samplers driven by a manual half-weight hammer otherwise in accordance with the Standard Penetration Test (SPT) procedure. The SPT 'N' values have been adjusted on the borehole logs to reflect the values that would be obtained from a standard-weight hammer. The water level in the open boreholes was observed throughout the drilling operations. The field work was observed on a full-time basis by a member of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and testing. The boreholes and annulus surrounding the piezometer pipe were backfilled with bentonite in accordance with Ontario Reg. 903. The piezometers will require decommissioning prior to or during construction in accordance with Ontario Reg.903. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing such as water content, Atterberg limits tests and grain size distribution were carried out on selected samples of the overburden soils. A second borehole was advanced next to borehole 06-4 on August 18, 2006 to obtain a Shelby tube sample for laboratory consolidation (oedometer) testing.

The borehole locations were identified in the field by Golder relative to site features, and the ground surface elevations at the borehole locations were surveyed by Transenco Limited, who was subcontracted by NCE. The borehole locations (including MTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to geodetic datum) from the recent investigation are summarized below.

<i>Borehole Number</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
06-3	4904664.9	391236.0	245.7
06-4	4904677.8	391268.5	245.6

The locations and ground surface elevations of all of the boreholes used for assessment of the subsurface conditions at the site (i.e. recent 2006 and previous 1957 investigations) are shown and tabulated on Drawing 1. The northings and eastings for the 1957 boreholes are considered approximate only.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

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, ranging in thickness from about 0.5 m to 2.5 m. “Wave-washed” drumlins, with exposed bouldery surfaces, are also present near the Simcoe Lowlands immediately south and east of Lake Simcoe. Localized deposits of silt, clay and peat are found in the low-lying areas between drumlins.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, two boreholes were advanced within the limits of the proposed Jackson Creek bridge replacement structure. This information was combined with the results from four boreholes advanced as part of the previous 1957 investigation. The approximate borehole locations, ground surface elevations and an interpreted stratigraphic profile along Highway 7 at Jackson Creek are shown on Drawing 1, and interpreted stratigraphic cross-sections along/across Highway 7 at the Jackson Creek Bridge Structure abutments are shown on Drawing 2.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are shown on the Record of Borehole sheets and Figures A1 to A6 in Appendix A following the text of the report. The Record of Borehole sheets from the 1957 investigation are included in Appendix B. The stratigraphic boundaries shown on the borehole records, and on the stratigraphic profile and cross sections on Drawings 1 and 2, are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

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.

4.2.1 Fill

Clayey silt fill material, containing trace amounts of sand, gravel and organics, was encountered at ground surface in borehole 06-4, located in the southeast quadrant of the creek crossing near the toe of the existing south approach embankment. The cohesive fill extends to a depth of about 0.8 m below ground surface. A single Standard Penetration Test (SPT) “N” value measured on the fill was 2 blows per 0.3 m of penetration, suggesting a very soft to soft consistency. A single water content measured on the sample of fill was 27 percent.

4.2.2 Peat

A 1.5 m thick deposit of fibrous peat was present beneath the cohesive fill material in borehole 06-4. Trace amounts of peat were also encountered near surface at the location of borehole 06-3. Two measured SPT “N” values obtained through the peat were 2 and 4 blows per 0.3 m of penetration indicating a very soft to soft consistency. Two water contents measured on the samples of the peat were 60 and 68 percent, suggesting the presence of organics.

4.2.3 Clayey Silt to Silty Clay

A deposit of cohesive soil was encountered from ground surface in borehole 06-3, 57-1 to 57-4 and beneath the peat at the location of borehole 06-4. This deposit varied from a clayey silt to silty clay containing trace to some sand and gravel. Trace amounts of organics and peat were noted either near surface or immediately below the existing peat deposit and the lower portion of the clayey silt strata contained interlayers of sand and gravel. The clayey deposit varies in thickness from about 1.5 m to 4.6 m, and extends to depths of about 1.5 m to 6.9 m (below ground surface at the time of drilling) at the borehole locations.

Atterberg limits testing was carried out on three samples of the cohesive soils from the recent investigation and four samples from the previous investigation, and measured plastic limits of 10 to 19 per cent, liquid limits of 14 to 48 per cent, and corresponding plasticity indices of 4 to 31 per cent. These results, which are plotted on a plasticity chart on Figure A1, indicate that this soil ranges from a silty clay of intermediate plasticity (at shallower depths) to a clayey silt of low plasticity (at deeper depths). Four water contents measured on samples of the clayey soils from the recent investigation and four samples from the previous investigation varied from about 11 to 44 percent; the higher water contents generally measured on samples containing greater clay content.

The measured SPT “N” values in the clayey deposits typically ranged from 1 to 18 blows per 0.3 m of penetration, as well as a single ‘N’ value of 31 blows per 0.3 m of penetration directly above the underlying till, indicating a very soft to very stiff material.

A laboratory consolidation test was conducted on a single sample of this cohesive material from borehole 06-4, and a preconsolidation pressure of approximately 165 kPa is estimated from the test results. The consolidation test results are shown on Figures A2 to A5 and summarized below.

Borehole/ Sample No.	Sample Depth/Elev.	Unit Wt. (kN/m ³)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
06-4 / 4B	3.2 m/ 242.4 m	19.6	33	165	132	5	0.307	0.036	0.776	2.0×10^{-2}

NOTES: * For stress range of $40 \leq \sigma_v' \leq 100$ kPa

where: σ_{vo}' is the estimated current effective overburden pressure in kPa
 σ_p' is the estimated preconsolidation pressure in kPa
OCR is overconsolidation ratio
 e_o is initial void ratio
 C_c is the compression index (based on void ratio)
 C_r is the recompression index
 c_v is the coefficient of consolidation in cm²/s

4.2.4 Silty Sand to Sandy Silt (Till)

An interlayer of silty sand containing some gravel and trace clay was encountered in borehole 06-3, within the clayey silt deposit. The results of grain size distribution tests conducted on a sample of the silty sand soils and on a sample of the silty sand till from the recent investigation are shown on Figures A6 and A7, respectively. A single water content measured on a sample of the silty sand was 10 percent. A single SPT “N” value measured within the thin interbedded layer of silty sand was 4 blows per 0.3 m of penetration, indicating a loose relative density.

4.2.5 Silty Sand to Sandy Silt (Till)

A cohesionless till deposit of silty sand to sandy silt containing trace clay, gravel and clayey silt pockets was encountered beneath the clayey silt deposit in each of the boreholes. All of the boreholes were terminated within the silty sand to sandy silt till.

A single grain size distribution test conducted on a sample of the silty sand till from the recent investigation is shown on Figure A7. Two water contents measured on the samples of the silty sand to sandy silt till were 9 and 14 percent. Measured SPT “N” values within the silty sand to

sandy silt till range from 20 blows per 0.3 m of penetration to greater than 100 blows per 0.3 m of penetration. Based on these 'N' values, the lower till deposit has a compact to very dense relative density.

4.3 Groundwater Conditions

Water levels were measured upon completion of the recent drilling in the open boreholes and varied from about 0.3 m to 0.8 m (Elevation 245.3 m and 244.9 m) below ground surface. A surface elevation of the water level in Jackson Creek was not obtained during the current investigation. The 1957 General Arrangement drawings show a creek water level elevation of 245.2 m, which is consistent with the water levels encountered in the recent boreholes (06-3 and 06-4). The lower portions of cohesionless fill materials and surficial sandy soils, where present, should be expected to be water-bearing during wet periods of the year, with water "perched" above the underlying cohesive soil deposits.

It should be noted that groundwater levels at the site are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Shannon Palmer, EIT., a geotechnical engineer and Paul Dittrich, P.Eng, an associate with Golder. Fintan Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

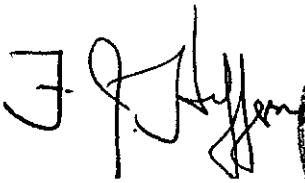
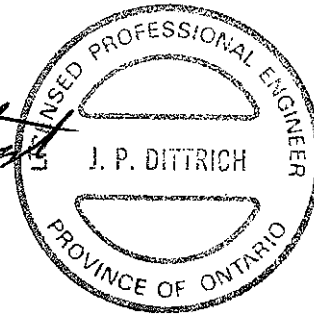
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PART B

**FOUNDATION DESIGN REPORT
PRELIMINARY DESIGN
JACKSON CREEK BRIDGE REPLACEMENT
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 28
PETERBOROUGH, ONTARIO
G.W.P. 73-99-00, SITE NO. 26-055**

6.0 PRELIMINARY ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the preliminary design of the proposed Jackson Creek bridge replacement and widening. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the recent preliminary and the 1957 subsurface investigations at the site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out preliminary design of the proposed structure 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 investigation design stage of the project and as the contract documents are prepared.

Further borehole drilling will be required during the detail design phase of the project, when the structure configuration and the locations to depth of the foundation elements are finalized. At that time, further investigation of the clayey silt to silty clay strata will be required to also confirm the magnitude and time rate of settlement under the embankment loading, and to further assess the settlement mitigation options and develop the necessary operational constraints and/or special provisions for the contract.

6.2 Bridge Foundation Options

It is our understanding that the Jackson Creek Bridge structure is to be replaced and widened to accommodate four lanes of traffic and associated service lanes. Based on preliminary plans provided by NCE, it is understood that Highway 7 is to be widened on the west side of the existing bridge structure. The natural ground surface in the immediate vicinity of the structure site varies from about Elevation 245 m to 246 m. Based on the original General Arrangement drawings for the existing bridge, dated March 1957, the existing Jackson creek structure has an approximate deck elevation of about 248.7 m and 2 horizontal to 1 vertical approach embankment slopes. From information recently provided by NCE to Golder on April 5, 2007 the proposed Jackson Creek bridge has been designed with a deck surface elevation of 249.8 m, resulting in about a 1 m grade raise. The existing bridge was to be supported on timber piles driven to refusal, at about an assumed elevation of 240 m. The underside of the pile cap was to be founded at an elevation of 243.2 m. The original 1957 General Arrangement drawings identified elevations for the creek bed and water level at about 244 m and 245.2 m, respectively. This 1957 water level is consistent with the current water levels encountered during recent drilling program. Further, the Conceptual General Arrangement drawing (rev. date of Feb. 22, 2006) provided by NCE for preliminary purpose shows a high water level of about 246.6 m.

The very soft to firm upper portions of the clayey silt to silty clay strata are not suitable for support of shallow foundations, and so it is recommended that the foundations are extended below this deposit, either by founding on drilled circular caisson footings on the underlying very dense silty sand till, or driving steel H-piles into the very dense silty sand till deposit at depth. Driven steel H-piles are suitable for support of the abutments (in either a conventional or integral abutment configuration) and may be more practical and cost-effective than caissons, due to the presence of the water-bearing silty sand and till layers which would necessitate the use of temporary liners for caisson construction. It is noted that a deeper borehole investigation will be required during the detailed design stage in order to confirm the tip depths for deep foundations, and to possibly provide higher geotechnical resistances.

As noted above, the loading imposed by the new wider and higher approach embankments is expected to cause post-construction consolidation settlement of the upper very soft to firm clayey soils and overlying clayey silt embankment fill, which will in turn impart downdrag loads on the caissons/piles and affect the performance of the abutment foundations. Given this, it is recommended that the widened approach embankments for the Jackson Creek structure be constructed in advance, and monitored over time to minimize post-construction consolidation settlement, improve the long-term performance of the approach embankments and minimize the downdrag loads on the piles. In this regard, the use of preloading/surcharging of the approach embankment widening is one feasible settlement mitigation measure, if the construction schedule can accommodate the time required.

