

GEOCRES No:
31F-146

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**FOUNDATION INVESTIGATION AND DESIGN REPORT
NORTH SERVICE ROAD BRIDGE
OVER LAVALLEE CREEK
HIGHWAY 7 TWINNING FROM 0.7 KM WEST
OF JINKINSON ROAD WESTERLY 10.5 KM TO
2.5 KM WEST OF ASHTON STATION ROAD
W.P. 251-99-00**

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PART A

**FOUNDATION INVESTIGATION REPORT
NORTH SERVICE ROAD BRIDGE OVER LAVALLEE CREEK
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WESTERLY 10.5 KM TO 2.5 KM WEST OF ASHTON STATION ROAD
W.P. 251-99-00**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Marshall Macklin Monaghan (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the twinning of Highway 7 from two to four lanes in the former West Carleton Township which is now part of the City of Ottawa, and in Beckwith Township in Lanark County. The section of Highway 7 included in this assignment (W.P. 251-99-00) extends from 0.7 km west of Jinkinson Road westerly for 10.5 km, to 2.5 km west of Ashton Station Road, and includes service roads to accommodate future construction on Highway 7.

Foundation investigation services are required for the following components under W.P. 251-99-00.

- A new bridge to carry the North Service Road over Lavallee Creek; and
- High fill embankments along Highway 7 and at the Dwyer Hill Road and Ashton Station Road interchanges.

This report addresses the proposed North Service Road bridge over Lavallee Creek and its approach embankments.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated July 2002, and in Section 5.8 of MMM's *Technical Proposal* for this project. Scope changes (Scope Change No. 1, related to additional borehole investigation work for ground-mounted signs and for in-water drilling at the Lavallee Creek abutments) have been addressed in Golder's letter to MMM dated August 23, 2004.

2.0 SITE DESCRIPTION

The site of the proposed North Service Road bridge over Lavallee Creek is located approximately 1.5 km east of Carleton Place, in Beckwith Township in Lanark County. The North Service Road is to be located approximately 20 m south of the existing Trans-Canada Trail (former rail right-of-way) at the proposed crossing location. The Trans-Canada Trail embankments are approximately 2 m to 2.5 m in height adjacent to Lavallee Creek.

At the location of the crossing, Lavallee Creek is oriented approximately north-south, and flows in a northerly direction. The creek bed is at approximately Elevation 125.0 m to 125.4 m, and the water in the creek at the time of the borehole investigation was about 0.6 m to 0.8 m deep. It is understood that the water level in the creek during the 100-year storm event is at approximately Elevation 127.1 m.

Immediately west and east of the creek, the land is relatively flat at about Elevation 125.5 m to 126 m; these areas are low-lying relative to the surrounding farm fields. On the west side of the creek at the proposed crossing location, a grass-covered marsh is present, and standing surface water has been observed covering most of this area. On the east side of the creek, up to about 0.3 m of standing water has been observed during wet periods of the year, although no standing water is present during warm, relatively dry periods of weather. It is noted that a steeply-sloping mound is present on the east bank of the Lavallee Creek, between the creek channel and the floodplain to the east. This "mound" is about 1 m wide at the top, and is about 2 m high relative to the creek water level.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out between November 22 and December 17, 2004. During this period, a total of six boreholes (Boreholes LC-1 to LC-6) were put down at the locations shown on Drawing 1. The boreholes were advanced using a portable drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The boreholes were advanced to depths which vary from 6.3 m to 15.6 m below present ground surface.

Boreholes LC-1 and LC-2 were advanced within the footprint of the proposed west abutment of the Lavallee Creek structure, and Boreholes LC-3 and LC-4 were advanced within the proposed east abutment footprint. All four of these boreholes were drilled using a raft located within Lavallee Creek. Two of the boreholes (Boreholes LC-1 and LC-3) were advanced about 3 m into the bedrock using BQ-size coring equipment. The other two boreholes (Boreholes LC-2 and LC-4) were advanced about 0.3 m into the bedrock using BQ-sized coring equipment; this shallow bedrock coring was necessary to confirm that the bedrock surface had been reached following refusal to split-spoon sampling (since the portable drilling equipment does not use a rotary auger system). Samples of the overburden were obtained at 0.6 m to 1.2 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure.

Boreholes LC-5 and LC-6 were advanced for the east and west approach embankment, respectively, about 20 m behind the proposed foundation elements. These boreholes were terminated within the overburden at a depth of about 6.3 m. Samples of the overburden were generally obtained at 0.6 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the SPT procedure. In Borehole LC-5, where it was possible to set up a "reaction frame" for the portable drill rig using existing trees in the area, the following additional sampling / testing was conducted:

- in situ vane testing (using a B-sized vane due to the size restrictions imposed by the portable rig) was carried out where possible in the silty clay to evaluate the undrained shear strength of this soil unit; and
- three relatively undisturbed, 75 mm diameter thin-walled Shelby tube (ASTM D1587) samples of the silty clay were obtained using a fixed piston sampler.

A standpipe piezometer was installed in Boreholes LC-5 and LC-6, within the silty clay deposit. The piezometers consist of 25 mm diameter PVC pipe with a slotted tip installed within a 0.7 m to 1.3 m thick filter sand pack. A bentonite seal was extended from the top of the filter sand pack to the ground surface.

The field work was supervised on a full-time basis by members 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 and bedrock samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Ottawa for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples. Laboratory oedometer consolidation testing was carried out on two samples of the silty clay deposit.

The borehole locations and ground surface elevations were determined by Golder relative to points staked by MMM. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawing 1.

<i>Borehole Number</i>	<i>Borehole Location</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
LC-1	East abutment	5,000,799.1	335,389.7	125.4
LC-2	East abutment	5,000,792.4	335,394.0	125.0
LC-3	West abutment	5,000,799.6	335,406.0	125.0
LC-4	West abutment	5,000,806.7	335,401.5	125.0
LC-5	West approach embankment	5,000,815.8	335,423.3	125.9
LC-6	East approach embankment	5,000,784.2	335,373.9	125.4

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the Smith Falls Limestone Plain, as delineated in *The Physiography of Southern Ontario*¹, that lies within the major physiographic region of the Ottawa-St. Lawrence Lowland.

The Smiths Falls Limestone Plain is characterized by shallow overburden deposits overlying sedimentary bedrock consisting of limestones, dolostones, sandstones and shales. The shallow overburden soils are typically between 1 m and 3 m in thickness and are commonly comprised of sandy to gravelly till derived from the Precambrian Shield to the north, overlain by glaciofluvial sediments that consist of layered sands and gravels. In the vicinity of and north of Carleton Place, clay has been deposited within depressions in the bedrock that have been caused by faulting. Large areas of the plain are covered with peat and muck, due to poor drainage as a consequence of the relatively flat topography and shallow depth to bedrock.¹

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, six boreholes were advanced within the limits of the foundation elements and the approach embankments for the proposed Lavallee Creek bridge. The borehole locations and ground surface elevations are shown on Drawing 1. Soil stratigraphy sections along the centreline of the proposed road and across the abutment foundation areas 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 given on the Record of Borehole sheets and Figures 1 to 6. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the soils encountered within the Lavallee Creek channel consist of alluvial deposits of silty sand and clay with organics to depths of about 0.4 m to 0.9 m, over an approximately 10.3 m to 10.7 m thick deposit of silty clay to clay. The upper 0 m to 2.2 m of this clay deposit has been weathered to a grey-brown crustal zone, while the underlying portions of the deposit are grey in colour. Below a depth of about 11.5 m to 11.8 m, the silty clay is underlain by about 0.2 m to 0.5 m of silty sand to sandy silt till. The till is, in turn, underlain by dolomitic limestone bedrock that was encountered between about 11.7 m and 12.1 m depth (at about Elevation 114.0 m to 113.4 m, respectively).

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.

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

4.2.1 Peat and Topsoil

About 500 mm of topsoil was encountered in Borehole LC-5, within the limits of the east approach embankment, and about 600 mm of peat was encountered in Borehole LC-6, which is located within the swampy area at the west approach embankment.

4.2.2 Alluvium

In Boreholes LC-1 and LC-2 at the proposed west abutment, about 0.3 m to 0.4 m of organic sandy silt to organic silty clay was encountered immediately below the creek bed. Below this, and from the creek bed level in Boreholes LC-3 and LC-4 at the east abutment, a 0.2 m to 0.5 m thick layer of alluvium is present. The alluvium consists of silty sand containing trace shells, clay or gravel and organic matter. The base of the alluvium was encountered in all of the boreholes at approximately Elevation 124.5 m. The measured SPT "N" values within this material vary from 0 (weight of rods) to 4 blows per 0.3 m of penetration, indicating that this material has a very loose to loose relative density.

At the west approach embankment, the peat encountered in Borehole LC-6 is underlain by alluvium consisting of about 0.3 m of silty clay containing some sand and organic matter, which is in turn underlain by approximately 0.9 m of silty sand containing trace shells. The base of the silty sand was encountered at approximately 1.8 m depth (Elevation 123.6 m). The measured SPT "N" values within the silty sand in Borehole LC-6 vary from 2 to 5 blows per 0.3 m of penetration, indicative of a very loose to loose relative density.

4.2.3 Silty Clay to Clay

The organic materials and alluvium are underlain by a deposit of silty clay to clay that is between 10.3 m and 10.7 m thick in the boreholes where it was completely penetrated. The surface of the silty clay to clay deposit was encountered at Elevation 124.5 m within the creek channel (Boreholes LC-1 to LC-4), and at about Elevations 125.5 m and 123.6 m in Boreholes LC-5 and LC-6 on the east and west sides of the creek, respectively.

Weathered Clay Crust

At Boreholes LC-1 and LC-4 to LC-6, the upper 0.6 m to 2.2 m of the silty clay to clay deposit has been weathered to a grey-brown crust. The results of a grain size distribution test carried out on one selected sample of this material are provided on Figure 1. The measured SPT "N" values in this portion of the deposit were 5 blows per 0.3 m of penetration. In situ vane testing carried

out within the weathered crust in Borehole LC-5 measured undrained shear strengths in excess of 120 kPa, and a laboratory vane test on the Shelby tube sample of this material measured a shear strength of 240 kPa; these test results indicate that the weathered crust has a very stiff consistency.

The results of Atterberg limit testing on selected samples of the weathered crust indicate plasticity indices ranging from 23 to 26 per cent and liquid limits ranging from 56 to 59 per cent. These results, summarized on the plasticity chart on Figure 2, confirm that this material is a clay of high plasticity. The measured natural water content of samples of the weathered crust ranges from 37 to 41 per cent.

Oedometer consolidation testing was carried out on one thin-walled Shelby tube sample of the weathered crust. The results of that testing, which are provided on Figure 3 and summarized in the table below, indicate that the weathered crust is overconsolidated, with a preconsolidation pressure of approximately 620 kPa.

