Golder Associates Ltd.

2390 Argentia Road
Mississauga, Ontario, Canada L5N 5Z7
Telephone: (905) 567-4444
Fax: (905) 567-6561

FOUNDATION INVESTIGATION AND DESIGN REPORT LAKE STREET UNDERPASS QEW WIDENING FROM HIGHWAY 406 TO GARDEN CITY SKYWAY ST. CATHARINES, ONTARIO G.W.P 607-00-00

Submitted to:

Morrison Hershfield
235 Yorkland Boulevard, Suite 600
Toronto, Ontario
M2J 1T1

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TABLE OF CONTENTS

SECTION **PAGE**

LIST OF TABLES

LIST OF DRAWINGS

LIST OF FIGURES

LIST OF APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
LAKE STREET UNDERPASS
QEW WIDENING FROM HIGHWAY 406
TO GARDEN CITY SKYWAY
ST CATHARINES, ONTARIO
G.W.P. 607-00-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the widening of the Queen Elizabeth Way (QEW) between Highway 406 and the Garden City Skyway in the City of St. Catharines, in the Region of Niagara. Foundation engineering services are required for the widening or replacement of five structures (Third Street overpass, Martindale Road underpass, Lake Street underpass, Geneva Street overpass, and Welland Avenue overpass), new retaining walls and noise barrier walls, culvert extensions, and high mast light poles.

This report addresses the foundation investigation carried out for the Lake Street underpass structure (MTO Structure Site No. 18-105).

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal for Agreement No. 2005-A-000564, issued in July 2002, and in Section 6.8 of MH's *Technical Proposal* for G.W.P. 607-00-00.

2.0 SITE DESCRIPTION

The proposed Lake Street underpass structure replacement is located along and immediately east of the existing Lake Street alignment, in the City of St. Catharines, in the Region of Niagara. The overall surface topography in the area is flat-lying to gently sloping toward Lake Ontario (to the north). The natural ground surface at the structure site is at about Elevation 96 m to 97 m, the existing QEW grade is at about Elevation 96.5 m, and the existing Lake Street grade in the immediate vicinity of the existing structure is at about Elevation 101.5 m to 102.2 m. The existing Lake Street approach embankments are about 5 m to 5.5 m high.

Vegetation within the existing interchange loops consists primarily of grasses, with some small shrubs and trees. Beyond the interchange area, commercial and residential properties are present.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the Lake Street underpass structure site between October 13 and December 5, 2004, at which time eleven boreholes (Boreholes 301 to 311) were advanced at the site using a track-mounted drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario.

The boreholes were advanced using either solid or hollow stem augers to depths ranging from 6.7 m to 30.9 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m and 1.5 m intervals of depth, using 50 mm outside diameter split-spoon samplers driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure. In situ vane shear strength testing was carried out where relatively soft cohesive soils were encountered, using an “N”-size vane. About 3 m of bedrock coring was carried out (between 27.4 m and 30.4 m depth) in Borehole 305, using an NQ-size rock core barrel.

The water level in the open boreholes was observed throughout the drilling operations, and standpipe piezometers were installed in Boreholes 303 and 310 to monitor the groundwater level at the site. The piezometers consist of a 1.5 m long slotted screen installed within a filter sand pack, then backfilled to ground surface using bentonite pellets; the details of the piezometer installation are depicted on the relevant borehole records. Where no piezometer was installed, the boreholes were backfilled to ground surface upon completion using bentonite pellets.

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 samples were identified in the field, placed in labelled containers and transported to Golder’s laboratory in Mississauga for further examination and 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. Oedometer testing was carried out on two soil samples, and point load index testing was carried out on selected samples of the bedrock core obtained in Borehole 305.

The borehole locations were measured by Golder relative to survey stakes placed at the site by MH surveyors, and to existing site features. The borehole locations (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.