Recommendations for preliminary design of steel H-pile and caisson foundations are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs and risks/consequences associated with each of the feasible foundation options is presented in Table 1 following the text of this report. From a foundations perspective, based on this comparison, it is considered at this preliminary stage that driven steel H-piles are the most practical and a technically preferred foundation solution for this site. As noted above, it is recommended that the approach embankments be preloaded and surcharged for a period of approximately one to two months to minimize post-construction consolidation settlement, minimize downdrag loads on the piles, and improve the long-term performance of the approach embankments and RSS walls (if adopted).

6.3 Steel H-Pile Foundations

The abutments may be supported on steel H-pile foundations driven to found within the very dense silty sand to sandy silt till deposit. The very dense till was encountered in the boreholes between approximately Elevations 238 m and 241 m, generally deeper toward the east. It is anticipated for preliminary design that the abutment pile tip elevations will vary from about 237 to 239 m.; however, the depth of the driven piles may vary depending on the presence of cobbles and boulders and the potential for the piles to “hang up” in this material. Therefore, for analysis purpose, it is assumed that the piles will have a length of about 8 m to 10 m and the abutment pile

caps will be perched within the approach embankments. The pile caps should have a minimum of 1.5 m of soil cover (or equivalent) to provide adequate protection against frost action.

For preliminary design, for HP 310x110 steel H-piles driven to the design pile tip elevation having some embedment within the very dense sandy till stratum, a factored axial resistance at Ultimate Limit States (ULS) of 1400 kN may be assumed. The axial geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 1100 kN.

The pile capacity must be verified in the field by the use of the Hiley formula (Standard Structural Drawing SS-103-11) during final stages of the driving to achieve an ultimate capacity equal to two times the design ULS value. The pile termination or set criteria for the pile capacity selected will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. The criteria must therefore be established at the time of construction after piling equipment is known.

As noted in Section 6.6, the loading from the new fills for the widened approach embankments will result in post-construction consolidation settlement of the existing clayey fills, and upper soft to form portions of the silty clay to clayey silt deposit. This settlement will result in negative skin friction and downdrag loads on the abutment piles. For preliminary design, an unfactored downdrag load of 150 kN acting on each HP 310x110 pile will need to be taken into account in the design of the abutment foundations; the structural capacity of the piles must be checked for the factored dead and downdrag loads, in accordance with Section 6.8.4 of the *CHBDC*. Consideration could be given to constructing the approach embankments approximately one to two months in advance of the bridge construction to allow the majority of the consolidation settlement to occur, and thereby minimize the downdrag loads on the piles.

It is generally recommended that the H-pile tips be stiffened when driving piles into soils which contain cobbles and boulders such as those found at this site. The piles should be stiffened with driving shoes (i.e. MTO flange plates in accordance with OPSD 3000.100 and OPSS 903.07.05.04, Titus “Standard” design, or equivalent). Pile installation and driving shoes should be in accordance with SP903S01.

6.4 Caissons

Consideration could also be given to the use of cast-in-place concrete caissons founded within the very dense till for support of the new Jackson Creek bridge abutments. Approximate preliminary design base elevations ranging from about 237 m to 239 m may be used at the abutments, assuming an approximate embedment of 2 m within the very dense till.

The caissons will derive the majority of their capacity from base resistance, although some shaft friction has also been taken into account based on “socketting” approximately 2 m into the very

dense till. Using the design elevations given above, and assuming that all caisson excavations are cleaned prior to pouring concrete, the factored axial geotechnical resistance at ULS and the axial resistance at SLS for 25 mm of settlement are given below for various caisson diameters. The estimated unfactored downdrag load acting on the caissons at the east and west abutments may be taken as shown below.

<i>Caisson Diameter</i>	<i>Axial Geotechnical Resistance</i>		<i>Unfactored Downdrag Load</i>
	<i>ULS</i>	<i>SLS</i>	
0.9 m	3,250 kN	2,500 kN	400 kN
1.2 m	5,500 kN	4,500 kN	525 kN
1.5 m	8,500 kN	6,500 kN	650 kN

Given that caissons are socketed at least 2 m into the very dense till, an axial geotechnical resistance of 5,000 kPa at ULS and 3,500 kPa at SLS can be used for preliminary design.

The performance of caissons will depend to a large degree upon the final cleaning and verification of the condition of the subgrade soils at the base. The base of each caisson excavation must be cleaned to remove all loose cuttings to ensure that the concrete is in intimate contact with the competent very dense till/native bearing stratum. Due to the anticipated water inflow, it may not be possible to inspect the base of each caisson; airlifting may have to be employed and the concrete will have to be placed using tremie techniques. A temporary liner will be required to support the sides of the caisson excavations during cleaning and concrete placement; the discharge pipe must be maintained below the surface of the concrete as the liner is withdrawn. The sandy soils and lower till should be expected to contain cobbles and boulders which may pose difficulties in advancing caissons / temporary liners. It is also noted that basal heave could occur where more pervious sand and silt till soils are present at/near the caisson base.

Caisson installation should be in accordance with SP903S01. An Non-Standard Special Provision (NSSP) may be required to address the need for control of groundwater during caisson construction.

6.5 Frost Protection

The pile caps should be provided with a minimum of 1.5 m of soil cover or equivalent thermal insulation for frost protection.

6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining 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 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 existing and 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 wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and 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 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 (SAND)	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 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/m³)
 • taken as soil unit weights given above for fill materials
 • taken as 19 kN/m³ for the native materials
 d is the depth below the top of the wall (m); and
 H is the height of the wall above the toe (m).

6.7 Approach Embankment Design and Construction

The upgrading of the existing Highway 7 embankments as part of the bridge replacement/widening will require placement of about 3.5 m to 4 m of new fill for the widening of the west side of the north and south approaches and about a 1 m grade raise on the existing Highway 7 embankment.

The foundation soils in the area consist of up to about 1 m of existing fill overlying about 3.4 m to 5.2 m of very soft to very stiff silty clay to clayey silt, overlying a very dense till deposit. A 1.5 m thick layer of peat was encountered below the fill and above the very soft silty clay deposit on the east side of the south approach embankment. Subexcavation of any very soft to firm portions of the fill, clayey silt fill, peat and/or native upper silty clay is recommended where it is present in the areas of proposed embankment widening. Additional drilling will be required at the detail design stage to better define the areas requiring subexcavation and replacement.

Placement of the new fill for embankment widenings directly on any peat and/or very soft clayey materials can result in excessive settlements and side slope instability. In addition, long-term settlements related to decay of the organic materials could also occur. As a result, these soils where present are unsuitable for support of any new embankment fills and should be removed and replaced. Removal and replacement of unsuitable soils may result in significant volumes of material requiring disposal as well as the requirement for imported fill material. The unsuitable foundation soils that are to be removed are also likely to be unsuitable for reuse in embankment construction widening or other structural fill areas on the site. Removal of the unsuitable soils could require subaqueous excavation and placement of backfill, which will prohibit conventional spreading and compaction of materials.

However, based on the limited subsurface information available at this time within the area of the proposed widenings (i.e. borehole 06-3 on the west side of the north approach), it is possible that

the depth of subexcavation and replacement may be limited to a depth of about 1 m (subject to confirmation at detail design).

6.7.1 Approach Embankment Stability

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the 3.5 m to 4 m high approach embankment widenings with side slopes and front slopes (adjacent to Jackson Creek) maintained at 2 horizontal to 1 vertical (2H:1V) are anticipated to have a factor of safety of 1.3 or greater against deep-seated slope instability.

Static slope stability analyses for this embankment configuration have been carried out using the following parameters, estimated from field and laboratory testing and accepted correlations, using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd. Both undrained (i.e. short-term) and effective (i.e. long-term) stress analyses were carried out. Seismic slope stability analyses should be performed at the detailed design stage for the project.

<i>Soil Deposit</i>	<i>Bulk Unit Weight (kN/m³)</i>	<i>Effective Friction Angle (degrees)</i>	<i>Undrained Shear Strength (kPa)</i>
Embankment fill (range of parameters assumed for earth and granular fill)	20 – 22	32 to 35	–
Firm to stiff surficial clayey silt	19 – 20	28 to 30	30 to 40
Loose silty sand	19 – 20	30	–
Soft to stiff clayey silt	19 – 20	30 to 32	40 to 50
Very dense sandy silt (till)	20 – 22	32 to 35	–

6.7.2 Approach Embankment Settlement

Settlement of the proposed embankment widenings at the site will occur due to compression of the new embankment fill itself, as well as consolidation of the underlying soft to stiff clayey silt deposit, and some minor compression of the silty sand to sandy silt till deposit. Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the approximately 4 m high approach embankment widening fill itself is expected to be less than approximately 25 mm. The use of granular fill for the new embankment construction would reduce this magnitude, since the majority of settlement of granular fills will occur during construction.

Settlement analyses of the foundation soils were carried out using hand calculations as well as the commercially available computer program Unisettle. The analyses performed assumes that any peat/organics, surficial soft silty clay/clayey silt and clayey silt fill as encountered during drilling has been removed and replaced with approved backfill prior to construction. The compression of the underlying soft to stiff silty clay to clayey silt was modelled using consolidation settlement

parameters based on the results of the laboratory consolidation test and correlations with the measured Atterberg limits. The compression of the very dense cohesionless soils was modelled using elastic deformation moduli based on correlations with the measured SPT “N” values. The parameters used in the analyses are summarized in the following tables:

West Side of North Approach

<i>Soil</i>	<i>Thickness (m)</i>	<i>Unit Weight (kN/m³)</i>	<i>Deformation Properties</i>
Embankment fill and backfill material (range of parameters assumed for earth fill and granular fill)	4 (high)	20-22	-
Firm to stiff clayey silt to silty clay	1.5 to 4.3	19.5	$m_v = 2.1 \times 10^{-4} \text{ kPa}^{-1}$
Firm to stiff clayey silt	1.0 to 1.5	19.5	$m_v = 4.1 \times 10^{-4} \text{ kPa}^{-1}$
Very dense silty sand to sandy silt till	0.5	21	$E' = 50 \text{ MPa}$

West Side of South Approach

<i>Soil</i>	<i>Thickness (m)</i>	<i>Unit Weight (kN/m³)</i>	<i>Deformation Properties</i>
Embankment fill and backfill material (range of parameters assumed for earth fill and granular fill)	4 (high)	20-22	-
Firm to stiff clayey silt to silty clay	0.8	19.5	$m_v = 2.5 \times 10^{-4} \text{ kPa}^{-1}$
Firm to stiff clayey silt	4.5	19.5	$m_v = 5.3 \times 10^{-4} \text{ kPa}^{-1}$
Very dense silty sand to sandy silt till	0.5	21	$E' = 50 \text{ MPa}$

The following magnitudes of consolidation settlement have been estimated within the firm to stiff silty clay to clayey silt:

- about 50 mm to 60 mm under the 4 m high north approach embankment; and
- about 25 mm to 35 mm under the 4 m high south approach embankment.

The above consolidation settlements have been estimated for the west sides of the north and south approaches at Jackson Creek only. The majority of this consolidation settlement is expected to occur within approximately 2 to 6 weeks of completion of the north and south embankment widening construction, respectively. Settlements of the cohesionless foundation soils are expected to be less than 25 mm and occur during construction/fill placement.