<i>Borehole/ Sample No.</i>	<i>Sample Depth/Elev.</i>	<i>Unit Weight</i>	<i>σ_p' (kPa)</i>	<i>σ_{vo}' (kPa)</i>	<i>$\sigma_p' - \sigma_{vo}'$ (kPa)</i>	<i>C_c</i>	<i>C_r</i>	<i>e_o</i>	<i>OCR</i>	<i>C_v</i>
LC-5 Sa 3	1.9/124.0 m	17.6 kN/m ³	620	23	597	0.51	0.017	1.203	27	0.01

NOTES:

- σ_p' - Apparent preconsolidation pressure
- σ_{vo}' - Computed existing vertical effective stress
- C_c - Compression index
- C_r - Recompression index
- e_o - Initial void ratio
- OCR - Overconsolidation ratio
- C_v - Coefficient of consolidation

Unweathered Silty Clay to Clay

The silty clay to clay below the depth of weathering, and below the alluvium and silty sands in Boreholes LC-2 and LC-3, is grey in colour. The results of grain size distribution testing carried out on three selected samples of this material are provided on Figure 4.

The measured SPT "N" values within the unweathered silty clay to clay range from 3 to 9 blows per 0.3 m of penetration, except at Borehole LC-5 where a measured SPT "N" value of 1 blow per 0.3 m of penetration was recorded. In situ vane testing carried out in Borehole LC-5 measured undrained shear strengths ranging from 57 kPa to in excess of 120 kPa, and a laboratory vane test on the Shelby tube sample of this material measured a shear strength of 124 kPa. These results indicate that the unweathered, grey silty clay to clay has a generally very stiff consistency at all the boreholes with the exception of Borehole LC-5, where the consistency ranges from stiff to very stiff. Remoulded shear strengths of approximately 6 kPa to 11 kPa were

measured in Borehole LC-5, corresponding to sensitivities of approximately 8 to 10; these results indicate that the unweathered silty clay to clay is sensitive.

The results of Atterberg limit testing on selected samples of this portion of the deposit indicate plasticity indices ranging from 23 to 27 per cent, and liquid limits ranging from 48 to 53 per cent. These results, which are summarized on the plasticity chart on Figure 5, indicate that this unweathered material is a silty clay to clay of intermediate to high plasticity. The measured natural water contents of samples of the unweathered, grey silty clay to clay range from 28 to 47 per cent.

Oedometer consolidation testing was carried out on one thin-walled Shelby tube sample of the silty clay to clay obtained just below the depth of weathering. The results of that testing, which are provided on Figure 6 and summarized in the table below, indicate that this material is overconsolidated, with a preconsolidation pressure about 300 kPa and an overconsolidation ratio of approximately 9 in the upper portion of the unweathered material. Based on the results of the field vane testing in Borehole LC-5, the overconsolidation ratio of the unweathered clay decreases with depth.

Borehole/ Sample Number	Sample Depth/Elev. (m)	Unit Weight (kN/m ³)	σ_p' (kPa)	σ_{vo}' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	Cc	Cr	e _o	OC R	Cv
LC-5 Sa 5	3.7 / 121.8	17.2	300	35	265	0.61	0.023	1.373	9	0.01

Notes:

- σ_p' - Apparent preconsolidation pressure
- σ_{vo}' - Computed existing vertical effective stress
- Cc - Compression index
- Cr - Recompression index
- e_o - Initial void ratio
- OCR - Overconsolidation ratio
- Cv - Coefficient of consolidation

4.2.4 Silty Sand to Sandy Silt Till

A 0.2 m to 0.5 m thick layer of glacial till was encountered below the silty clay to clay deposit in Boreholes LC-1 to LC-4. The surface of this till deposit was encountered between about Elevations 113.7 m and 114.2 m in the boreholes (at depths below ground surface ranging from 10.8 m to 11.8 m).

Based on local experience and observations of the drilling resistance, the glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand to sandy silt, containing trace clay. Due to the limited thickness of this deposit, only limited standard penetration testing could be carried out before sampler refusal was encountered on the bedrock surface. However, one measured SPT "N" value of 15 blows per 0.3 m of penetration indicates the deposit to have a compact relative density.

4.2.5 Dolomitic Limestone Bedrock

Dolomitic limestone bedrock, containing interbeds of limestone and sandstone, underlies the till deposit at this site. In the four boreholes put down at the proposed abutments, the surface of the bedrock was encountered between Elevations 113.4 m and 114.0 m. The following table summarizes the depth to the bedrock surface and its elevation as encountered at the locations of Boreholes LC-1 to LC-4; the bedrock was cored in all four of these boreholes.

<i>Borehole Location</i>	<i>Borehole Number</i>	<i>Ground Surface Elevation (m)</i>	<i>Depth to Bedrock (m)</i>	<i>Bedrock Surface Elevation (m)</i>
West Abutment	LC-1	125.4	12.1	113.4
	LC-2	125.0	12.0	113.6
East Abutment	LC-3	125.0	11.7	114.0
	LC-4	125.0	12.0	113.8

The dolomitic limestone bedrock at the site is a member of the Rockcliffe Formation; it is fresh, medium strong, and thinly- to medium-bedded. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 50 to 100 per cent, indicating that the bedrock is of fair to excellent quality. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although minor vertical to sub-vertical jointing was also observed. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.3 Groundwater Conditions

Piezometers were installed in Boreholes LC-5 and LC-6 within the overburden soil deposits, outside of the Lavallee Creek channel, and the water levels measured in these piezometers on May 9, 2005 are summarized in the following table:


<i>Borehole No.</i>	<i>Borehole Location</i>	<i>Depth</i>	<i>Elevation</i>
LC-5	East approach embankment	0.0 m	125.9 m
LC-6	West approach embankment	0.8 m above g.s.	126.2 m

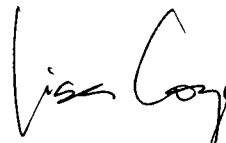
The water levels measured in the piezometers in May 2005 are about 0.2 m to 0.6 m above the Lavallee Creek stage of Elevation 125.6 m to 125.7 m that was measured during drilling in November and December 2004. Based on site observations in the fall and winter of 2004 and the spring of 2005, these high water conditions (i.e. water at or above ground surface) should be expected throughout the year. Slightly lower water levels, though still at or near the ground surface, should be expected during drier periods in the summer months.

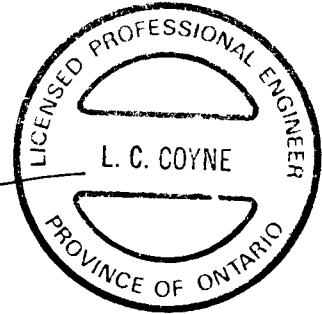
5.0 CLOSURE

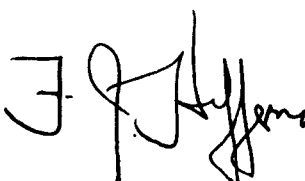
This Foundation Investigation Report was prepared by Mr. William Cavers, EIT, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and Senior Engineer with Golder, with technical input from Mr. Michael Cunningham, P.Eng., an Associate and Senior Engineer with Golder. Mr. Fintan Heffernan, a Designated MTO Contact for Golder, conducted an independent review of the report.

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WC/LCC/MIC/FJH/wc/lcc/cr/al

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PART B

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6.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed single-span bridge that will carry the new North Service Road over Lavallee Creek. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

5.2 Bridge and Retaining Wall Foundation Options

The main concern for design and construction of this single-span bridge is settlement of the approximately 10.5 m thick, compressible silty clay to clay stratum that underlies the Lavallee Creek site, and the impacts that this settlement will have on the performance of the bridge and the North Service Road. As discussed further in Section 5.7.3, approximately 80 mm of primary consolidation settlement will occur within this deposit as a result of the 3 m to 4 m high approach embankment loading (assuming the use of conventional earth or granular fill) at this site; it is estimated that it will take approximately two years to achieve 90 per cent of this primary consolidation settlement.

It is understood that an "open footing", rigid frame bridge will be adopted at this site in order to accommodate fisheries requirements. Since this type of structure is relatively intolerant of settlement, neither the use of shallow foundations for support of the abutments and associated retaining walls, nor the use of retained soil system (RSS) walls, is recommended. Although the settlement of the silty clay to clay deposit could be mitigated with the use of embankment surcharging, wick drains or lightweight fill (as discussed further in Sections 5.7.4 to 5.7.6), it is considered that steel H-piles driven to found on the bedrock represent the most feasible and cost-effective foundation solution for the North Service Road bridge. As an alternative to steel H-pile foundations, caissons socketted into the bedrock could also be used for support of the bridge.

Since the abutments are located within the Lavallee Creek channel, it is not possible to preload or surcharge the abutment area prior to installation of the pile foundations to eliminate downdrag due to consolidation of the clayey soils under the embankment loading. Extruded polystyrene (EPS) fill could be used behind the abutments to eliminate downdrag, as discussed in Section 5.7.6. However, since MMM's structural designers have indicated that deep foundations for this

structure can be readily designed to accommodate the downdrag load (based on the relatively small magnitude of the downdrag load relative to the factored axial resistance at Ultimate Limit States), the preferred alternative from a foundations perspective is the use of steel H-pile foundations in conjunction with conventional earth fill approach embankments constructed in advance and surcharged.

Geotechnical recommendations for the design of foundations for the bridge abutments and associated retaining walls are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options is presented in Table 1 following the text of this report.

5.3 Steel H-Pile Foundations

Steel H-piles driven to found on the dolomitic limestone bedrock may be used for support of the abutments. The Contract Drawings should indicate that pile installation should be in accordance with SP903S01, include the note "Piles to be driven to bedrock", and indicate that the piles should be equipped with driving shoes or rock points, as discussed below.

In the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the thin layer of glacial till which underlies the silty clay deposit at this site, and to the potential for deflection of the piles along the bedrock surface owing to the relatively hard nature of the bedrock and the relatively softer consistency of the overlying strata. Based on these considerations, vertically driven piles should be equipped with flange reinforcement (driving shoes) as per SS103-12. Any battered piles should be equipped with suitable driving points (such as Titus Ejector or equivalent) to ensure adequate seating of the piles on the bedrock. If there are relatively few vertical piles, rock points can be used for all of the piles since this may be more practicable for contractual purposes.

If necessary to resist seismic forces, the piles could be socketted into the bedrock. It is noted that the dolomitic limestone bedrock is generally medium strong, and this would require socket formation using coring or churn drilling to advance the hole. Further, the dolomitic limestone as well as the limestone and sandstone interbeds have high silica contents, are considered to be rather abrasive, and could result in relatively high equipment wear.

5.3.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to found on or socketed at least 2 m into the bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be

greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

It should be noted that the construction of the approach embankments will raise the effective stress level in the unweathered silty clay to clay, leading to some consolidation of the deposit. As discussed subsequently in Section 5.7.3 of this report, the embankment subgrade settlements are estimated to be between about 50 mm and 100 mm (from both primary consolidation and secondary compression). The elastic shortening of the piles will be significantly less, at about 5 mm under service loads, and therefore the differential settlements would be sufficient to generate downdrag forces.