Borehole Number	***Borehole Location***	***MTM NAD83 Northing (m)***	***MTM NAD83 Easting (m)***	***Ground Surface Elevation (m)***
301	North Approach	4,782,009.1	325,032.6	98.4
302	North Abutment	4,781,993.2	325,020.0	101.3
303	North Abutment	4,781,989.3	325,035.3	97.2
304	North Pier	4,781,963.0	325,029.0	96.8
305	North Pier	4,781,963.0	325,039.3	96.2
306	Center Pier	4,781,946.9	325,034.1	96.8
307	South Pier	4,781,924.6	325,033.3	97.2
308	South Pier	4,781,919.0	325,040.5	96.4
309	South Abutment	4,781,899.9	325,024.3	101.3
310	South Abutment	4,781,895.4	325,041.3	96.4
311	South Approach	4,781,875.5	325,039.1	99.8

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This area of the QEW lies within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario*[1] and *Urban Geology of Canadian Cities*[2].

The Iroquois Plain extends around the western shores of Lake Ontario; on the south side of the lake, in the St. Catharines area, the Plain is located between the present Lake Ontario shorebluffs and the foot of the Niagara Escarpment. The Plain is comprised of the flat to undulating lake bed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in the Iroquois Plain are typically comprised of glaciolacustrine clays and silts. However, in the St. Catharines area, surficial deposits of beach sand and gravel are present. The surficial sands, silts and clays are underlain by an extensive till deposit; portions of the till are considered to be “water-lain” (that is, formed by sediment rain-out either from a floating ice margin or from iceberg dumping), resulting in a predominantly massive, matrix-supported structure, as well as relatively thin sand to silt stringers or interlayers. This extensive till deposit may be underlain by or interlayered with a lower glaciolacustrine clay deposit, although this glaciolacustrine layer is absent in some portions of the Iroquois Plain in the St. Catharines area. Finally, the till and/or glaciolacustrine layer may be underlain by a lower till unit, that typically has increasing gravel content with proximity to the underlying bedrock (Menzies and Taylor, 1998).

The overburden soils are underlain by red shale bedrock of the Queenston Formation. This shale formation contains siltstone interlayers as well as “occasional patches of gypsum” (Menzies and Taylor, 1998).

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, eleven boreholes were advanced in the vicinity of the proposed foundation elements for the new underpass structure. The borehole locations and ground surface elevations are shown on Drawing 1.

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[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.

[2] J. Menzies and E.M. Taylor. “Urban Geology of St. Catharines-Niagara Falls, Region Niagara”. In *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory testing are given on the Record of Borehole sheets and on Figures 1 to 12. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

The subsurface conditions encountered at the site consist of fill (associated with the existing Lake Street and QEW embankments), overlying relatively thin surficial deposits of loose to compact silty sand to sand and silt and, at the north abutment/pier, soft to very stiff clayey silt to silty clay. These surficial deposits are underlain by an extensive till deposit, comprised predominantly of stiff to hard clayey silt, although a layer of silty sand to sand and silt till is present near the base of the deposit. The till also contains a weaker zone of soft to stiff glaciolacustrine clayey silt to silty clay that increases in thickness toward the south. The overburden soils are underlain by shale bedrock of the Queenston Formation, which was encountered below about 27 m depth in some of the boreholes.

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

4.2.1 Topsoil

Approximately 200 mm of topsoil was encountered immediately below the ground surface in Boreholes 301, 303, 310, and 311.

4.2.2 Fill

Fill, associated with the existing Lake Street or QEW embankments, was encountered immediately below the ground surface or topsoil in the all of the boreholes (except Borehole 305), as follows:

- In Boreholes 302 and 309, which were advanced through the existing Lake Street embankment, about 200 mm of asphalt was encountered overlying approximately 5 m of fill.
- In Boreholes 301, 303, 304, 307, 308, 310, and 311, which were advanced at the east toe of the existing embankments or immediately in front of the existing abutment foreslopes, about 0.4 m to 1.5 m of fill was encountered.
- In Borehole 306, which was advanced in the centre median of the QEW, about 0.8 m of fill was encountered.