If the construction schedule cannot accommodate a maximum preload schedule of up to six weeks, consideration can be given to surcharging of the foundation soils, backfill and embankment fill to accelerate and minimize post-construction settlements. Based on preliminary analysis, it is estimated that 95 percent of the primary consolidation settlement (within the foundation soils) may be completed in about half the time by incorporating a 1 m thick surcharge. If considered, the feasibility to surcharge should be reviewed further during the detail design.

Further examination of the magnitude and time rate of settlement of the soft to stiff silty clay to clayey silt deposits will be required during the detail design stage, with appropriate provision made for settlement mitigation measures (subexcavation and/or preloading) in the contract.

6.8 Preliminary Construction Considerations

6.8.1 Excavations

Excavations during preparation of the subgrade and construction of the pile caps at the abutment locations will typically extend through the existing embankment fill and underlying very soft to stiff clayey silt to silty clay and/or peat soils. Open-cut excavations into these materials 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. The type and consistency of the existing fill at the abutment locations should be investigated during detailed design; however, assuming the embankment fill free of organics and deleterious material, it is likely that the soil can be classified as a Type 2 or 3 soil, as such, side-slopes can be excavated no steeper than 1H:1V. The underlying submerged peat and very soft to soft silty clay to clayey silt soils are classified as Type 4 soil, as such, temporary excavations should be made with side slopes no steeper than 3H:1V.

It is understood that the proposed replacement of Jackson Creek bridge will be constructed in stages to maintain traffic on Highway 7 during construction and adjacent to the existing Jackson Creek. Therefore, excavation support may be required at the site for temporary roadway protection and/or property and space restrictions (i.e. adjacent to Jackson Creek). Where required, temporary excavation support systems can be used and should be designed and constructed in accordance with MTO's Special Provision 105S19. In general, the lateral movement of the temporary shoring system should meet Performance Level 3 as specified in SP 105S19. If the temporary excavation support system is required for roadway or utility protection, then the temporary shoring system should meet Performance Level 2.

6.8.2 Groundwater and Surface Water Control

Based on the borehole information, the groundwater level is at about Elevation 245 m (about 0.3 m below ground surface). Given the close vicinity to Jackson Creek, it is anticipated that groundwater will be encountered during stripping and subexcavation required for the construction of the bridge replacement structure and approach embankments.

It is anticipated that removal and replacement of the peat and very soft silty clay soils will be performed and will require excavation below the current groundwater levels. As a result, consideration should be given to using a Granular B Type II soil or rock fill for placement and compaction of backfill below the groundwater table and above the clayey silt to silty clay subgrade. Depending on the final design subexcavation depths, ditching and dewatering using filtered sumps and/or pumps may be sufficient for adequate placement and compaction of the backfill materials.

More extensive dewatering measures may be required to allow for construction of the pile caps in the dry. The extent of dewatering efforts will depend on the design founding elevation of the pile caps, as such, the founding elevation should be kept as high as possible. It is likely that sheetpiling or some form of barrier between the excavation and Jackson Creek would be required to form a cut-off to water flow. Alternatively, active dewatering options (e.g. filtered sump pumps for shallow excavations or elaborate dewatering systems such as eductor wells for deeper excavations) may be required and should be discussed further after the detailed design investigation is complete and design founding elevations are known.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Shannon Palmer, EIT., a geotechnical engineer and Paul Dittrich, P.Eng, an associate with Golder. Fintan Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

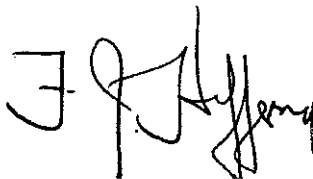
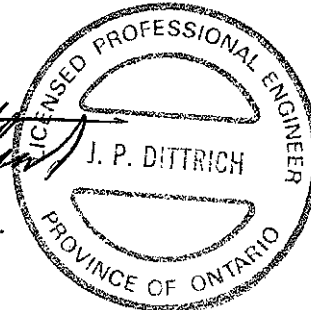
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Designated MTO Contact



SLP/JPD/FJH/al

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TABLE 1
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
JACKSON CREEK STRUCTURE

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Spread footings supported on the soft to stiff clayey soils	<ul style="list-style-type: none"> • Not practical for support of abutments 	<ul style="list-style-type: none"> • Not applicable 	<ul style="list-style-type: none"> • Low geotechnical resistance • Differential settlement across abutments and between foundation elements • Significant subexcavation required • Excavations may extend below water level (particularly where “perched” water conditions are present) 	<ul style="list-style-type: none"> • Not applicable 	<ul style="list-style-type: none"> • Differential settlement depending on the thickness of the underlying clayey soils • Controls for groundwater seepage into excavations may be required resulting in difficulties to maintain a clean, undisturbed foundation surface
Steel H-pile foundations driven to found within very dense silty sand till	<ul style="list-style-type: none"> • Feasible for support of all foundation elements and associated retaining walls (if required) 	<ul style="list-style-type: none"> • Site conditions appropriate for use of integral abutments • Downdrag loads can be mitigated by preloading approach embankment areas, and by subexcavation of very soft to firm clayey soils and peat within approach embankment areas • Minimize differential settlement across abutments and between foundation elements • Readily installed 	<ul style="list-style-type: none"> • Downdrag loads will act on piles due to consolidation of very soft to firm silty clay to clayey silt under embankment loading; to eliminate, schedule would have to accommodate one to two months of preloading time and 0.8 m to 2.3 m subexcavation of peat and very soft to firm clayey soils • Piles may “hang up” on boulders within lower till deposit 	<ul style="list-style-type: none"> • Less expensive than caissons 	<ul style="list-style-type: none"> • Potential for piles to “hang-up” on cobbles, boulders or be deflected away from vertical (i.e. seating problems) during driving
Caissons bored to socket within the lower very dense silty sand till	<ul style="list-style-type: none"> • Feasible for support of abutments 	<ul style="list-style-type: none"> • Minimize differential settlement across abutments and between foundation elements • Higher bearing resistances than for steel H-piles 	<ul style="list-style-type: none"> • Possibility of basal heave; temporary liners required during installation • Difficult installation/socketting with cobbles and boulders within till deposit • Difficult to sufficiently clean base and carry out inspections 	<ul style="list-style-type: none"> • More expensive than steel H-piles 	<ul style="list-style-type: none"> • Difficulties may occur due to the presence of cobbles and boulders • Concrete may need to be placed with tremie techniques

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

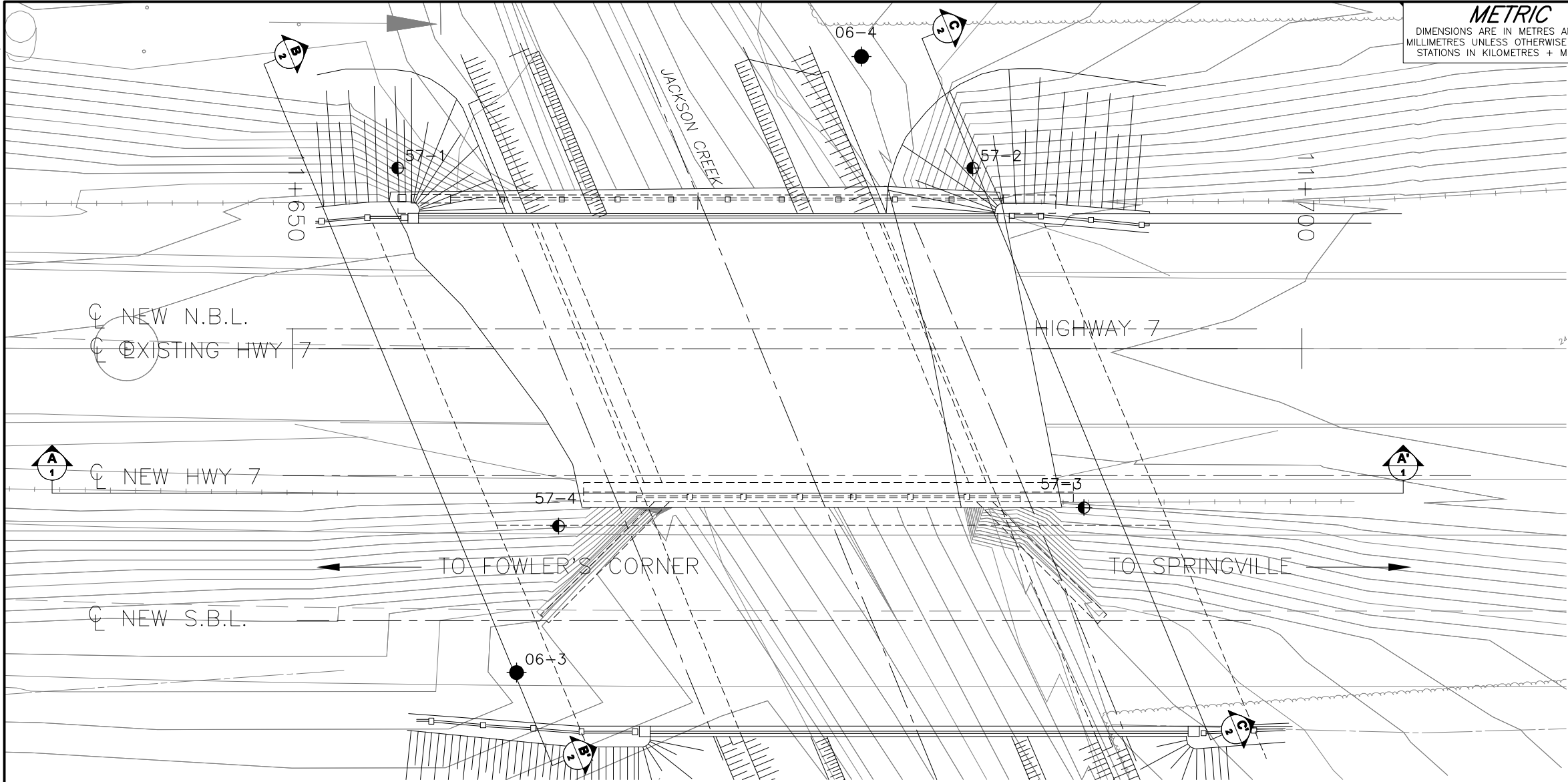
τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
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 04-1111-024A		RECORD OF BOREHOLE No 06-3				1 OF 1 METRIC								
W.P. 73-99-00		LOCATION N 4904664.9; E 391236.0				ORIGINATED BY SB								
DIST _____ HWY 7		BOREHOLE TYPE Portable Manual Equipment, Continuous Split-Spoons				COMPILED BY DD								
DATUM Geodetic		DATE July 31, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
245.7	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
0.0	CLAYEY SILT, trace sand, organics and peat Soft Brown and grey Moist		1	SS	3		245							19 45 27 9
244.9	CLAYEY SILT, trace to some sand Firm to stiff Brown to grey Moist		2	SS	5		244							
243.4	SILTY SAND, some gravel, trace clay Loose Grey Wet		3	SS	12		243							
242.6	CLAYEY SILT, trace sand and gravel Soft to stiff Grey Moist	4	SS	6	242									
241.1		5	SS	4	241									
4.6	SANDY SILT, trace clay and gravel (TILL) Very dense Grey Moist	6	SS	8										
240.5		7	SS	48/0.25										
5.2	End of Borehole	8	SS	50/0.18										
<p>Note:</p> <p>1. Water level measured in open hole at 0.8 m depth upon completion of drilling.</p> <p>2. Borehole advanced using portable drilling equipment with a half-weight hammer. The SPT 'N' Values have been adjusted on this log to reflect the values that would be obtained using a standard weight hammer.</p>														