In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] were considered. Considering the larger predicted settlement of the silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the silty clay deposit.

Based on the above, the unfactored downdrag load acting on a single HP 310 x 110 pile over the length of pile within the native soils is estimated to be 400 kN. 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.

5.3.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. In the case of battered piles, precautions during driving are necessary in some situations (such as for specific soil/bedrock conditions/pile lengths and where the batter is shallower than 6 vertical to 1 horizontal) to ensure that the piles do not deflect along the bedrock surface even with relatively flat-lying bedrock. It is recommended that the pile batter be restricted to 3V:1H or steeper.

If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesive soils:

$$k_h = \frac{k_{s1}}{B} \quad \text{Where: } B \text{ is the pile diameter/width (m)}$$

k_{s1} is the constant of horizontal subgrade reaction, as given below

Since the underside of the pile caps is at approximately Elevation 123.5 m, the piles will extend only through the silty clay to clay soils at the site. The constant of horizontal subgrade reaction, k_{s1} , for the silty clay to clay may be taken as 8 MPa in the structural analysis. It should be noted that although lower undrained shear strengths of approximately 60 kPa (which would correspond to a k_{s1} value of approximately 4 MPa) were measured within the unweathered silty clay to clay at the east approach embankment, the unweathered silty clay to clay encountered in Boreholes LC-1 to LC-4 is very stiff. Hence, the k_{s1} value of 8 MPa given above is recommended for both the weathered and unweathered portions of the silty clay to clay at the abutment locations.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

5.3.3 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

5.4 Caissons

As an alternative to steel H-piles, caissons may be used for support of the bridge abutments. Based on the proposed North Service Road grade at about Elevation 129 m and the assumed pile cap base at about Elevation 124 m, the caissons would be about 10 m in length (excluding the rock socket).

It is noted that the native marine (Champlain Sea) clay at this site is a sensitive soil. The disturbed clay could "flow" into the auger hole during caisson installation if left unsupported. The use of a temporary liner or casing will be required in order to advance the caissons with minimal loss of ground. In this regard, it is important that the liner be advanced ahead of the augers to avoid inflow of the sensitive clay into the caisson hole; provided that this done, it is not necessary to place restrictions on the use of a vibratory hammer to install the temporary casing. For constructability reasons, it is recommended that the caissons be seated (socketed nominally) into the bedrock in order to avoid loss of the till soils into the auger hole at the bedrock interface.

The limestone bedrock at the site is moderately strong. If socketing of the caissons into the bedrock is required, the sockets will have to be advanced by rock coring or churn drilling. The dolomitic limestone as well as the limestone and sandstone interbeds have high silica contents, are considered to be rather abrasive, and could result in relatively high equipment wear.

5.4.1 Axial Geotechnical Resistance

Caissons socketed nominally (less than 1 m) into the bedrock should be designed based on end-bearing resistance using a factored geotechnical resistance at ULS of 6 MPa. Serviceability Limit States (SLS) resistances do not apply to caissons founded on the dolomitic limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

It should be noted that the construction of the approach embankments will raise the effective stress level in the grey silty clay deposit at depth close to its estimated preconsolidation pressure. That stress increase will lead to some consolidation of the deposit and will result in downdrag forces on caissons supporting the abutments and wing walls or retaining walls. The unfactored downdrag load acting on a single 1.5 m diameter caisson over its length is estimated to be 2,000 kN. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*. The assumptions and methods used in assessing that downdrag force are the same as those described in Section 5.3.1 of this report with respect to steel H-piles. The guidelines provided in that section of the report for reducing or eliminating the downdrag forces are equally applicable to this foundation option.

5.4.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 5.3.2.

5.4.3 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

5.5 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site in accordance with Section 4.4.6 of the *CHBDC* may be taken as 1.0, consistent with Soil Profile Type I.

5.6 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. Seismic (earthquake) loading must also be taken into account in the design.

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' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with MTO's Special Provision SP105S10. 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 3501.00 and 3504.00.
- 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 MTO's 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 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(I) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(I) 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:

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
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 the National Building Code of Canada, this site is located in Seismic Zone 4. The site-specific zonal acceleration ratio for Ottawa is 0.18. Based on experience, for the subsurface conditions at this site, a 50 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.18g to about 0.27g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.27$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which do not allow lateral yielding (i.e. the abutment walls for this structure), the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.41$). For structures which allow lateral yielding (i.e. the wing walls for this structure), k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.14$). The seismic active earth pressure coefficient is 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 $k_v = +2/3 k_h$, $k_v = 0$, and $k_v = -2/3 k_h$.
- The following seismic active pressure coefficients (K_{AE}) for the two backfill 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. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

Wall Type	Case I	Case II
Yielding wall	0.43	0.34
Non-yielding wall	1.5	0.9

- 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 0.2. This corresponds to displacements of up to approximately 50 mm at this site.
- The earthquake-induced dynamic 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:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d)$$

where $\sigma_h(d)$ is the lateral earth pressure at depth, d , (kPa)
 K_a is the static active earth pressure coefficient;
 K_{AE} is the seismic active earth pressure coefficient;
 γ is the unit weight of the backfill soil (kN/m^3), as given previously;
 d is the depth below the top of the wall (m); and
 H is the total height of the wall (m).

5.7 Approach Embankment Design and Construction

In the immediate vicinity of the proposed North Service Road bridge, the existing ground surface is at about Elevation 125 m to 126 m. Based on the information contained in the drawings provided by MMM, the proposed North Service Road grade at the bridge site will be at about Elevation 129 m. The east and west approach embankments will, therefore, be approximately 3 m to 4 m high relative to the existing natural ground surface at the site. The construction of the North Service Road approach embankments will actually require placement of about 3.5 m to 4.5 m of fill (following subexcavation of any peat or organic material).

Following removal of the organic materials, the embankment subgrade soils will consist of up to about 1 m of silty sand overlying about 10.3 m to 10.7 m of silty clay to clay.

5.7.1 Subgrade Preparation and Approach Embankment Construction

All topsoil, peat and organic alluvium should be stripped from beneath the approach embankment footprints prior to embankment fill placement. The following stripping depths should be given in the Contract:

<i>Location</i>	<i>Depth of Stripping</i>
West approach	0.5 m
East approach:	
Within creek channel	0.6 m
Within bank/floodplain area	0.9 m

It should be noted that, following stripping of the peat/organics, the clayey subgrade soils will be highly susceptible to disturbance as a result of equipment travelling over the surface. An Operational Constraint has been developed (and is provided in Appendix A) to restrict equipment travel over the subgrade surface and require timely placement of the first lift(s) of embankment fill.

The approach embankment fill should be placed and compacted in accordance with MTO's Special Provision SP105S10, as well as the provisions of OPSS 209. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

5.7.2 Approach Embankment Stability

Static and seismic slope stability analyses of the approach embankments and abutment foreslopes were carried out with the commercially available program SLOPE-W (produced by Geo-Slope International Ltd.), using the soil parameters given in the following table.

Soil Deposit	Bulk Unit Weight	Effective Friction Angle	Undrained Shear Strength
Embankment Fill (range of parameters assumed for earth and granular fill)	20 – 22 kN/m ³	32° to 35°	–
Silty sand	19 kN/m ³	28°	–
Weathered clay crust	18 kN/m ³	–	80 kPa
Unweathered silty clay to clay	18 kN/m ³	–	40 kPa
Silty sand to sandy silt till	21 kN/m ³	N/A (See note)	

NOTE: Due to the significant difference in shear strength and stress-strain response of the (marine) silty clay versus the underlying compact to dense granular till deposit, the critical failure surface for a deep-seated instability of the embankment does not penetrate the till deposit.

Although the undrained shear strength of the weathered silty clay to clay was measured to be greater than 120 kPa, an undrained shear strength of 80 kPa was selected for the weathered crust for these analyses, based on strain compatibility with the underlying, unweathered clay. The lowest undrained shear strength of the unweathered silty clay to clay was measured to be 57 kPa; the value of 40 kPa used in these analyses reflects corrections for the plasticity of this material.

The results of the slope stability analyses indicate that, with appropriate subgrade preparation and proper placement and compaction of the embankment fill materials, 3.5 m to 4.5 m high approach embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have a factor of safety of greater than 1.3 against deep-seated slope instability for the undrained conditions during and immediately after embankment construction. Under seismic loading conditions, using a seismic coefficient of 0.1, a factor of safety of greater than 1.1 is obtained.

Scour protection should be provided as necessary to prevent erosion/undermining and preserve the stability of the embankment slopes adjacent to the creek channel.

5.7.3 Approach Embankment Settlement

Settlement of the approach embankments will occur as a result of compression of the new embankment fill itself, as well as consolidation of the clayey soils on which the approaches will be founded.

Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the embankment fill itself is expected to be less than 25 mm. The use of granular fill for the new embankment construction would reduce this magnitude of settlement since the majority of settlement of granular fills will occur during construction, whereas the majority of the settlement of cohesive fill, if used, would occur after construction.

Some settlement of the embankment subgrade can be expected due to compression of the clay soils (i.e., the weathered clay crust and, in particular, the underlying grey silty clay to clay). The results of the oedometer consolidation testing indicate that the effective stress level in the clayey deposits will approach and potentially exceed the deposit's preconsolidation pressure, at least near the bottom of the deposit. The resulting consolidation settlements therefore correspond to recompression of the clayey deposits and some potential consolidation in the virgin compression range.

The total estimated magnitude of the primary consolidation settlement is up to about 80 mm, based on the values measured in the oedometer test and the vane shear strength results (as summarized in Section 4.2.3 of this report). It is estimated that 90 per cent of the primary consolidation settlement should be completed within about two years. Based on the anticipated loading (assuming the use of earth fill or granular fill for construction of the approach embankments), approximately 20 mm of additional secondary compression is anticipated over a twenty-year time span, by which time resurfacing of the highway might be expected.

Based on the estimated settlements, it is recommended that the approach embankments be constructed as early as possible in the contract to allow the maximum amount of time available for settlement prior to paving of the North Service Road. However, it may still be necessary to pad and overlay the approach embankments for the North Service Road bridge in the first couple of year following paving. If this maintenance is considered unacceptable, the following options could be considered for mitigation of post-construction settlement:

- Surcharge the approach embankment areas to increase the magnitude of settlement during the preload period, prior to paving.
- Install wick drains to accelerate the consolidation settlement within the silty clay to clay.

- Employ lightweight fill in the construction of the approach embankments to reduce the magnitude of primary consolidation settlement.