The fill material varies in composition from sand and gravel, to sand containing trace to some silt, to silty sand; trace quantities of wood fragments, organics, or silty clay pockets were observed within the fill samples recovered from some of the boreholes. The measured Standard Penetration Test (SPT) "N" values range from 7 to 88 blows per 0.3 m of penetration, although the highest measured value of 88 blows per 0.3 m of penetration is likely attributable to the presence of gravel or cobbles within the fill. The SPT "N" values within the existing approach embankment fill in Boreholes 302 and 309 more typically vary from 13 to 50 blows per 0.3 m of penetration, indicative of a compact to dense relative density, while the SPT "N" values measured in other fill materials at the site vary from 7 to 22 blows per 0.3 m of penetration, indicative of a loose to compact relative density.

4.2.3 Surficial Silty Sand to Sand and Silt

A surficial layer of silty sand to sand and silt, about 0.7 m to 2.4 m thick, is present immediately below ground surface in Borehole 305 and underlies the existing fill in all other boreholes. This surficial deposit contains trace gravel, as well as trace organics. The results of two grain size distribution tests are shown on Figure 1.

The measured SPT "N" values within the surficial silty sand to sand and silt deposit range from 2 to 25 blows per 0.3 m of penetration, indicating that this deposit has a very loose to compact relative density.

4.2.4 Surficial Clayey Silt to Silty Clay

A surficial clayey silt to silty clay deposit was encountered in Boreholes 303 and 304, at the north abutment and north pier. This deposit is 1.6 m to 2.3 m thick as encountered in the boreholes, and is present immediately below the surficial silty sand to sand and silt deposit and atop the clayey silt till deposit. The clayey silt to silty clay contains trace sand; the result of one grain size distribution test is shown on Figure 2.

Atterberg limit tests were carried out on two samples of this deposit, and measured plastic limits of 18 and 20 per cent, liquid limits of 26 and 40 per cent, and plasticity indices of 9 and 20 per cent. The results, plotted on Figure 3, confirm that this deposit varies from a low plasticity clayey silt to an intermediate plasticity silty clay.

The measured SPT "N" values within the surficial clayey silt to silty clay typically range from 13 to 21 blows per 0.3 m of penetration, indicative of a stiff to very stiff consistency. However, an SPT "N" value of 2 blows per 0.3 m of penetration was measured in Borehole 304 at a depth of 2.3 m below ground surface (Elevation 94.5 m), indicating that the upper portion of the deposit in this borehole has a soft consistency.

4.2.5 Upper Clayey Silt Till

A 4.9 m to 11.9 m thick upper till deposit is present below the fill and surficial deposits. The surface of this upper till was encountered in all of the boreholes between 1.5 m and 7.6 m depth, at about Elevation 92.3 m to 97.9 m.

The glacial till consists of clayey silt containing some sand and trace gravel; the results of grain size distribution tests carried out on six samples of this material are shown on Figure 4. Atterberg limit testing was conducted on nineteen samples of the upper clayey silt till, and measured plastic limits of 10 to 16 per cent, liquid limits of 24 to 29 per cent, and plasticity indices of 9 to 17 per cent. The results, plotted on Figure 5, confirm that the material is a clayey silt of low plasticity.

The measured SPT “N” values range from 7 to 28 blows per 0.3 m of penetration. Field vane testing carried out within the less stiff portions of the upper till deposit measured undrained shear strengths ranging from about 40 kPa to greater than 100 kPa, and remoulded shear strengths of about 26 kPa to 72 kPa. Based on the measured ‘N’ values and undrained shear strengths, the upper clayey silt till has a generally stiff to very stiff consistency. The clayey silt till has a low sensitivity, based on calculated sensitivities of less than 2 from the field vane testing.

Oedometer testing was carried out on one specimen of the clayey silt till that was obtained from a thin-walled Shelby tube sample. The following table summarises the consolidation parameters for this soil type as interpreted from the oedometer test results as shown on Figures 6A to 6D.