PROJECT		04-1111-024A		RECORD OF BOREHOLE No 06-4		1 OF 1 METRIC								
W.P.		73-99-00		LOCATION		N 4904677.8 ; E 391268.5								
DIST		HWY 7		BOREHOLE TYPE		Portable Manual Equipment, Continuous Split-Spoons								
DATUM		Geodetic		DATE		July 31, 2006								
				ORIGINATED BY		SB								
				COMPILED BY		DD								
				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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
245.6	GROUND SURFACE							20 40 60 80 100						
0.0	Clayey silt, trace sand, gravel and organics (FILL) Very soft to soft Brown and grey Moist		1	SS	2									
244.8	PEAT, fibrous Very soft to soft Black Wet		2	SS	4									
0.8			3	SS	2									
243.3	SILTY CLAY, trace sand and gravel Very soft Blackish brown to grey Moist to wet		4/4A	SS/PH	1									
2.3			5/4B	SS/PH	1									
241.0			6	SS	1									
4.6	CLAYEY SILT, some sand, trace gravel, contains sand and silt interlayers Soft to hard Grey Wet		7	SS	2									
			8	SS	7									
			9	SS	31									
238.7	SILTY SAND, trace clay and gravel, trace clayey silt pockets (TILL) Very dense Grey Moist End of Borehole		10	SS	50/0.25									4 63 27 6
7.1	Notes: 1. On August 18, 2006 a second borehole was advanced beside borehole 06-4 to a depth of 3.5 m. Two Shelby tube samples were obtained at 2.3 m and 2.9 m depth (Samples 4A and 4B). 2. Water level measured in open borehole at 0.3 m depth upon completion of drilling on August 18, 2006. 3. Borehole advanced using portable drilling equipment with a half-weight hammer. The SPT 'N' Values have been adjusted on this log to reflect the values that would be obtained using a standard weight hammer.													

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 6/12/07 DD



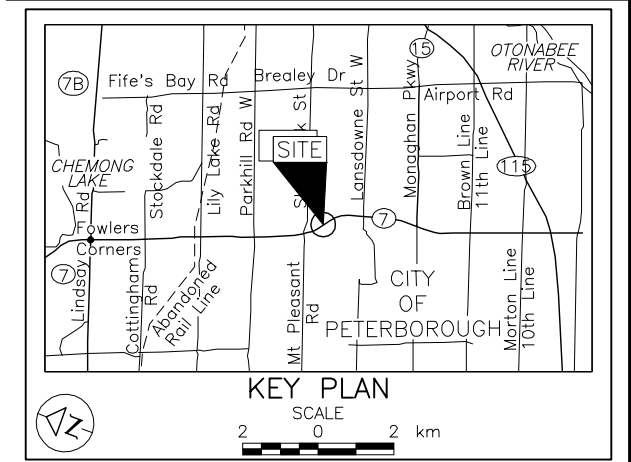
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 73-99-00

HIGHWAY 7 FOUR-LANING
JACKSON CREEK STRUCTURE NO. 26-55
BOREHOLE LOCATIONS
AND SOIL STRATA PROFILE

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

Borehole - Current Investigation (2006)

Borehole - Previous Investigation
Approximate Location (1957)

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
57-1	245.2	4904688.0	391247.2
57-2	245.5	4904670.1	391269.3
57-3	245.6	4904653.6	391263.0
57-4	245.3	4904669.2	391242.2
06-3	245.7	4904664.9	391236.0
06-4	245.6	4904677.8	391268.5

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only. Design details and elevations were not known during the preliminary investigation.

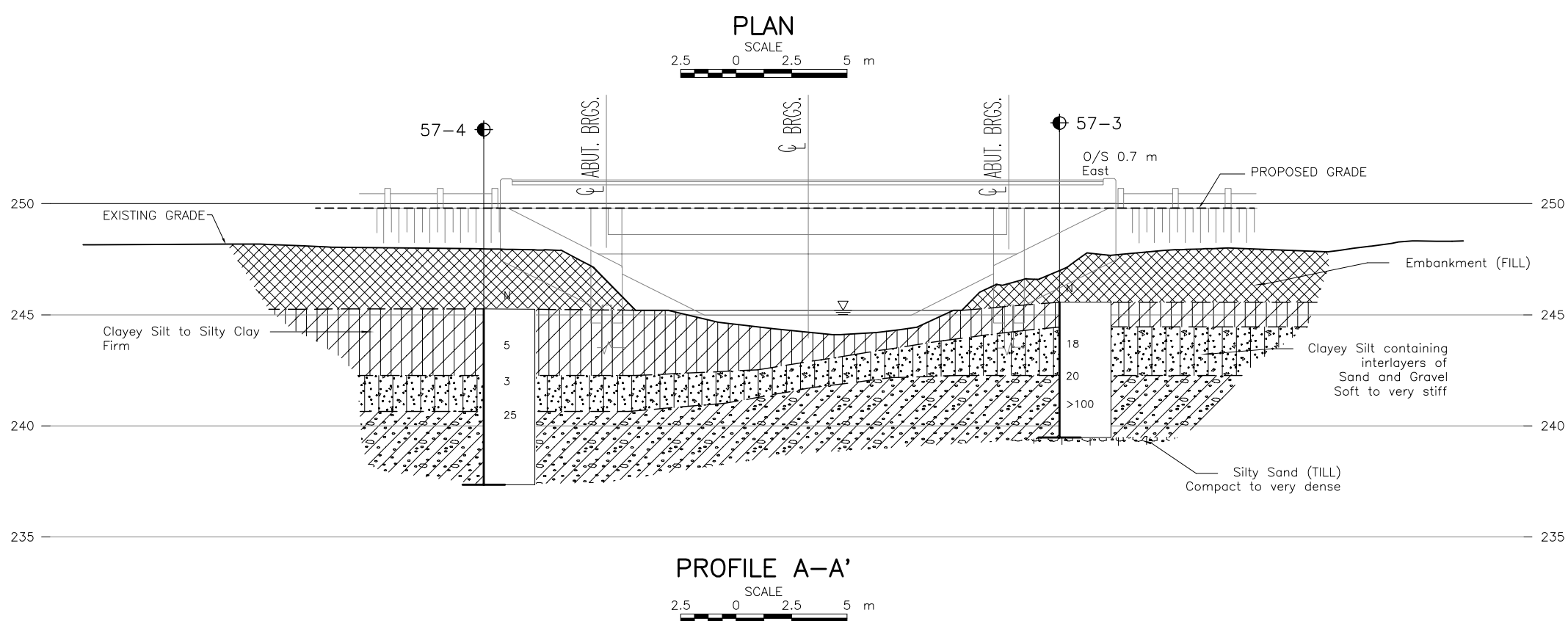
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

REFERENCE

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

NO.	DATE	BY	REVISION
Geocres No. 31D-430			
HWY. 7		PROJECT NO. 04-1111-024A	DIST.
SUBM'D. SLP	CHKD. KJB	DATE: JAN 2007	SITE: 26-055
DRAWN: MSM	CHKD. KJB	APPD.	DWG. 1



PROFILE A-A'

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

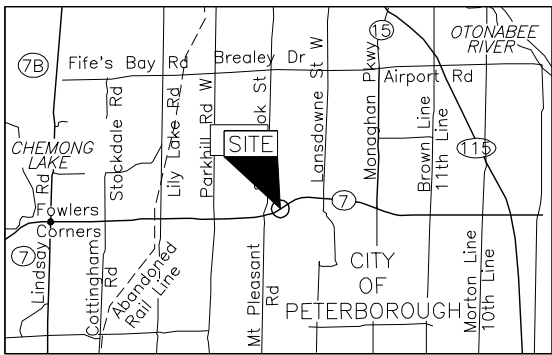
CONT No.
WP No. 73-99-00

HIGHWAY 7 FOUR-LANING
JACKSON CREEK STRUCTURE NO. 26-55
SOIL STRATA CROSS-SECTIONS AT
NORTH AND SOUTH ABUTMENTS

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
0 2 km

LEGEND

- Borehole - Current Investigation (2006)
- ⊕ Borehole - Previous Investigation Approximate Location (1957)

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
57-1	245.2	4904688.0	391247.2
57-2	245.5	4904670.1	391269.3
57-3	245.6	4904653.6	391263.0
57-4	245.3	4904669.2	391242.2
06-3	245.7	4904664.9	391236.0
06-4	245.6	4904677.8	391268.5

NOTES

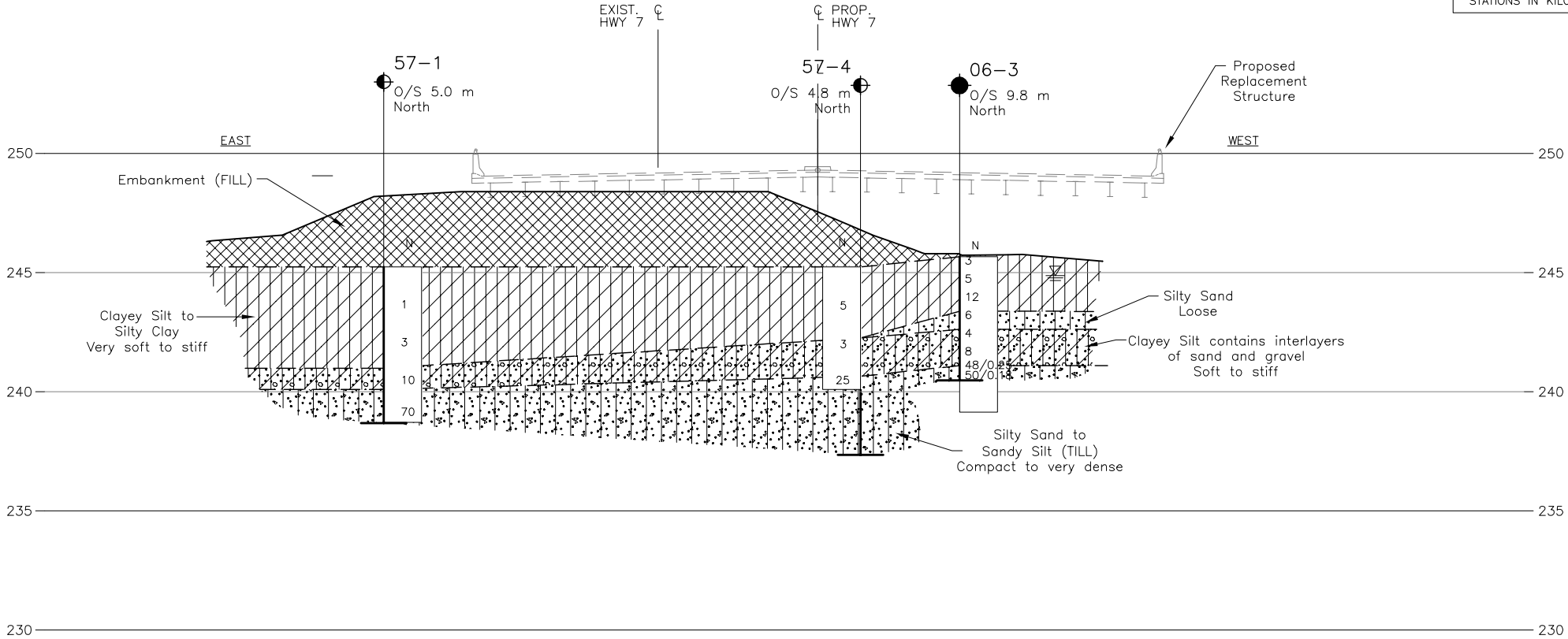
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only. Design details and elevations were not known during the preliminary investigation.