Some discussion regarding these options is provided in the following sections of this report. Based on discussions with MMM regarding the available time period (i.e. two full construction seasons from the beginning of bridge construction to the time of paving), the recommended option for mitigation of the settlement due to approach embankment loading is as follows:

- Build the approach embankments to full height and construct a 1 m surcharge; this will allow the surcharged embankment to remain within the constrained property limits at the site.
- Allow the surcharged embankment to remain in place for a minimum period of one year prior. Settlement monitoring (using settlement plates/rods) is recommended during this period to confirm the magnitude and rate of settlement.
- After one year and/or once at least 90 per cent of the consolidation settlement is achieved, remove the surcharge and pave the North Service Road approach embankments.

An Operational Constraint has been developed to address the preloading and surcharging for the approach embankments. This OC and a Non-Standard Special Provision to address the settlement monitoring are included in Appendix A of this report.

5.7.4 Mitigating Settlement by Surcharging

Following construction of the approach embankments, a surcharge could be placed to increase the settlement magnitude and potentially reduce the preload time. The feasibility of placing a surcharge has been evaluated relative to the available right-of-way space and any restrictions on allowable impacts to fisheries and wetlands.

In order to maintain the surcharged embankment within the right-of-way limits, a maximum 1 m high surcharge can be constructed atop the preload embankment. Settlement analyses indicate that 90 per cent of the 80 mm of predicted primary consolidation settlement will occur within approximately nine months to one year. This would limit the post-paving, primary consolidation settlement of the North Service Road to approximately 10 mm. Stability analyses indicate that the resulting 4 m to 5 m high embankments at this site will have an adequate factor of safety against global instability, provided that the 2H:1V side slopes are maintained.

5.7.5 Mitigating Settlement Using Wick Drains

Wick drains could be considered to accelerate the settlements and reduce the preloading time. The required duration for the preloading in this case will depend on the number and configuration of the wick drains, but it is roughly estimated that the majority of the settlement would occur in three months.

5.7.6 Mitigating Settlement Using Lightweight Fill

The amount of time-dependent settlement and the associated roadway maintenance may be reduced by employing lightweight fill materials below the pavement structure. Lightweight fill could be used in place of conventional earth fill to reduce the applied loading to below the pre-consolidation range.

Three types of lightweight fill are available for use:

- Extruded polystyrene (EPS) fill, with a bulk unit weight of less than 1 kN/m^3 ;
- Ultra-lightweight slag fill from Hamilton (Litex-143), with a bulk unit weight of about 11.5 kN/m^3 ; and
- Lightweight slag fill (Superior Slag) from Sault Ste. Marie or from Hamilton (Litex-149), with a bulk unit weight of about 14 kN/m^3 .

If, for example, EPS fill is adopted for construction of the approach embankments a 3 metre thick layer of EPS behind the abutments would reduce the applied load sufficiently to limit the post-paving, primary consolidation settlement of the North Service Road to approximately 25 mm.

5.8 Design and Construction Considerations

5.8.1 Installation of Driven Steel H-Piles

Due to the hardness of the bedrock and the relatively softer consistency of the overburden, there is potential for damage to the piles and/or deflection of the piles, particularly for battered piles, unless appropriate care is taken during driving of the piles. The pile driving criteria must be reviewed and accepted by the Quality Verification Engineer as appropriate to the site conditions and proposed pile type and configuration.

5.8.2 Formation of Bedrock Sockets

If rock sockets are required at this site (either for steel H-pile toe fixity, or if caissons are adopted for support of the bridge abutments), it is recommended that a Non-Standard Special Provision be included in the Contract Documents to warn the contractor of the following:

- The dolomitic limestone bedrock at the site is generally medium strong, and this will require socket formation using coring or churn drilling to advance the hole. Further, the dolomitic limestone and its sandstone interbeds have high silica contents, are considered to be rather abrasive, and could result in relatively high equipment wear.

5.8.3 Excavation and Groundwater / Surface Water Control

Based on the anticipated pile cap underside at approximately Elevation 124 m, excavations to approximately 1 m depth will be required at the abutment locations; these excavations will extend through the surficial organic materials and alluvium, and into the silty clay to clay. In addition, within the limits of the approach embankments, subexcavation of up to about 1 m of organic material will be required. It is noted that the clayey soils that will be exposed in the pile cap excavations, as well as in portions of the subexcavated areas within the approach embankment footprints, are very sensitive to disturbance from construction traffic. As discussed in Section 5.7, an Operational Constraint has been developed for inclusion in the Contract Documents to restrict travelling over the exposed clayey subgrade in order to minimize such disturbance; this Operational Constraint is included in Appendix A to this report.

Since the abutments and a portion of the approach embankments are located within the Lavallee Creek channel, temporary cofferdams will be required. A Non-Standard Special Provision (NSP) has been developed to "red-flag" this requirement in the Contract Documents; the NSP is included in Appendix A of this report. If steel sheet pile cofferdams are used, the sheeting should be keyed into the silty clay to clay deposit. The steel sheet pile system should be designed and constructed in accordance with MTO's Special Provision 539S01. The lateral movement of the system should meet Performance Level 3 as specified in SP 539S01.

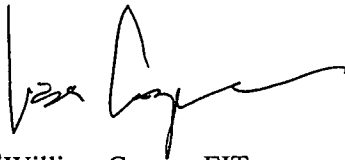
Outside of the creek channel, where standing water has been observed on the floodplain within the limits of the east and west approach embankments, temporary open-cut excavations may be adopted for subexcavation of the organic material. The excavation and embankment construction should be carried out in accordance with the provisions of OPSS 209.

Consideration should be given to scheduling construction to avoid excavation works at this site in the spring and fall periods when water levels are likely to be highest.

7.0 CLOSURE

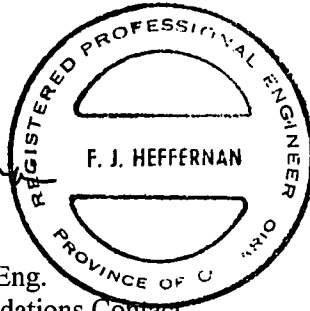
This Foundation Design Report was prepared by Mr. William Cavers, EIT, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and Senior Engineer with Golder, with technical input from Mr. Michael Cunningham, P.Eng., an Associate and Senior Engineer with Golder. Mr. Fintan Heffernan, a Designated MTO Contact for Golder, conducted an independent review of the report.


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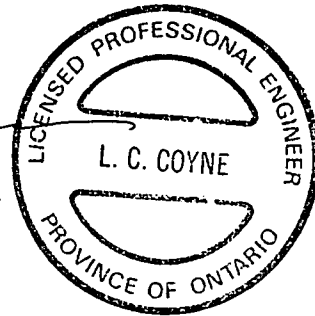

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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
NORTH SERVICE ROAD BRIDGE OVER LAVALLEE CREEK
W.P. 251-99-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on native silty clay to clay soils	<ul style="list-style-type: none"> Not considered feasible or cost-effective 	<ul style="list-style-type: none"> None 	<ul style="list-style-type: none"> Time and cost of settlement mitigation measures Even if mitigation measures adopted, additional secondary compression could result in settlement of shallow foundations 	<ul style="list-style-type: none"> Expected to be more expensive than deep foundation options due to requirement for settlement mitigation 	<ul style="list-style-type: none"> Even if mitigation measures in place for primary consolidation, shallow foundations will still be affected by secondary compression of silty clay to clay deposit Proposed bridge structure is settlement intolerant.
RSS Walls	<ul style="list-style-type: none"> Not considered feasible or cost-effective 	<ul style="list-style-type: none"> None 	<ul style="list-style-type: none"> Similar to spread footings, as noted above 	<ul style="list-style-type: none"> Similar to spread footings, as noted above 	<ul style="list-style-type: none"> Even if mitigation measures in place for primary consolidation, RSS wall will still be affected by secondary compression of clay
Steel H-pile foundations founded on or socketted into bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> High bearing resistance Negligible settlement Faster installation than caissons, if no rock socket is required for pile toe fixity 	<ul style="list-style-type: none"> If lateral / seismic loading conditions merit, pile toe may have to be socketted into medium strong bedrock, requiring coring/churn drilling If sockets required, temporary liner necessary Possibility of encountering cobbles or boulders during pile driving Care must be taken with driving of battered piles to ensure that the piles do not deflect along the bedrock surface 	<ul style="list-style-type: none"> Expected to be least expensive option, even if rock sockets are drilled (owing to the smaller socket diameter than that required for caissons) 	<ul style="list-style-type: none"> If required for pile toe fixity, socketting into the medium strong bedrock will be difficult and time-consuming
Caissons socketted nominally into bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> High bearing resistance Negligible settlement 	<ul style="list-style-type: none"> Temporary liners required to keep auger hole open Possibility of encountering cobbles or boulders during drilled shaft installation Coring or churn drilling will be required to form socket in medium strong bedrock 	<ul style="list-style-type: none"> Expected to be more expensive than steel H-pile option 	<ul style="list-style-type: none"> Socketting into the medium strong bedrock will be difficult and time-consuming

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

	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

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 test¹
 CIU consolidated isotropically undrained triaxial test with porewater pressure measurement¹
 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.

S:\FINALDATA\ABBREV\2000\LOFA-D00.DOC

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains > 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	

PROJECT 04-1111-007-5300		RECORD OF BOREHOLE No LC-1		1 OF 2	METRIC
W.P. 251-99-00	LOCATION N 5000799.1 ; E 335389.7	ORIGINATED BY W.C.			
DIST HWY 7	BOREHOLE TYPE Portable Drill	COMPILED BY J.M.			
DATUM Geodetic	DATE November 22, 2004	CHECKED BY L.C.C.			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
125.6	TOP OF WATER													
0.0	WATER													
0.2	Sandy organic SILT Very loose Dark brown Wet		1	SS	PM									
125.0														
0.6	Silty SAND Loose Grey Wet		2	SS	4									
124.5														
1.1	Silty CLAY (Weathered Crust) Very stiff Grey brown Wet		3	SS	5									
			4	SS	5									
123.1														
2.4	Silty CLAY Very stiff Grey Wet		5	SS	8									
			6	SS	7									
			7	SS	6									
			8	SS	7									
			9	SS	6									
			10	SS	5									
119.2			11	SS	4									
6.4	Silty CLAY, containing sand seams Very stiff Grey Wet		12	SS	4									0 0 37 63
118.9			13	SS	5									
6.7	Silty CLAY Very stiff Grey Wet		14	SS	7									
			15	SS	7									
			16	SS	7									

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+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 04-1111-007-5300		RECORD OF BOREHOLE No LC-1		2 OF 2	METRIC
W.P. 251-99-00		LOCATION N 5000799.1 ; E 335389.7		ORIGINATED BY W.C.	
DIST HWY 7		BOREHOLE TYPE Portable Drill		COMPILED BY J.M.	
DATUM Geodetic		DATE November 22, 2004		CHECKED BY L.C.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
								20	40	60	80						

	— CONTINUED FROM PREVIOUS PAGE —		17	SS	5											
	Silty CLAY Very stiff Grey Wet															
113.7																
11.8	Sandy SILT, some gravel, trace clay (TILL)															
113.4	Compact		18	SS	15											
12.1	Grey Wet DOLOMITIC LIMESTONE (BEDROCK) containing limestone and sandstone interbeds Fresh Grey Medium strong Thinly to medium-bedded Bedrock cored between 12.1m 15.2m depth. For bedrock coring details refer to Record of Drillhole LC-1.															
														</		

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PROJECT 04-1111-007-5300

RECORD OF BOREHOLE No LC-2

1 OF 2

METRIC

W.P. 251-99-00

LOCATION N 5000792.4 : E 335394.0

ORIGINATED BY W.C.