Borehole/ Sample No.	Sample Depth/Elev.	Unit Wt. (kN/m^3)	σ_{vo}' (kPa)	σ_p' (kPa)	σ_p' - σ_{vo}' (kPa)	C_c	C_r	e_o	OCR
305 / 7	4.9 m / 91.3 m	21.1	70	235	165	0.141	0.033	0.514	3.3

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

Based on the oedometer test results and the measured SPT “N” values and undrained shear strengths, it is considered that the clayey silt till is a relatively overconsolidated deposit that may have been subjected to some degree of softening.

4.2.6 Clayey Silt to Silty Clay

A softer zone of glaciolacustrine clayey silt to silty clay was encountered in all of the deeper boreholes (Boreholes 302 to 310) below the upper clayey silt till. This deposit ranges from about 1.5 m to 9.9 m in thickness, and its surface was encountered between about 8.0 m and 17.8 m depth (Elevations 82.0 m to 88.4 m); the thickness of the deposit increases and its surface rises toward the south, as shown on the stratigraphic profile on Drawing 1.

The clayey silt to silty clay deposit contains trace sand and gravel, and is generally massive; however, layering of the clayey silt to silty clay, including thin sand interlayers, was observed in some of the recovered samples. The results of five grain size distribution tests are shown on Figure 7.

Atterberg limit testing was carried out on thirteen samples of this deposit, and measured plastic limits of 14 to 23 per cent, liquid limits of 23 to 47 per cent, and plasticity indices of 9 to 23 per cent. The results, plotted on Figure 8, confirm that the material grades from a low plasticity clayey silt to an intermediate plasticity silty clay. The natural water contents measured on selected samples of this deposit range from 21 to 40 per cent, typically near the liquid limit for the clayey silt to silty clay.

The SPT "N" values measured within this deposit range from 2 to 10 blows per 0.3 m of penetration. Field vane tests within the clayey silt to silty clay deposit measured undrained shear strengths ranging from 23 kPa to 66 kPa and remoulded shear strengths of about 12 kPa to 37 kPa. Based on the measured SPT "N" values and the undrained shear strengths, this clayey silt to silty clay deposit has a soft to stiff consistency.

Oedometer testing was carried out on one specimen of the clayey silt to silty clay that was obtained from a thin-walled Shelby tube sample. The following table summarises the consolidation parameters for this soil type as interpreted from the oedometer test results as shown on Figures 9A to 9D.

Borehole/ Sample No.	Sample Depth/Elev.	Unit Wt. (kN/m^3)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	C_c	C_r	e_o	OCR
308 / 13	14.0 m / 82.4 m	19.5	170	255	85	0.327	0.060	0.514	1.5

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

Based on the results of the field vane and oedometer testing, this deposit is slightly overconsolidated near the top, becoming normally consolidated near its base.

4.2.7 Lower Clayey Silt Till / Sand and Silt Till

A lower till deposit was encountered below the clayey silt to silty clay stratum in the deeper boreholes, with its surface between Elevations 78.0 m and 81.5 m (at about 15.3 m to 18.3 m depth below the natural ground surface at the site).. This till varies in thickness from about 5.2 m to 8.4 m in Boreholes 303 to 305, where it was fully penetrated; however, it is up to at least 9.2 m in thickness in Borehole 306, which was terminated within this deposit.

The majority of the lower till is composed of a clayey silt containing trace to some sand and trace gravel, shale and limestone fragments; however, a 1.0 m to 2.9 m thick zone of sand and silt containing some gravel and trace clay is present within this deposit, as shown on the borehole records and on the interpreted stratigraphic sections on Drawings 1 and 2. Grain size distribution test results for two samples of the sand and silt till are shown on Figure 10.

Atterberg limit testing was carried out on five selected samples of the lower clayey silt till, and measured plastic limits of 12 to 13 per cent, liquid limits of 18 to 21 per cent, and plasticity indices of 5 to 8 per cent. The results, plotted on Figure 11, confirm that this material is a clayey silt of low plasticity.