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

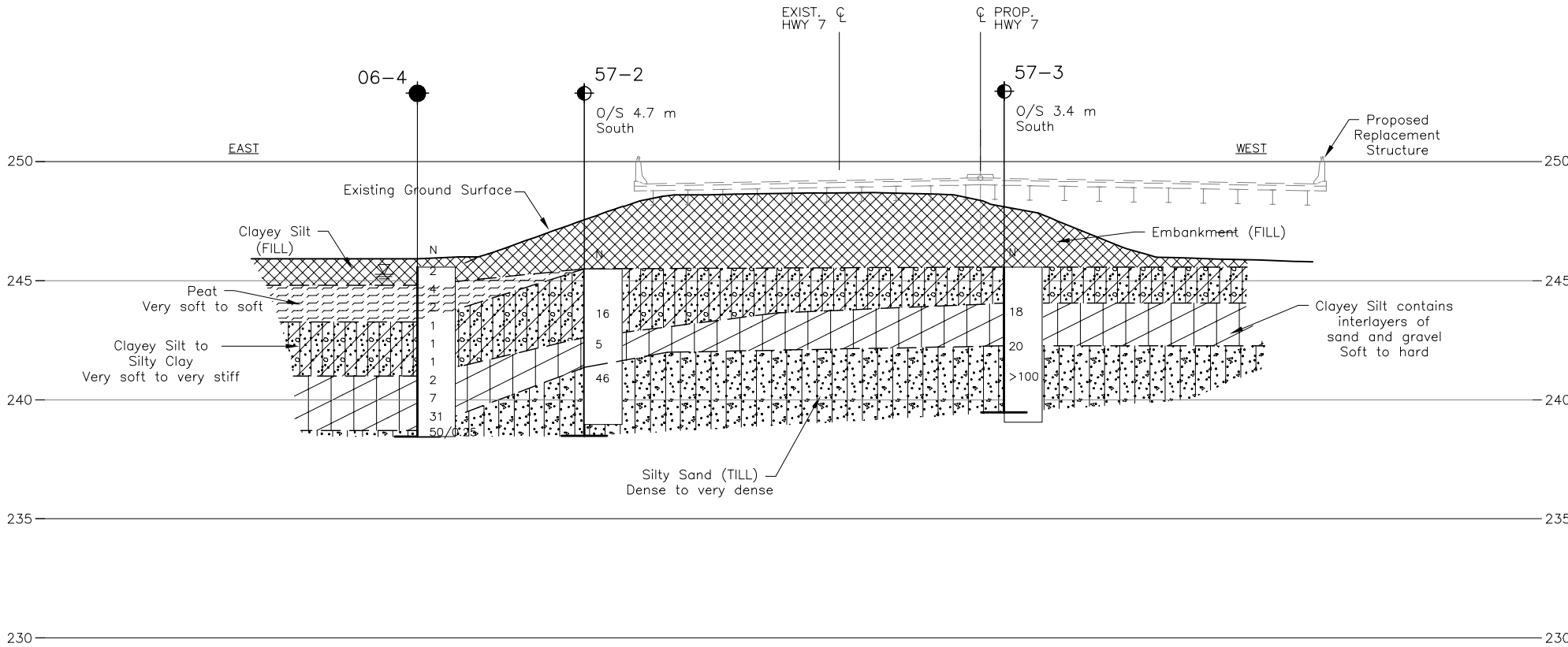
The preliminary foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of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

REFERENCE

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



SECTION B-B'

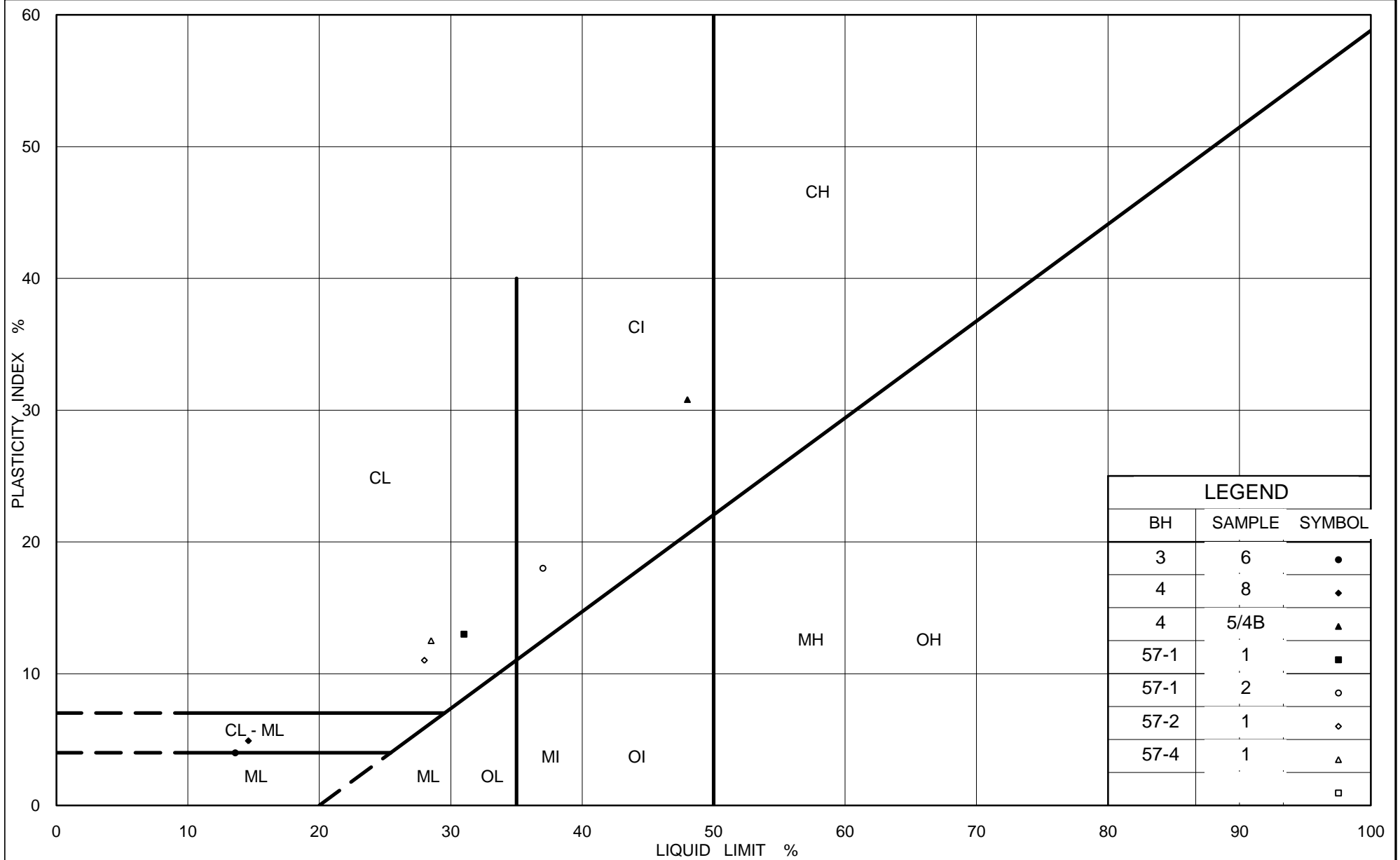


SECTION C-C'

SCALE
2.5 0 2.5 5 m

NO.	DATE	BY	REVISION
Geocres No. 31D-430			
HWY. 7	PROJECT NO. 04-1111-024A		DIST.
SUBM'D. SLP	CHKD. KJB	DATE: JAN 2007	SITE:
DRAWN: MSM	CHKD. KJB	APPD.	DWG. 2

APPENDIX A
LABORATORY TEST DATA



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt to Silty Clay

FIG No. A1

Project No. 04-1111-024A

Checked by: SLP

OEDOMETER CONSOLIDATION SUMMARY

FIGURE A2

SAMPLE IDENTIFICATION

Project Number	04-1111-024A	Sample Number	4B
Borehole Number	4	Sample Depth, m	2.9-3.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	11		
Date Started	08/23/2006		
Date Completed	09/03/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.27	Unit Weight, kN/m ³	19.61
Sample Diameter, cm	4.96	Dry Unit Weight, kN/m ³	15.19
Area, cm ²	19.32	Specific Gravity, measured	2.75
Volume, cm ³	24.54	Solids Height, cm	0.715
Water Content, %	29.16	Volume of Solids, cm ³	13.82
Wet Mass, g	49.08	Volume of Voids, cm ³	10.72
Dry Mass, g	38.00	Degree of Saturation, %	103.4

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	0.776	1.270				
5.01	1.238	0.731	1.254	76	4.39E-03	5.03E-03	2.16E-06
10.00	1.230	0.720	1.234	28	1.15E-02	1.26E-03	1.43E-06
20.00	1.217	0.702	1.224	53	5.99E-03	1.02E-03	6.01E-07
40.00	1.199	0.677	1.208	29	1.07E-02	7.09E-04	7.41E-07
80.06	1.176	0.644	1.188	60	4.98E-03	4.52E-04	2.21E-07
160.06	1.146	0.602	1.161	141	2.03E-03	2.95E-04	5.86E-08
320.00	1.102	0.541	1.124	60	4.46E-03	2.17E-04	9.48E-08
640.00	1.058	0.479	1.080	85	2.91E-03	1.08E-04	3.09E-08
1280.00	1.012	0.415	1.035	76	2.99E-03	5.66E-05	1.66E-08
2560.00	0.970	0.356	0.991	15	1.39E-02	2.58E-05	3.51E-08
1280.00	0.974	0.362	0.972				
320.00	0.987	0.380	0.981				
80.06	1.002	0.401	0.995				
20.00	1.019	0.425	1.011				
5.01	1.040	0.454	1.030				

Note:
k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.04	Unit Weight, kN/m ³	22.05
Sample Diameter, cm	4.96	Dry Unit Weight, kN/m ³	18.54
Area, cm ²	19.32	Specific Gravity, measured	2.75
Volume, cm ³	20.09	Solids Height, cm	0.715
Water Content, %	18.89	Volume of Solids, cm ³	13.82
Wet Mass, g	45.18	Volume of Voids, cm ³	6.28
Dry Mass, g	38		