DIST _____ HWY 7

BOREHOLE TYPE Portable Drill

COMPILED BY J.M.

DATUM Geodetic

DATE December 3, 2004

CHECKED BY L.C.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	25 50 75		
125.6	TOP OF WATER												
0.0	WATER												
125.0							125						
0.6	Organic silty clay, trace sand (ALLUVIUM)												
124.7	Dark brown Wet		1	SS	2								
1.1	Silty SAND, trace shells Loose Grey Wet		2	SS	7		124						
	Silty CLAY Very stiff Grey Wet		3	SS	4								
			4	SS	5		123						
			5	SS	7								
			6	SS	6		122						
			7	SS	4								
			8	SS	5		121						
			9	SS	5		120						
119.2	Silty CLAY, containing sand seams Very stiff Grey Wet		10	SS	5		119						
6.4													
118.9	Silty CLAY Very stiff Grey Wet						118						
6.7			11	SS	5								
							117						
			12	SS	7		116						

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+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

+³, X³: Numbers refer to Sensitivity **O^{3%}** STRAIN AT FAILURE

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PROJECT 04-1111-007-5300		RECORD OF BOREHOLE No LC-3		1 OF 2	METRIC
W.P. 251-99-00	LOCATION N 5000799.6 ; E 335406.0			ORIGINATED BY W.C.	
DIST HWY 7	BOREHOLE TYPE Portable Drill			COMPILED BY J.M.	
DATUM Geodetic	DATE December 8, 2004			CHECKED BY L.C.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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		ELEVATION SCALE	SHEAR STRENGTH kPa					
125.7 0.0	TOP OF WATER WATER												
125.0 0.8	Silty sand, trace shells and clay (ALLUVIUM) Very loose Grey Wet		1	SS	PM								
124.5 1.2	Silty CLAY Stiff to very stiff Grey Wet		2	SS	6								
			3	SS	3								
			4	SS	4								
			5	SS	5								
			6	SS	4								
			7	SS	4								
			8	SS	5								
			9	SS	4								
			10	SS	4								
			11	SS	3								
			12	SS	4								
			13	SS	6								

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

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PROJECT 04-1111-007-5300			RECORD OF BOREHOLE No LC-3			2 OF 2			METRIC										
W.P. 251-99-00			LOCATION N 5000799.6 ; E 335406.0			ORIGINATED BY W.C.													
DIST HWY 7			BOREHOLE TYPE Portable Drill			COMPILED BY J.M.													
DATUM Geodetic			DATE December 8, 2004			CHECKED BY L.C.C.													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			γ			GR SA SI CL		
--- CONTINUED FROM PREVIOUS PAGE ---								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			W _P W W _L 25 50 75			kN/m ³					
114.2	Silty CLAY Stiff to very stiff Grey Wet		14	SS	5		115												
114.0	Sandy SILT, some gravel, trace clay (TILL) Dense Grey Wet		15	SS	>100		114												
11.7	DOLOMITIC LIMESTONE (BEDROCK) containing limestone and sandstone interbeds Fresh Grey Medium strong Thinly to medium-bedded Bedrock cored between 11.7m 14.8m depth. For bedrock coring details refer to Record of Drillhole LC-3.						113												
							112												
							111												
110.9	End of Borehole																		
14.8																			

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Continued Next Page

+³, X³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT <u>04-1111-007-5300</u>		RECORD OF BOREHOLE No LC-4		2 OF 2	METRIC
W.P. <u>251-99-00</u>	LOCATION <u>N 5000806.7, E 335401.5</u>	ORIGINATED BY <u>W.C.</u>			
DIST <u>HWY 7</u>	BOREHOLE TYPE <u>Portable Drill</u>	COMPILED BY <u>J.M.</u>			
DATUM <u>Geodetic</u>	DATE <u>December 15, 2004</u>	CHECKED BY <u>L.C.C.</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										25 50 75		
— CONTINUED FROM PREVIOUS PAGE —																				
	Silty CLAY Stiff to very stiff Grey Wet		13	SS	6															
114.1																				
11.6	Sandy SILT, some gravel, trace clay (TILL)		14	SS	>100															
113.8	Compact																			
12.0	Grey Wet																			
	DOLOMITIC LIMESTONE (BEDROCK)																			
113.2	Fresh																			
12.5	Slightly fractured Grey Medium strong Thinly to medium-bedded																			
	Bedrock cored between 12.0m 12.5m depth. For bedrock coring details refer to Record of Drillhole LC-3. End of Borehole																			

PROJECT <u>04-1111-007-5300</u>		RECORD OF BOREHOLE No LC-5		1 OF 1		METRIC	
W.P. <u>251-99-00</u>		LOCATION <u>N 5000815.8 ; E 335423.3</u>		ORIGINATED BY <u>W.C.</u>			
DIST <u> </u> HWY <u>7</u>		BOREHOLE TYPE <u>Portable Drill</u>		COMPILED BY <u>J.M.</u>			
DATUM <u>Geodetic</u>		DATE <u>December 17, 2004</u>		CHECKED BY <u>L.C.C.</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _p W W _L	WATER CONTENT (%)				
125.9	GROUND SURFACE													
0.0	TOPSOIL Wet													
125.5			1	50 DO	2									
0.5	Silty CLAY (Weathered Crust) Very stiff Grey brown Wet		2	50 DO	5									
			3	75 TP	PH									
123.2														
2.7	Silty CLAY Stiff Grey Wet		4	75 TP	PH									
			5	75 TP	PH									
			6	50 DO	1									
119.7														
6.3	End of Borehole													
	Note: Water level in standpipe at ground surface May 9, 2005													

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PROJECT 04-1111-007-5300

RECORD OF BOREHOLE No LC-6

1 OF 1

METRIC

W.P. 251-99-00

LOCATION N 5000784.2 E 335373.9

ORIGINATED BY W.C.

DIST HWY 7

BOREHOLE TYPE Portable Drill

COMPILED BY J.M.

DATUM Geodetic

DATE November 25, 2004

CHECKED BY L.C.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								20 40 60 80 100	20 40 60 80 100						25 50 75
125.4	GROUND SURFACE														
0.0	PEAT Dark brown Wet		1	SS	PM										
124.9							125								
0.6	Silty clay, some sand and organic matter (ALLUVIUM)														
124.5	Grey brown		2	SS	2										
0.9	Wet Silty SAND, trace shells Very loose to loose Grey Wet						124								
123.6			3	SS	5										
1.8	Silty CLAY (Weathered Crust) Very stiff Grey brown Wet		4	SS	6		123								
123.0															
2.4	Silty CLAY Very stiff Grey Wet		5	SS	5										
			6	SS	7		122								
			7	SS	8										
			8	SS	9		121								
			9	SS	9										
			10	SS	7		120								
119.0															
6.4	End of Borehole														
	Note: Water level in standpipe at 0.84m above ground surface May 9, 2005.														

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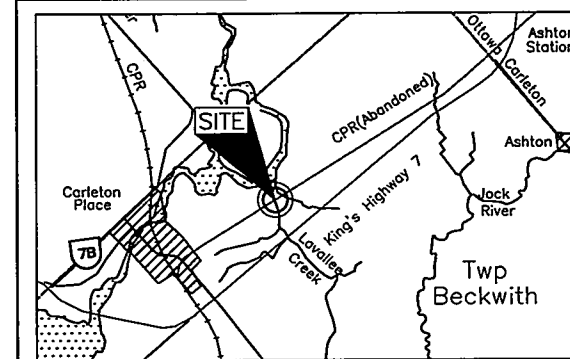
CONT No.
WP No. 251-99-00



SHEET
356








Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
1.6 0 1.6 km

LEGEND

- | | |
|--|--|
|  | Borehole — Current Golden Associates Ltd.
Investigation |
|  | Seal |
|  | Piezometer |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
| 100% | Rock Quality Designation (RQD) |
|  | WL of creek |
|  | Location of cross-section |

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
LC-1	125.57	5000799.13	335389.71
LC-2	125.64	5000792.38	335394.00
LC-3	125.72	5000799.65	335405.97
LC-4	125.72	5000806.65	335401.51
LC-5	125.93	5000815.79	335423.31
LC-6	125.42	5000784.23	335373.94

NOTES

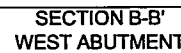
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The 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

Electronic general arrangement file provided by Marshall Macklin Monaghan
on July 11, 2005.

[illegible]



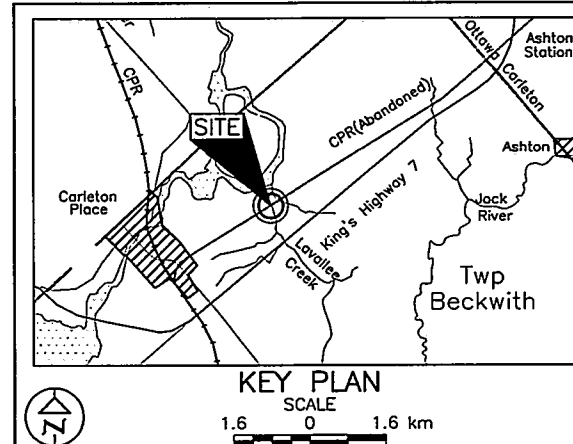
CONT No.
WP No. 251-99-00

LAVALLEE CREEK
SOIL STRATA





SHEET
357



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

-  Barehole — Current Golden Associates Ltd.
Investigation
-  Seal
-  Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
-  WL of creek

No.	ELEVATION	CO—ORDINATES	
		NORTHING	EASTING
LC—1	125.57	5000799.13	335389.7
LC—2	125.64	5000792.38	335394.0
LC—3	125.72	5000799.65	335405.9
LC—4	125.72	5000806.65	335401.5
LC—5	125.93	5000815.79	335423.3
LC—6	125.42	5000784.23	335373.9