The measured SPT “N” values within the upper 2 m to 3 m of the lower till, immediately below the glaciolacustrine clayey silt to silty clay deposit, range from 11 to 28 blows per 0.3 m of penetration; this portion of the till therefore has a stiff to very stiff consistency. The measured SPT “N” values within the sand and silt till zone vary from 53 to greater than 100 blows per 0.3 m of penetration, except in Borehole 306 where an SPT “N” value of 14 blows per 0.3 m of penetration was measured; based on these results, the sand and silt till zone of the lower till has a compact to very dense, though generally very dense, relative density. The measured SPT “N” values in the clayey silt till below the sand and silt till zone range from 59 to greater than 100 blows per 0.3 m of penetration, indicative of a hard consistency in this portion of the till.

4.2.8 Residual Soil

A residual soil is present in Boreholes 303 to 305, below the lower till and atop the bedrock; its surface was encountered between

A layer of clayey silt containing trace to some sand (including thin sand seams) and trace gravel, shale and limestone fragments is present below the lower till and immediately above the bedrock in Boreholes 303 to 305. The result of one grain size distribution test on a sample of this material is shown on Figure 12. Based on its composition and its position directly atop the shale bedrock, this deposit is interpreted to be a residual soil derived from weathering of the underlying bedrock.

This residual soil layer is between 1.2 m and 2.6 m in thickness, and its surface was encountered between Elevations 70.6 m and 73.8 m (at about 24.6 m to 26.2 m depth) in the boreholes.

The measured SPT "N" values within the residual soil range from 48 to greater than 100 blows per 0.3 m of penetration, indicating that this deposit has a hard consistency.

4.2.9 Bedrock

Bedrock was encountered at the proposed north pier in Borehole 304, where it was confirmed by split-spoon sampling, and in Borehole 305, where it was confirmed by NQ-coring. The bedrock surface was encountered at Elevations 69.4 m and 68.8 m in Boreholes 304 and 305, respectively (at 27.4 m depth in both boreholes).

The bedrock encountered at the site consists of slightly weathered to fresh, very weak to medium strong, thinly bedded, calcareous red shale of the Queenston Formation, containing bands of grey shale and thin (5 mm to 15 mm thick) seams of clay (as noted on the drillhole record). Although not present in the bedrock core recovered from Borehole 305, interlayers of strong to very strong limestone and siltstone are anticipated to be present within the Queenston Formation shale bedrock.

Diametral point load testing was carried out on selected samples of the shale bedrock obtained from Borehole 305. The correlated uniaxial compressive strength of the shale samples varies from 15 MPa to 49 MPa, as shown in the table below. These results indicate that the tested shale samples are weak to medium strong; however, based on detailed logging of the bedrock core, the recovered shale is considered to vary from very weak to medium strong.

Borehole No.	***Rock Type***	***Sample Depth (m)***	***Sample Elevation (m)***	***Approx. Unconfined Compressive Strength***
305	Shale	28.2	68.0	17 MPa
	Shale	28.5	67.7	15 MPa
	Shale	29.7	66.5	49 MPa

NOTE: Approximate unconfined compressive strength determined using ISRM correlation ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech., Vol. 22, No. 2, 1985, pp. 51-60).

4.3 Groundwater Conditions

A standpipe piezometer was installed within the lower till deposit in each of Boreholes 303 and 310; the piezometer installation details are shown on the borehole records.

The piezometric level associated with the deeper soil deposits is between Elevations 94.3 m and 95.6 m (about 1.6 m to 2.1 m below ground surface), as measured in the piezometers. The measured water levels, which are summarized in the following table, indicate seasonal fluctuations of approximately 0.2 m to 0.4 m.

Borehole No.	***November 26, 2004***		***May 13, 2005***		***December 6, 2005***	
	Depth	***Elevation***	***Depth***	***Elevation***	***Depth***	***Elevation***
303	1.8 m	95.4 m	1.6 m	95.6 m	1.6 m	95.6 m
310	2.1 m	94.3 m	1.7 m	94.7 m	1.9 m	94.5 m

In addition, “perched” groundwater should be expected within the surficial silty sand to sand and silt deposit, especially during the wetter months of the year.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Shannon Palmer, EIT, and reviewed by Lisa Coyne, P.Eng., an Associate and Senior Engineer of Golder, with technical input from Murty Devata, P.Eng., a Specialist Foundations Consultant with Golder. Fin Heffernan, P.Eng., a Designated MTO Contact for Golder, carried out an independent review of the report.