Prepared By: LFG

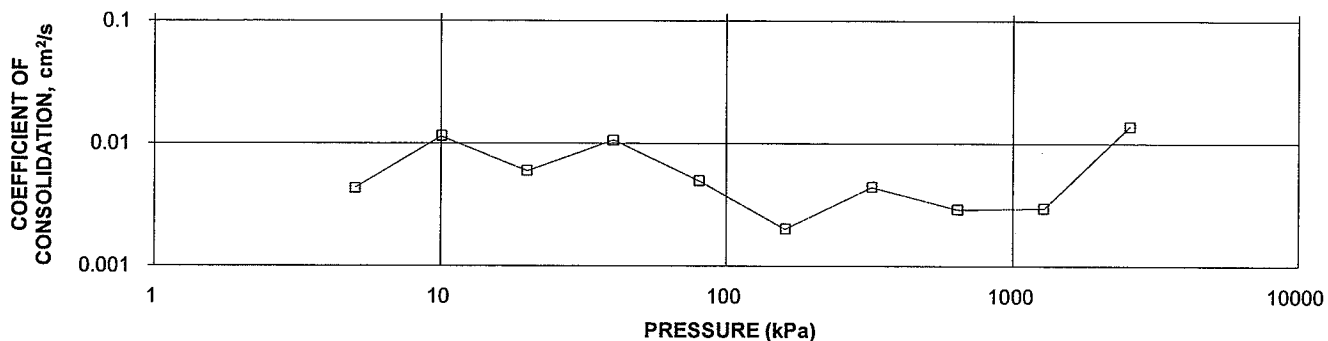
Golder Associates

Checked By: MM

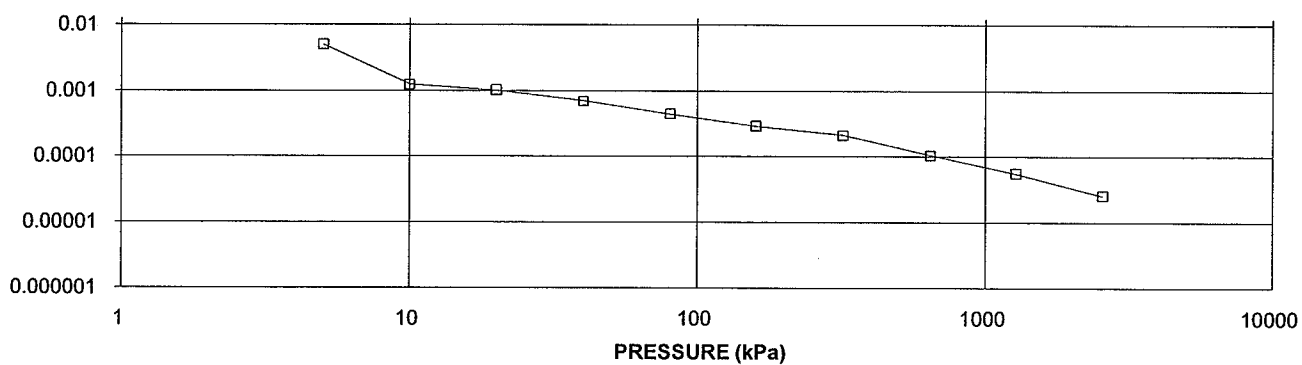
OEDOMETER CONSOLIDATION SUMMARY

FIGURE A3

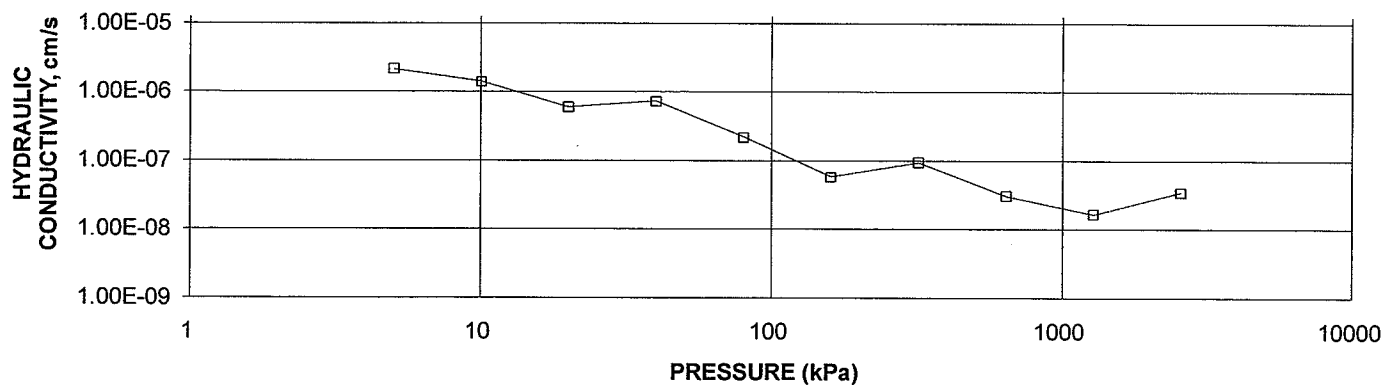
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH 4 SA 4B



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 4 SA 4B



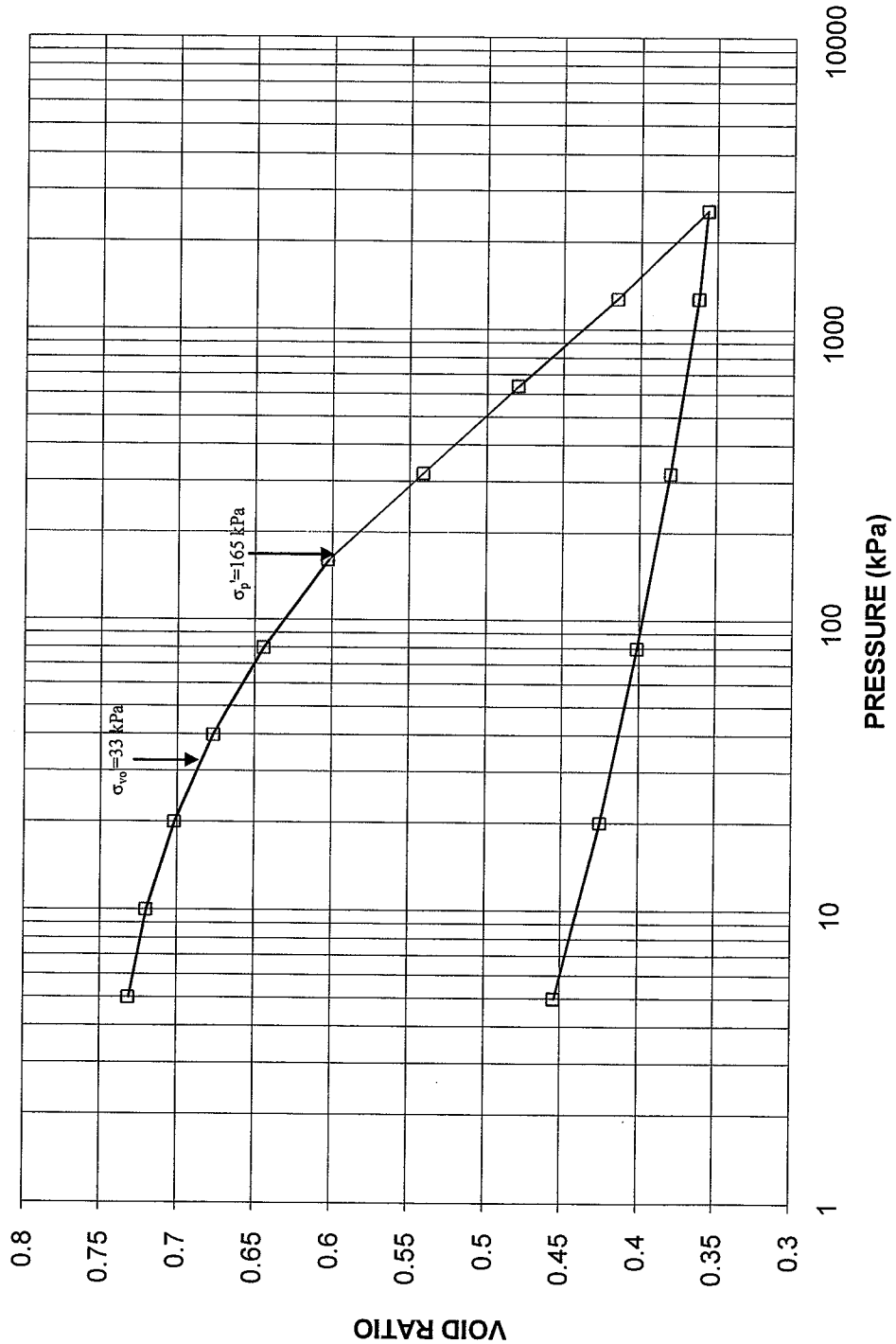
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 4 SA 4B