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

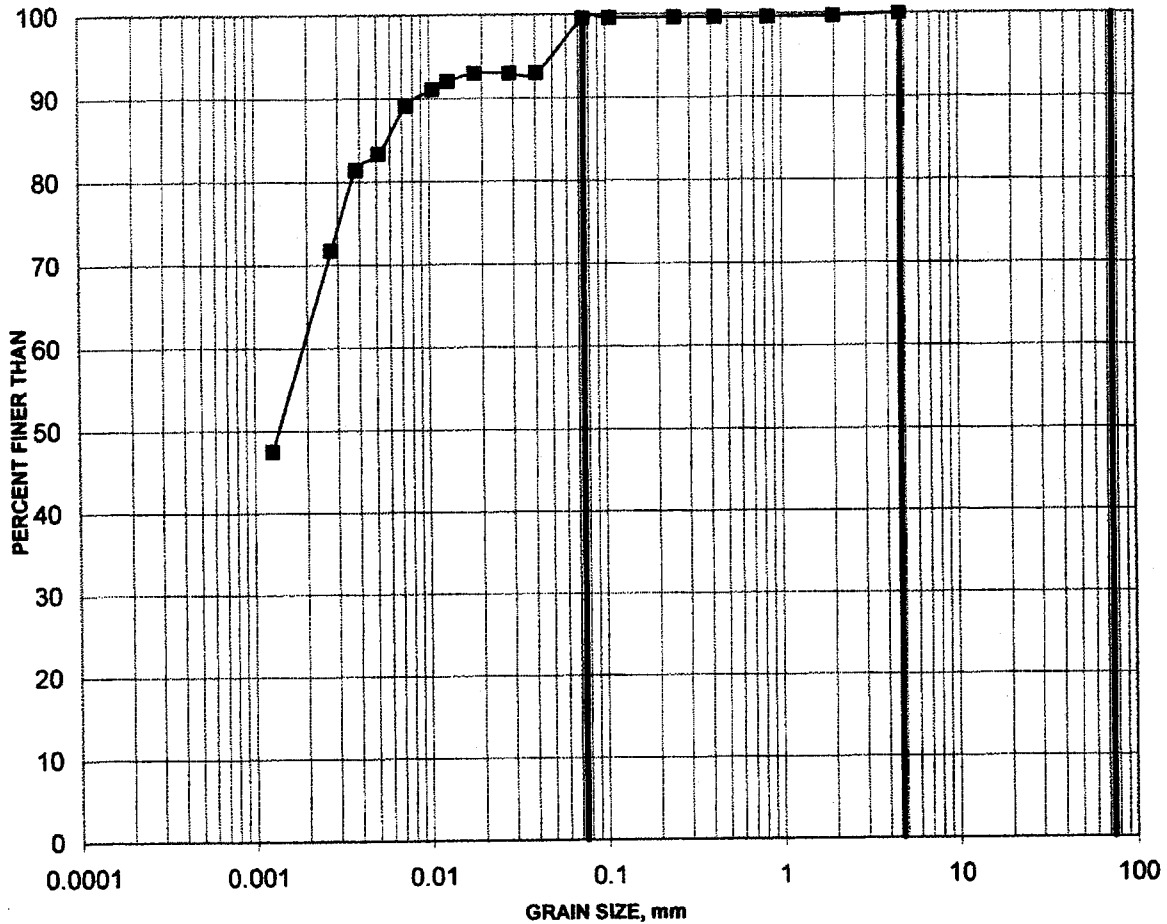
Electronic general arrangement file provided by Marshall Macklin Monaghan
on July 11, 2005.

NO.	DATE	BY	REVISION		
Geocres No.					
HWY. 7			PROJECT NO. 04-1111-007		DIST.
SUBM'D. WC		CHKD. MIC	DATE: JULY 2005		SITE:
DRAWN: JM/MSM		CHKD. WC	APPD. LCC		DWG. 2

GRAIN SIZE DISTRIBUTION TEST RESULT

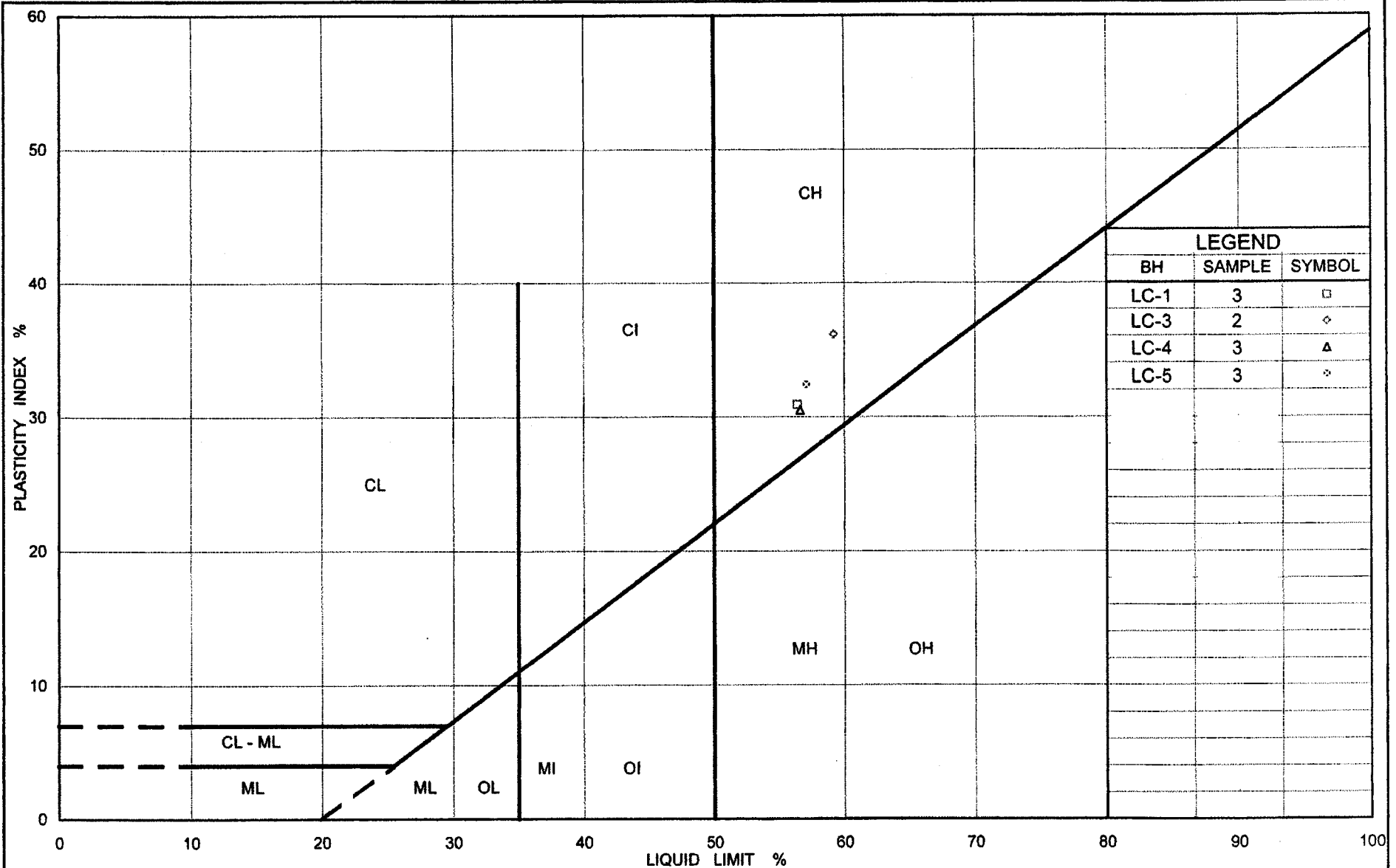
Clay (Weathered Crust)

FIGURE 1



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

Borehole	Sample	Depth (m)
LC-5	3	1.9



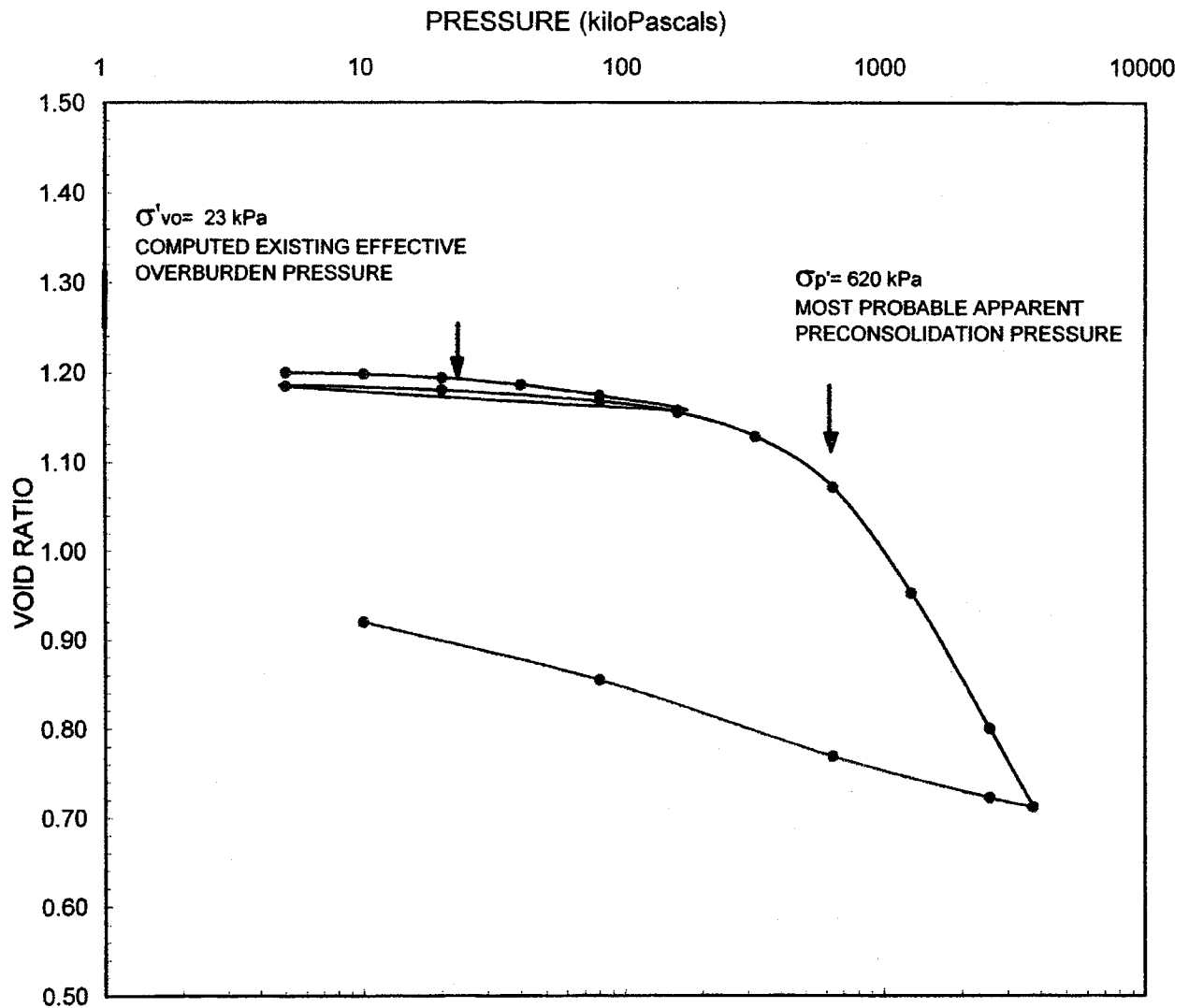
Ministry of Transportation

Ontario

PLASTICITY CHART CLAY (WEATHERED CRUST)