GOLDER ASSOCIATES LTD.

Shannon L. Palmer, EIT
Geotechnical Group

Lisa C. Coyne, P.Eng.
Associate

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

SLP/LCC/MSD/FJH/lcc
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PART B

FOUNDATION DESIGN REPORT
LAKE STREET UNDERPASS
QEW WIDENING FROM HIGHWAY 406
TO GARDEN CITY SKYWAY
ST CATHARINES, ONTARIO
G.W.P. 607-00-00

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for the design of the proposed Lake Street underpass structure replacement. 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 carry out design of 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.

6.2 Bridge Foundation Options

The proposed Lake Street underpass will be a four-span structure, supporting four lanes of traffic with sidewalks along each side. The finished grade of the QEW will be maintained at approximately Elevation 96.5 m, similar to the natural ground surface, which varies from about Elevation 96 m to 97 m. The new Lake Street grade at the structure and immediate approach embankments will vary from approximately Elevation 102.5 m to 103.7 m, about 1.5 m higher than the existing Lake Street grade of Elevation 101 m to 102.2 m. The new approach embankments will therefore require up to about 6 m to 6.5 m of fill east of the existing Lake Street alignment, where the widened embankments will be constructed, and about 1.5 m of fill atop the existing approach embankments to achieve the proposed grade.

It is understood that the proposed replacement underpass is to be constructed in stages in order to maintain two operational lanes on Lake Street at all times. The east half of the new structure will be constructed first, immediately east of the existing structure, after which the existing structure will be demolished and the west half of the new structure constructed.

The main concern for design and construction of the new Lake Street underpass is consolidation settlement within the 1.5 m to 9.9 m thick, compressible clayey silt to silty clay stratum that underlies the site, and the impacts that this settlement will have on the performance of the structure and the approach embankments.

Shallow foundations are not feasible for support of the new underpass structure at this site, since the geotechnical resistance at Serviceability Limit States (SLS) for the abutment and pier footings would have to be restricted to less than 150 kPa in order to limit the settlement to less than 25 mm. Steel H-piles driven to found within the 100-blow lower till or bedrock represent the most feasible and cost-effective foundation solution for this structure. As an alternative to steel H-pile foundations, caissons supported on the 100-blow till or on the bedrock could also be used for support of the new structure.

As discussed further in Section 6.6, up to about 100 mm of primary consolidation settlement will occur within this deposit as a result of the widened approach embankment loading (assuming the use of a 6 m to 6.5 m height of conventional earth or granular fill) at this site; it is estimated that it will take approximately two to three years to achieve 90 per cent of this primary consolidation settlement. The design of deep foundations at this site will therefore have to accommodate the downdrag loads generated by consolidation of the clayey silt to silty clay deposit under the approach embankment loading, unless settlement mitigation measures are adopted. In this regard, the use of lightweight, ultra-lightweight or EPS fill for construction of the approach embankments is one feasible settlement mitigation measure. Alternatively, if there is time (three to six months) in the staging schedule to allow preloading/surcharging with the use of wick drains installed to the base of the normally or near-normally consolidated portion of the clayey silt to silty clay deposit at about Elevation 78 m, this settlement mitigation measure could be cost-effective. In this option, consideration could be given to providing a wedge of EPS fill behind the abutments to lessen the settlements adjacent to the abutments and to eliminate the downdrag forces on the piles.

Geotechnical recommendations for the design of foundations for the bridge piers, 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.