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A4

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 4 SA 4B



Project No. 04-1111-024A

Prepared By: LFG

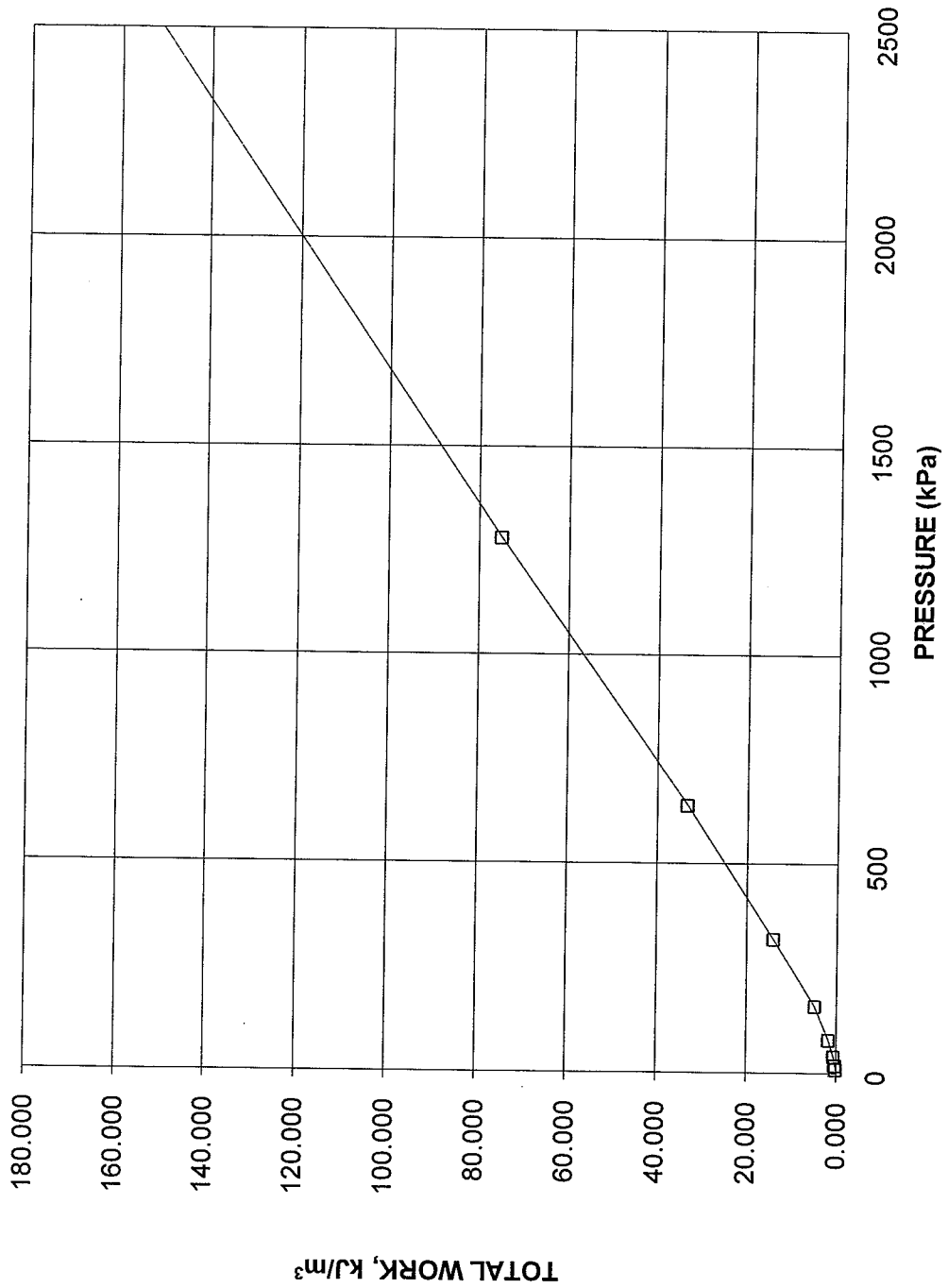
Golder Associates

Checked By: MM

**CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE**

FIGURE A5

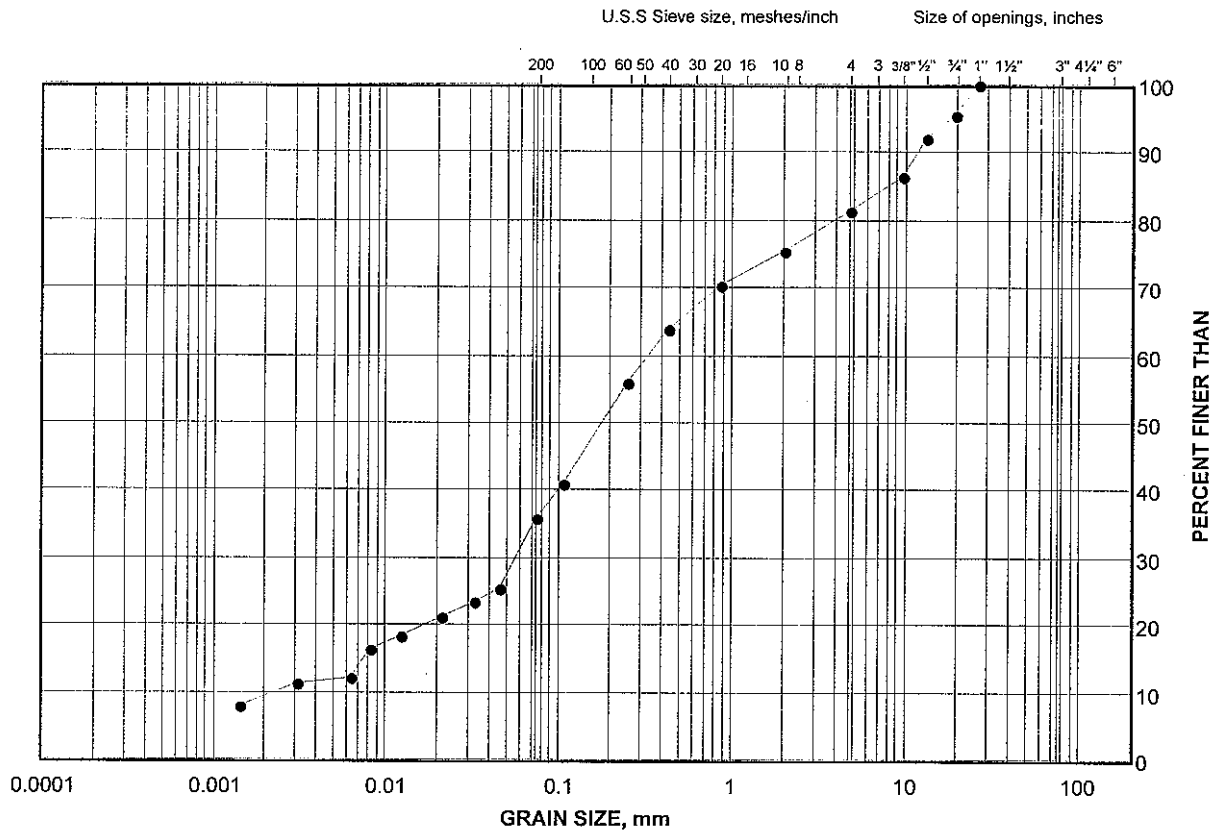
**CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 4 SA 4B**



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	ELEVATION(m)
•	06-3	4	243.0

Project Number: 04-1111-024A

Checked By: *SP*

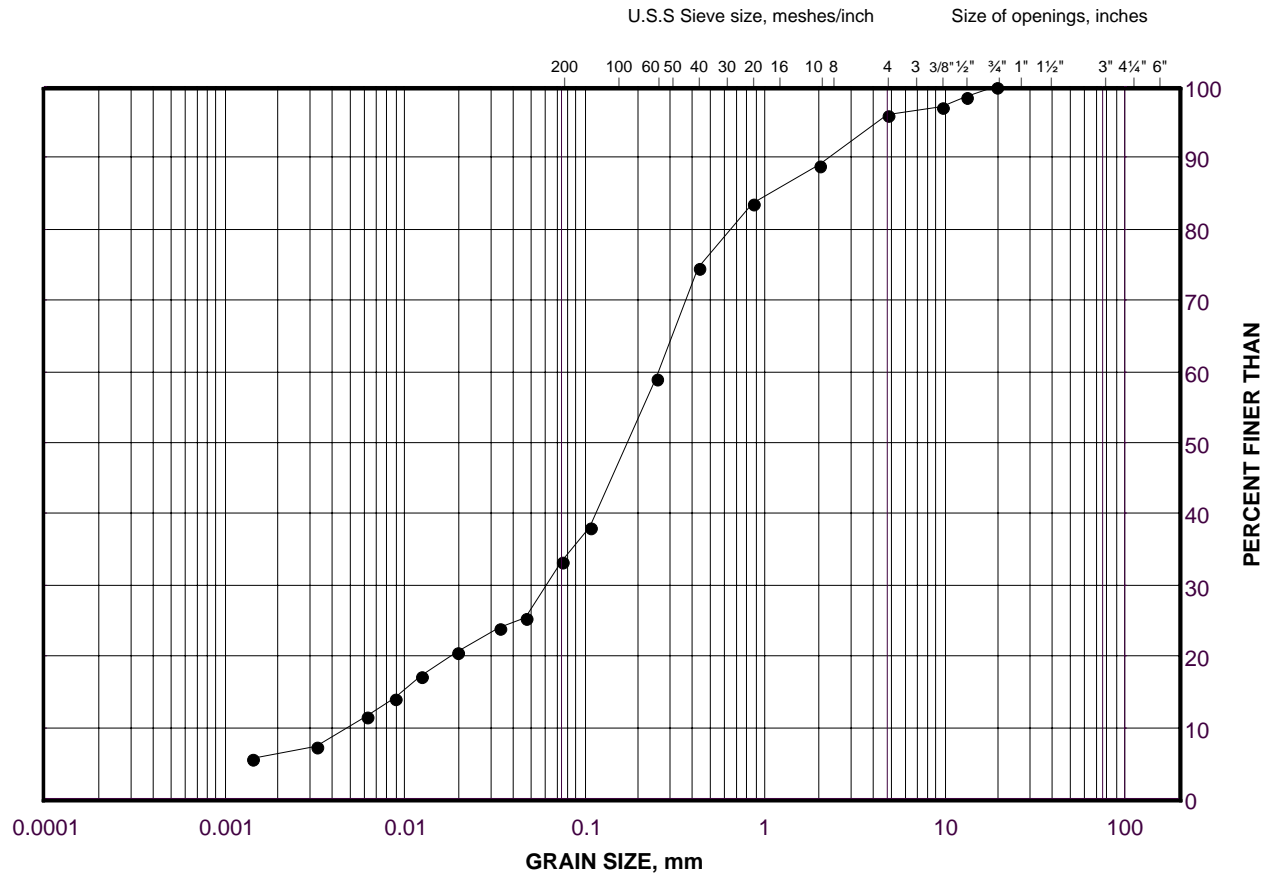
Golder Associates

Date: 15-Jun-07

GRAIN SIZE DISTRIBUTION

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)
•	06-4	10	238.7

Project Number: 04-1111-024A

Checked By: _____

Golder Associates

Date: 15-Jun-07

APPENDIX B

**SUBSURFACE INFORMATION FROM
PREVIOUS 1957 INVESTIGATION**

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 57-1

Project: Bridge over Jackson Creek
 Location: Near Peterborough, Ontario
 Hole Location As shown on enclosure No. 1.
 Hole Elevation and Datum: 804.6 Ft.: M.S.L.
 Field Work Begun Ended

Field Supervisor: A. H.
 Driller: Chevrier
 Prep.: J.S.
 Checked:

LEGEND

Sampling Method

2" Dia. split tube

2" Shelby tube

Penetration Resistance

2" Split tube

3" Dia. Cone

Casing 2" T.W. Sampler

Strength

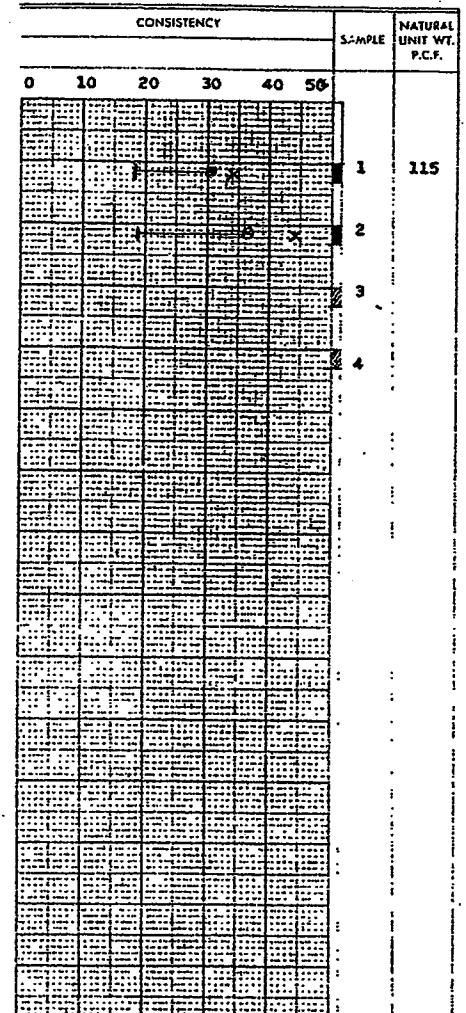
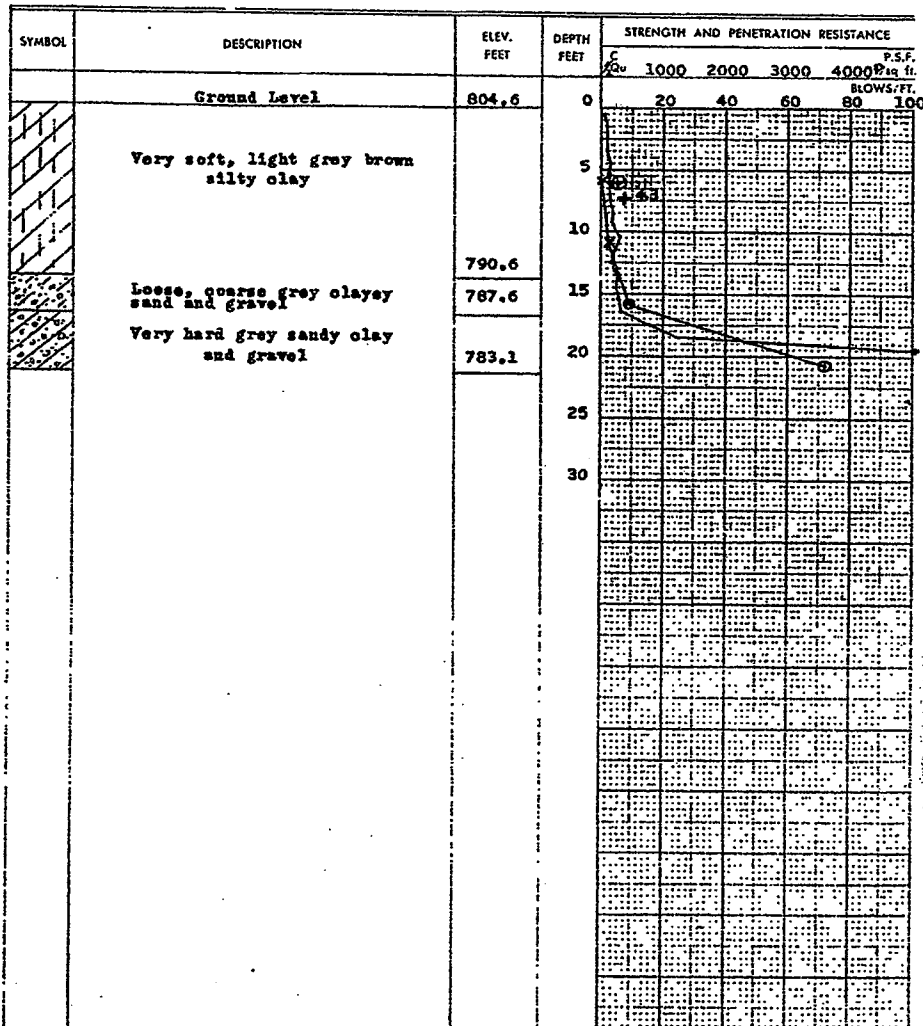
Unconfined compression (Q_u)Vane test (C) and sensitivity (S)