FIG No. 2

Project No. 04-1111-007



LEGEND

Borehole: LC-5	$w_l = 39\%$	$S_o = 92\%$
Sample: 3	$w_f = 33\%$	$C_c = 0.51$
Depth (m): 1.91	$w_l = 57\%$	$C_r = 0.017$
	$w_p = 25\%$	



SCALE	AS SHOWN
DATE	02/21/05
DESIGN	NA
CADD	NA
CHECK	EWK
REVIEW	MIC

CONSOLIDATION TEST RESULTS Clay (Weathered Crust)

FILE No.	Consolidation summary	
PROJECT No.	04-1111-007	REV. 0

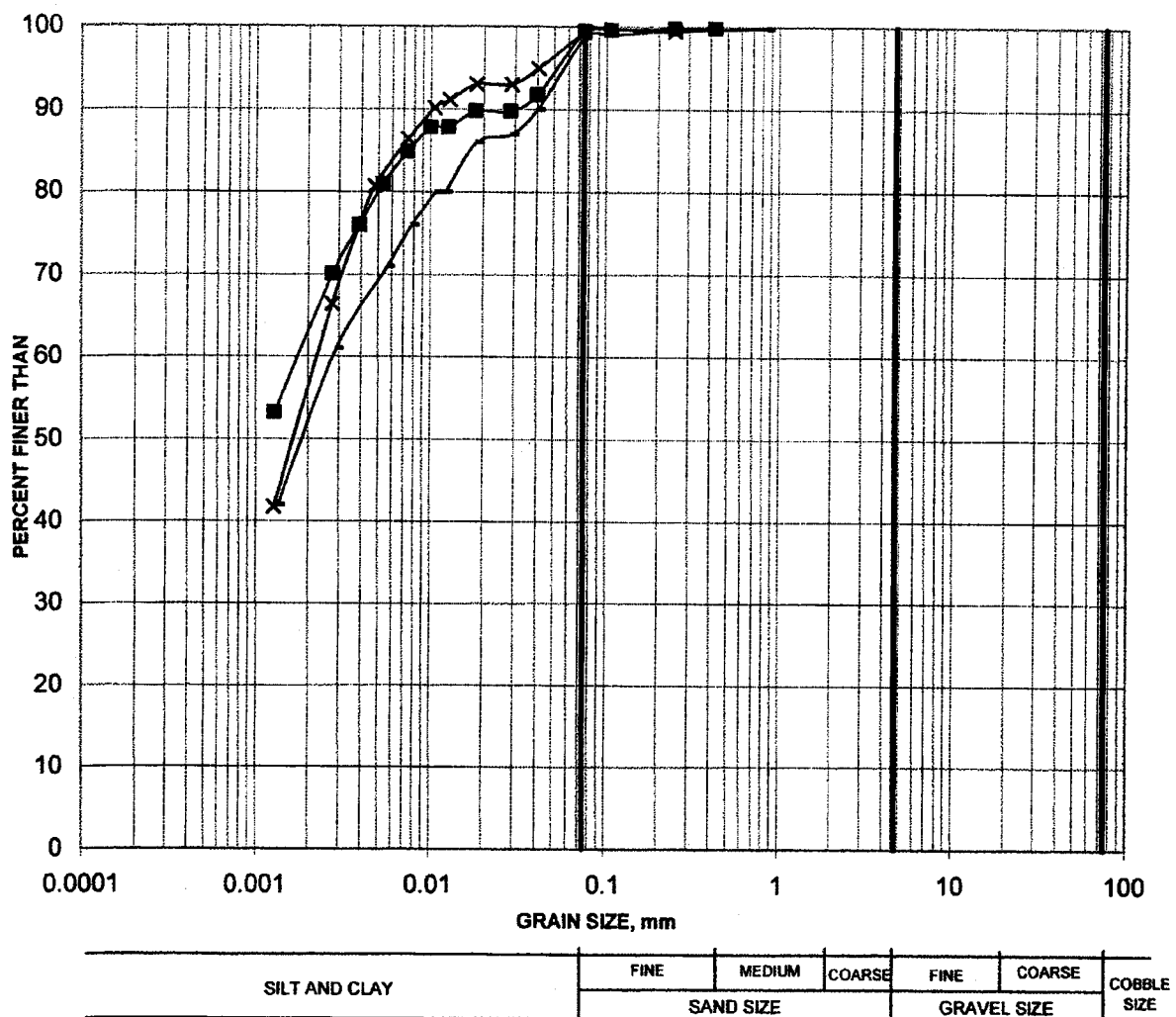
FIGURE

3

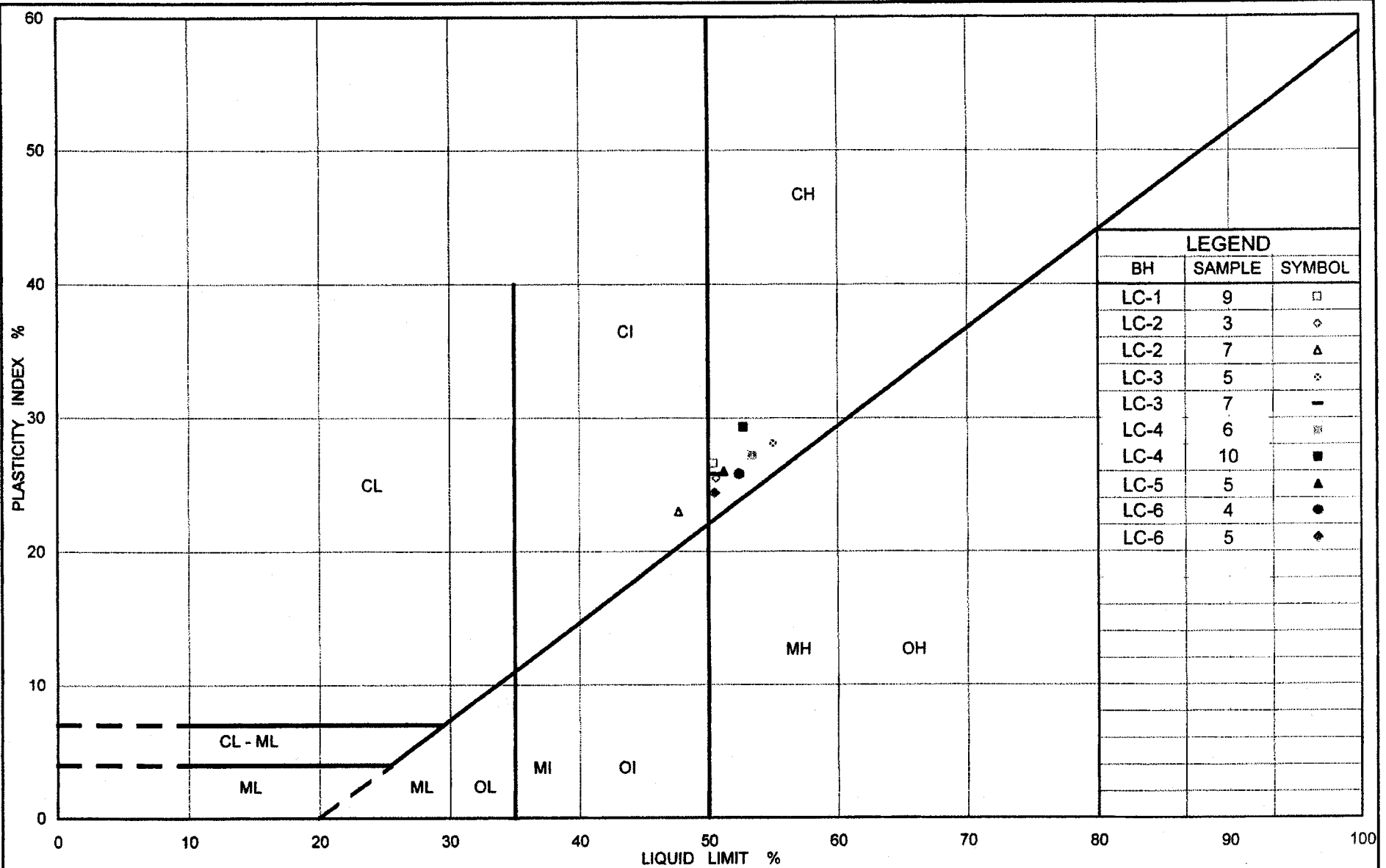
GRAIN SIZE DISTRIBUTION TEST RESULT

Unweathered Silty Clay to Clay

FIGURE 4



Borehole	Sample	Depth (m)
LC-1	11	6.4
LC-2	13	11.0
LC-5	5	4.0



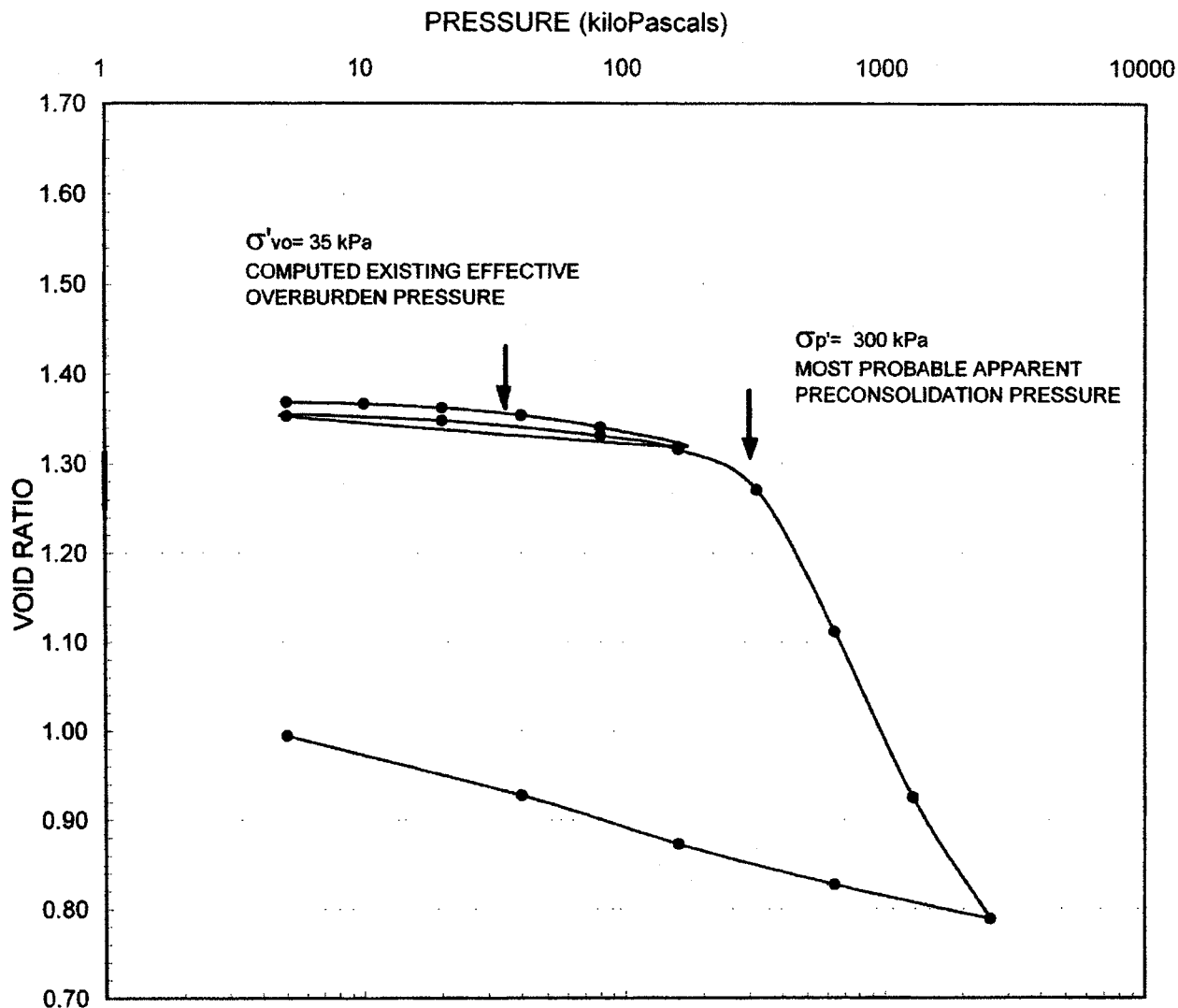
Ministry of Transportation

Ontario

PLASTICITY CHART UNWEATHERED SILTY CLAY TO CLAY

FIG No. 5

Project No. 04-1111-007



LEGEND

Borehole: LC-5	$w_i = 47\%$	$S_o = 97\%$
Sample: 5	$w_f = 36\%$	$C_c = 0.61$
Depth (m): 3.73	$w_l = 51\%$	$C_r = 0.023$
	$w_p = 25\%$	



SCALE	AS SHOWN
DATE	02/21/05
DESIGN	NA
CADD	NA

TITLE	CONSOLIDATION TEST RESULTS Unweathered Silty Clay to Clay
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FILE No.	Consolidation summary
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CHECK	EWK
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PROJECT No.	04-1111-007	REV.	0
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REVIEW	MIC
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FIGURE	6
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APPENDIX A

**OPERATIONAL CONSTRAINTS
AND
NON-STANDARD SPECIAL PROVISIONS**

OPERATIONAL CONSTRAINT

Special Provision

Preload/Surcharge Requirements - Approach Embankment at the North Service Road/Lavallee Creek Bridge

The following shall govern the construction of the surcharged approach embankments at the North Service Road bridge over Lavallee Creek. Note that settlement rods are required to be installed within the embankment preload/surcharge areas, in order to monitor the consolidation settlement of the foundation soils; the requirements for installation of the settlement rods are set out elsewhere in the Contract Documents.

The Contractor shall preload and surcharge both the west and east approach fill embankments to the North Service Road Bridge over Lavallee Creek, from Station 11+400 to Station 11+463 and Station 11+473 to Station 11+515, in Stage 1. The preload and 1 m thick surcharge shall be constructed with earth fill, as shown elsewhere in the contract drawings. From the preload limits indicated above, the fills shall be placed at 2:1 slopes both transversely and longitudinally.

Following completion of the surcharged embankments adjacent to the North Service Road Bridge over Lavallee Creek, the preloaded and surcharged fills shall remain in place for a minimum of twelve (12) months or until substantial (i.e. greater than 90 per cent) completion of settlement has occurred, as indicated by the monitoring instrumentation and confirmed by the Geotechnical Consultant and/or Contract Administrator, before paving of the North Service Road. North Service Road shall be paved in Stage 3.

The surcharge shall not be removed until approval is received from the Contract Administrator. Once approval is received, the surcharge shall be removed and the final pavement structure constructed.

SETTLEMENT RODS - Item No.

Non-Standard Special Provision

July 2005

GENERAL

Scope

This non-standard special provision contains the requirements for the supply and installation of settlement rods and survey benchmarks.

The purpose of the settlement rods is to monitor the progress of settlement under the preload embankments (approach embankments) adjacent to the new North Service Road bridge over Lavallee Creek. Settlement is measured by survey of the top of the rod with reference to stable, non-settling benchmarks.

The timing of final paving of the North Service Road will be controlled by the settlement rod readings. The completed preload embankment shall remain undisturbed until such time as the monitoring indicates that a sufficient degree of consolidation of the foundation soil has been achieved.

General Procedure

Rods shall be attached to a settlement plate installed at ground level prior to construction of the North Service Road approach embankments. As the embankment construction proceeds, the rods shall be extended above the new ground level.

Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate. As the embankment construction proceeds, the sleeves shall be extended above the new ground level.

A protective surround shall be extended with the rods and sleeves as the embankment construction proceeds.

Locations

The Contractor shall install the settlement rods at the approximate locations shown on the "Monitoring Instrument Location Plan and Typical Instrument Installation Detail" drawing contained elsewhere in this Contract.

Notification

The Contract Administrator shall be notified a minimum of 15 working days in advance of commencing the installation of instruments.

MATERIALS

General

The Contractor shall supply all materials and equipment required for the installation of the settlement rods. All instrumentation shall be and shall remain in proper working condition until the completion of the contract monitoring period.

Or Equal

The term "or equal" shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

Plate

The Contractor shall supply a steel plate with thickness of at least 6.35 mm. It shall be at least 0.5 m by 0.5 m in plan dimensions.

Rod

The Contractor shall supply a steel pipe with an outside diameter of at least 25 mm. The top of the rod shall be capped in such a way that a single survey point can be clearly identified and returned to.

Friction Reducing Sleeve

The Contractor shall supply a PVC pipe, friction reducing sleeve with an internal diameter slightly larger than the rod diameter.

Protective Surround