6.3 Steel H-Pile Foundations

The abutments and piers may be supported on steel H-piles driven to found within the "100-blow" lower till deposit. The surface of the 100-blow till was encountered between Elevations 74 m and 76.5 m in the boreholes, as summarized in the table below. For design, the following pile tip levels may be assumed based on 1.5 m of penetration into the 100-blow till:

Foundation Element	***Relevant Boreholes***	***Estimated Elevation Of 100-Blow Till***	***Estimated Pile Tip Elevation***
North Abutment	302, 303	74 m to 76 m	72.5 m to 74.5 m
North Pier	304, 305	75 m to 75.5 m	73.5 m to 74 m
Centre Pier	306	75.5 m	74 m
South Pier	307, 308	75.5 m to 76.5 m	74 m to 75 m
South Abutment	309, 310	74 m to 76.5 m	72.5 m to 75 m

In the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the till deposits at this site; a sample Non-Standard Special Provision (NSSP) is provided in Appendix A to warn the contractor of the presence of cobbles and boulders within the till that could affect the installation of the piles. Steel H-piles should be stiffened with MTO flange plates for protection during driving, in accordance with OPSS 903.07.05.04.

6.3.1 Axial Geotechnical Resistance

For HP 310x110 piles driven to practical refusal within the 100-blow lower clayey silt till, a factored axial resistance at Ultimate Limit States (ULS) of 1,800 kN may be assumed for design. The axial geotechnical resistance at Serviceability Limit States (SLS) may be taken as 1,600 kN.

Pile installation should be in accordance with MTO's Special Provision SP903S01. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. The criteria must therefore be established at the time of construction after the piling equipment is known. For piles driven into the 100-blow till, the following note is considered appropriate for the design and site conditions assuming a resistance factor of 0.5 is applied to the use of the Hiley formula:

> *"Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,600 kN per pile."*

6.3.2 Downdrag Load (Negative Skin Friction)

Assuming the use of conventional earth or granular fill, the embankment loading will cause consolidation settlement of the soft to stiff, clayey silt to silty clay deposit at this site (as discussed further in Section 6.6). Negative skin friction or downdrag loads will need to be taken into account in the design of the piles supporting the abutments, unless measures to eliminate downdrag loads are adopted as discussed at the end of this section.

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 clayey silt to 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 clayey silt to 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 500 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.

Downdrag loads could be eliminated with the use of EPS fill as backfill behind the abutments; further discussion on the use of lightweight fill is provided in Section 6.6. Alternatively, consideration could be given to the use of bitumen coating on the piles to eliminate the downdrag loads; however, the use of bitumen coating increases the pile costs by approximately 20 to 45 per cent depending on the size of the job; for the QEW widening project, it is estimated that the cost increase would be closer to the upper limit.

6.3.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. 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 is determined based on the equations given below:

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where
k_h is the coefficient of horizontal subgrade reaction (MPa/m);
n_h is the constant of subgrade reaction (MPa/m);
z is the depth (m); and
B is the pile diameter (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

where
k_h is the coefficient of horizontal subgrade reaction (kPa/m);
s_u is the undrained shear strength of the soil (kPa); and
B is the pile diameter (m).

The following ranges for the value of n_h and s_u may be assumed in the structural analyses. Approximate elevation intervals are given in this table for each deposit at each of the foundation elements; however, the deposit boundaries vary across each of the foundation elements and reference should also be made to the interpreted stratigraphic sections on Drawings 1 and 2.