Consistency

Natural moisture and

Liquid limit (LL)

Plastic limit



Foundation Engineering Division

Engineering Data Sheet for Boreholes

57-2

Project: Bridge over Jackson Creek
Location: Near Peterborough, Ontario.
Hole Location As shown on enclosure No. 1
Hole Elevation and Datum: 805.4 ft.; M.S.L.
Field Work Begun Ended

Field Supervisor: A. H.
Driller: Chevrier
Prep.: J.S.
Checked:

LEGEND

Sampling Method

2. Das sind sehr tolle

2' Shelly tubs

Ecotropica **Ex-silence**

2. 3. 4. 5. 6. 7. 8. 9. 10. 11. 12. 13. 14. 15. 16. 17. 18. 19. 20. 21. 22. 23. 24. 25. 26. 27. 28. 29. 30. 31. 32. 33. 34. 35. 36. 37. 38. 39. 40. 41. 42. 43. 44. 45. 46. 47. 48. 49. 50. 51. 52. 53. 54. 55. 56. 57. 58. 59. 60. 61. 62. 63. 64. 65. 66. 67. 68. 69. 70. 71. 72. 73. 74. 75. 76. 77. 78. 79. 80. 81. 82. 83. 84. 85. 86. 87. 88. 89. 90. 91. 92. 93. 94. 95. 96. 97. 98. 99. 100. 101. 102. 103. 104. 105. 106. 107. 108. 109. 110. 111. 112. 113. 114. 115. 116. 117. 118. 119. 120. 121. 122. 123. 124. 125. 126. 127. 128. 129. 130. 131. 132. 133. 134. 135. 136. 137. 138. 139. 140. 141. 142. 143. 144. 145. 146. 147. 148. 149. 150. 151. 152. 153. 154. 155. 156. 157. 158. 159. 160. 161. 162. 163. 164. 165. 166. 167. 168. 169. 170. 171. 172. 173. 174. 175. 176. 177. 178. 179. 180. 181. 182. 183. 184. 185. 186. 187. 188. 189. 190. 191. 192. 193. 194. 195. 196. 197. 198. 199. 200. 201. 202. 203. 204. 205. 206. 207. 208. 209. 210. 211. 212. 213. 214. 215. 216. 217. 218. 219. 220. 221. 222. 223. 224. 225. 226. 227. 228. 229. 230. 231. 232. 233. 234. 235. 236. 237. 238. 239. 240. 241. 242. 243. 244. 245. 246. 247. 248. 249. 250. 251. 252. 253. 254. 255. 256. 257. 258. 259. 260. 261. 262. 263. 264. 265. 266. 267. 268. 269. 270. 271. 272. 273. 274. 275. 276. 277. 278. 279. 280. 281. 282. 283. 284. 285. 286. 287. 288. 289. 290. 291. 292. 293. 294. 295. 296. 297. 298. 299. 300. 301. 302. 303. 304. 305. 306. 307. 308. 309. 310. 311. 312. 313. 314. 315. 316. 317. 318. 319. 320. 321. 322. 323. 324. 325. 326. 327. 328. 329. 330. 331. 332. 333. 334. 335. 336. 337. 338. 339. 340. 341. 342. 343. 344. 345. 346. 347. 348. 349. 350. 351. 352. 353. 354. 355. 356. 357. 358. 359. 360. 361. 362. 363. 364. 365. 366. 367. 368. 369. 370. 371. 372. 373. 374. 375. 376. 377. 378. 379. 380. 381. 382. 383. 384. 385. 386. 387. 388. 389. 390. 391. 392. 393. 394. 395. 396. 397. 398. 399. 400. 401. 402. 403. 404. 405. 406. 407. 408. 409. 410. 411. 412. 413. 414. 415. 416. 417. 418. 419. 420. 421. 422. 423. 424. 425. 426. 427. 428. 429. 430. 431. 432. 433. 434. 435. 436. 437. 438. 439. 440. 441. 442. 443. 444. 445. 446. 447. 448. 449. 450. 451. 452. 453. 454. 455. 456. 457. 458. 459. 460. 461. 462. 463. 464. 465. 466. 467. 468. 469. 470. 471. 472. 473. 474. 475. 476. 477. 478. 479. 480. 481. 482. 483. 484. 485. 486. 487. 488. 489. 490. 491. 492. 493. 494. 495. 496. 497. 498. 499. 500. 501. 502. 503. 504. 505. 506. 507. 508. 509. 510. 511. 512. 513. 514. 515. 516. 517. 518. 519. 520. 521. 522. 523. 524. 525. 526. 527. 528. 529. 530. 531. 532. 533. 534. 535. 536. 537. 538. 539. 540. 541. 542. 543. 544. 545. 546. 547. 548. 549. 550. 551. 552. 553. 554. 555. 556. 557. 558. 559. 560. 561. 562. 563. 564. 565. 566. 567. 568. 569. 570. 571. 572. 573. 574. 575. 576. 577. 578. 579. 580. 581. 582. 583. 584. 585. 586. 587. 588. 589. 590. 591. 592. 593. 594. 595. 596. 597. 598. 599. 600. 601. 602. 603. 604. 605. 606. 607. 608. 609. 610. 611. 612. 613. 614. 615. 616. 617. 618. 619. 620. 621. 622. 623. 624. 625. 626. 627. 628. 629. 630. 631. 632. 633. 634. 635. 636. 637. 638. 639. 640. 641. 642. 643. 644. 645. 646. 647. 648. 649. 650. 651. 652. 653. 654. 655. 656. 657. 658. 659. 660. 661. 662. 663. 664. 665. 666. 667. 668. 669. 670. 671. 672. 673. 674. 675. 676. 677. 678. 679. 680. 681. 682. 683. 684. 685. 686. 687. 688. 689. 690. 691. 692. 693. 694. 695. 696. 697. 698. 699. 700. 701. 702. 703. 704. 705. 706. 707. 708. 709. 710. 711. 712. 713. 714. 715. 716. 717. 718. 719. 720. 721. 722. 723. 724. 725. 726. 727. 728. 729. 730. 731. 732. 733. 734. 735. 736. 737. 738. 739. 740. 741. 742. 743. 744. 745. 746. 747. 748. 749. 750. 751. 752. 753. 754. 755. 756. 757. 758. 759. 760. 761. 762. 763. 764. 765. 766. 767. 768. 769. 770. 771. 772. 773. 774. 775. 776. 777. 778. 779. 780. 781. 782. 783. 784. 785. 786. 787. 788. 789. 790. 791. 792. 793. 794. 795. 796. 797. 798. 799. 800. 801. 802. 803. 804. 805. 806. 807. 808. 809. 810. 811. 812. 813. 814. 815. 816. 817. 818. 819. 820. 821. 822. 823. 824. 825. 826. 827. 828. 829. 830. 831. 832. 833. 834. 835. 836. 837. 838. 839. 840. 841.

2005 年 12 月 31 日

2° T.W. Sampler

שיעורי

Unconfined compression Q:

220. 20. 2. and similarly

ಪ್ರತಿಭಟನೆ

[illegible]

6. 2004 年 12 月 31 日

உருவம் இல்லை

$p_{1,2} = 1, 2$

| SYMBOL | DESCRIPTION | ELEV.
FEET | DEPTH
FEET | STRENGTH AND PENETRATION RESISTANCE | | | | |
|--------|---|---------------|---------------|-------------------------------------|------|------|------|----------------------------------|
| | | | | 1000 | 2000 | 3000 | 4000 | P ₁₀ ft.
BLOWS/FT. |
| | Ground Level | 805.4 | 0 | | | | | |
| | Soft, light brown, sandy silty clay | 800.4 | 5 | | | | | |
| | Stiff gray silty clay | 796.4 | 10 | | | | | |
| | Loose, gray clayey sand with fine to medium gravel. | 792.4 | 15 | | | | | |
| | Very dense clayey sand and gravel | 782.4 | 20 | | | | | |

| CONSISTENCY | | | | | | SAMPLE | NATURAL
UNIT WT.
P.C.F. |
|-------------|----|----|----|----|----|--------|-------------------------------|
| 0 | 10 | 20 | 30 | 40 | 50 | | |
| | | X | X | | | 1 | 122 |
| | | | | | | 2 | |
| | | | | | | 3 | |

Foundation Engineering Division

57-3

Field Supervisor: A. H.
Driller: Chevrier
Prep.: J.S.
Checked:

Sampling Method

- ### Penetration Resistance

2. $S_{\mu} = 1$ and $\mu = 1$

2. 15. 1944
1944. 12. 12

24

५६९१

True $\rho = 0$ and sensitivity : 5

Co. Ltd. 20

Nationalist minister and

2000-2001 Index 26

Partic: 12.1

[illegible]

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole

57-4

Project: Bridge over Jackson Creek
Location: Near Peterborough, Ontario
Hole Location As shown on enclosure No. 1
Hole Elevation and Datum: 804.7 ft., M.S.L.
Field Work Begun _____ Ended _____

Field Supervisor: A. H.

Driller: **Chevrier**

Prep.: J. S.

Checked:

Ended

LEGEND

Sampling Method

2' Dia split tube

2" Shelby tube

Penetration: Resistance

2" Spint tube

2. 210 6.000

Casing 2" T.W. Sampler

Strangin

Unconfined compression (Qu)

Value test C and sensitivity (S):

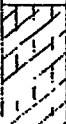


Corvinus

Rate of moisture and

1. Index

Liquid time:

Plastic limit

| SYMBOL | DESCRIPTION | ELEV.
FEET | DEPTH
FEET | STRENGTH AND PENETRATION RESISTANCE | | | | |
|---|--|---------------|---------------|-------------------------------------|---------------------------------|----|----|-----|
| | | | | C
Qu | P.S.F.
1"sq. ft.
SICWS/FT | | | |
| | Ground Level | 804.7 | 0 | 20 | 40 | 60 | 80 | 100 |
|  | Soft, light brown silty clay | 794.7 | 5 | | | | | |
|  | Loose, light brown clayey sand
with gravel | 789.7 | 10 | | | | | |
|  | Dense to very dense coarse
clayey sand with gravel and
boulders up to 6" thick | 778.7 | 15 | | | | | |
| | | | 20 | | | | | |
| | | | 25 | | | | | |
| | | | 30 | | | | | |

[illegible]