The Contractor shall supply a protective surround for the portion of the rod and sleeve within the embankment. The surround shall consist of a 300 mm diameter corrugated metal pipe (CMP) filled with compacted sand.

Monitoring Equipment

The elevation of the top of the settlement rods shall be surveyed by an experienced surveyor, retained by the Contractor, to provide the datum readings. The surveyor shall provide suitable equipment capable of surveying settlement rod elevations to an accuracy of ± 2 mm or better.

INSTALLATIONS**Survey Benchmarks**

The Contractor shall provide non-yielding, deep-seated survey benchmarks outside of the swamp subexcavation area, and shall establish the geodetic elevation of each such benchmark.

The number and locations of benchmarks shall be such that direct sighting is possible from all geotechnical instruments to at least one benchmark.

Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling holes for the installation of the deep-seated survey benchmarks. Any damage to underground utilities caused by the Contractor's work in this regard shall be repaired by the Contractor at no cost to the Contract Administrator.

Settlement Plate

The settlement plate shall be installed horizontally on undisturbed native soil, just below the existing ground.

The elevation of the base of the plate shall be surveyed by the Contractor before backfilling.

Rod

The rod shall be fixed to the centre of the plate and perpendicular to the plate.

The rod will be extended in 1.5 m increments as the embankment increases in height.

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

Friction Reducing Sleeve

The friction reducing sleeve should be extended in 1.5 m increments with the rods, over the entire length of the rod that is within the embankment fill.

Protective Surround

The CMP protective surround shall be extended in 1.5 m increments with the rods.

The settlement rod shall be in the centre of the CMP.

The annulus between the CMP and the friction reducing sleeve shall be filled with compacted sand to a level no higher than the top of the friction reducing sleeve.

Installation Details

The elevation, easting and northing of the centre of the base of the plate shall be surveyed by the Contractor.

The elevation, easting and northing of the top of the rod shall be surveyed by the Contractor.

The total distance from the base of the plate to the top of the rod shall be measured and recorded to an accuracy ± 2 mm or better.

Marking and Labelling

The location of all above-ground monitoring fixtures shall be made clearly visible to nearby traffic before, during and after embankment preload construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

Instruments shall be clearly labelled in the field, with each instrument having a unique identifier. The labelling shall remain legible for the entire period of monitoring.

Protection of Instruments

All instruments shall be adequately protected by the Contractor such that they are not damaged during construction operations. Any instrument damaged by the Contractor's work shall be immediately replaced at the Contractor's cost.

MONITORING

Personnel / Access

Data collection, interpretation and reporting shall be conducted by others, under the direction of the Contract Administrator.

The Contractor shall provide safe access and assistance to others reading the settlement rods.

Monitoring Program

The Contractor shall meet with the Contract Administrator and staff responsible for the ongoing monitoring immediately after installation of all of the instruments. At this meeting, the

Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments and all equipment to be supplied by the Contractor.

The relevant installation details required to be reported to the Contract Administrator include, but are not limited to, the following:

- Settlement rod and plate location, easting and northing;
- Elevation of plate and rod;
- Distance between base of plate and top of rod;
- Dates of installation and datum readings;
- Installation notes / sketches;
- Description of settlement rods, sleeve, plate.

Monitoring by others shall commence immediately following completion of installation of the instruments, and shall continue on a schedule to be determined by the Contract Administrator for a period of approximately two years following completion of the embankments.

PAYMENT

Measurement for Payment

Measurement of the item, "Settlement Rods", including all appurtenances, is by Plan Quantity as may be revised by adjusted Plan Quantity. The unit of measurement is each.

Basis of Payment

Payment at the contract price for the above item shall be full compensation for all labour, equipment and material to do the work, including the establishment of the required benchmarks and surveying required to establish the locations and initial elevations for each settlement rod and the required reporting.

OPERATIONAL CONSTRAINT

Special Provision

Protection of Subgrade Soils at Lavallee Creek Bridge Site

In order to limit disturbance to the sensitive clayey subgrade soils that will be exposed within the embankment footprints at the Lavallee Creek site, following stripping of any peat/organics:

- The Contractor shall minimize travelling over the clayey subgrade soils.
- The initial layers of embankment fill should be placed as soon as possible following the stripping of peat and organics.

UNWATERING

Special Provision

Unwatering at Lavallee Creek Bridge Site

Since the abutments and a portion of the approach embankments are located within the Lavallee Creek channel, temporary surface water cut-offs / cofferdams will be required during construction.

If steel sheetpile cofferdams are used, the sheeting should be keyed into the silty clay to clay deposit. The steel sheetpile system should be designed and constructed in accordance with MTO's Special Provision 539S01, and the lateral movement of the system should meet Performance Level 3 as specified in SP 539S01.