Soil Unit	***Elevation***	n_h	s_u
North abutment:			
Loose to compact surficial silty sand	Above 95 m	10 MPa/m	–
Very stiff surficial clayey silt and stiff to very stiff upper clayey silt till	95 m to 83.5 m	–	125 kPa
Soft to stiff clayey silt to silty clay	83.5 m to 78.5 m	–	40 kPa
Very stiff lower clayey silt till	78.5 m – 75 m	–	150 kPa
Hard/very dense (100-blow) lower till	Below 75 m	–	500 kPa
North pier:			
Very loose to compact surficial silty sand	Above 94.5 m	10 MPa/m	–
Soft to very stiff surficial clayey silt and stiff to very stiff upper clayey silt till	94.5 m to 82 m	–	125 kPa
Soft to stiff clayey silt to silty clay	82 m to 78 m	–	40 kPa
Very stiff lower clayey silt till	78 m to 75 m	–	150 kPa
Hard/very dense (100-blow) lower till	Below 75 m	–	500 kPa
Centre pier:			
Very loose to compact surficial silty sand	Above 94.5 m	10 MPa/m	–
Stiff to very stiff upper clayey silt till	94.5 m to 85 m	–	125 kPa
Soft to stiff clayey silt to silty clay	85 m to 81 m	–	40 kPa
Very stiff lower clayey silt till	81 m to 75.5 m	–	150 kPa
Very dense/hard (100-blow) lower till	Below 75.5 m	–	500 kPa
South pier:			
Loose to compact surficial silty sand	Above 94.5 m	10 MPa/m	–
Stiff to very stiff upper clayey silt till	94.5 m to 87 m	–	125 kPa
Soft to stiff clayey silt to silty clay	87 m to 79.5 m	–	40 kPa
Very stiff lower clayey silt till	79.5 m to 76 m	–	150 kPa
Very dense/hard (100-blow) lower till	Below 76 m	–	500 kPa
South abutment:			
Compact surficial silty sand	Above 94 m	10 MPa/m	–
Stiff to very stiff upper clayey silt till	94 m to 88 m	–	125 kPa
Soft to stiff clayey silt to silty clay	88 m to 78 m	–	40 kPa
Very stiff lower clayey silt till	78 m to 75 m	–	150 kPa
Hard/very dense (100-blow) lower till	Below 75 m	–	500 kPa

A maximum factored lateral resistance of 200 kN at ULS, and a maximum lateral resistance of 110 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310x110 piles. These values are based on the “Assessed Horizontal Passive Resistance Values for Various Pile Types” provided in Table C6.8.7.1(a) of the *Commentary* to the *CHBDC*.

Group action for lateral loading should also 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 horizontal subgrade reaction in the direction of loading by a reduction factor, R, as follows:

Pile Spacing in Direction of Loading ***(d = Pile Diameter)***	***Subgrade Reaction Reduction Factor***
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

6.3.4 Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

6.4 Caissons

Consideration could be given to the use of caissons founded within the 100-blow till for support of the new Lake Street underpass. The following design base elevations may be used at the abutments and piers, based on approximately 2 m of embedment within the 100-blow till:

Foundation Element	***Relevant Boreholes***	***Estimated Elevation Of 100-Blow Till***	***Estimated Caisson Base Elevation***
North Abutment	302, 303	74 m to 76 m	72 m to 74 m
North Pier	304, 305	75 m to 75.5 m	73 m to 73.5 m
Centre Pier	306	75.5 m	73.5 m
South Pier	307, 308	75.5 m to 76.5 m	73.5 m to 74.5 m
South Abutment	309, 310	74 m to 76.5 m	72 m to 74.5 m

The lower till should be expected to contain cobbles and boulders which may pose difficulties in advancing caissons and liners.

It is also noted that basal heave could occur where more pervious sand and silt till soils are present at/near the caisson base. If caisson foundations are adopted for this site, temporary or permanent caisson liners would be required to support the soils during construction and permit inspection and cleaning of the caisson base in order to achieve the recommended geotechnical resistances. However, construction experience in similar soil conditions has demonstrated that temporary liners can be difficult to withdraw, owing to the length of the liners and the hard/very dense nature of the 100-blow material, and that such difficulties can result in "necking" of the caisson. As such, permanent liners would be preferred over temporary liners for the construction of the caissons in these soils conditions.

If caisson foundations are adopted for this site, an NSSP will be developed to address the potential presence of cobbles and boulders and the need for control of the ground and groundwater during caisson construction.

6.4.1 Axial Geotechnical Resistance

The caissons will derive the majority of their capacity from base resistance, although some shaft friction has also been taken into account based on "socketting" approximately 2 m into the 100-blow till. Using the design elevations given above, and assuming that all caisson excavations are inspected prior to pouring concrete, the factored axial geotechnical resistance at ULS and the axial resistance at SLS are given below for various caisson diameters:

Caisson Diameter	***Axial Geotechnical Resistance***	
	ULS	**SLS**
0.9 m	3,600 kN	2,900 kN
1.2 m	6,000 kN	4,800 kN
1.5 m	9,000 kN	7,200 kN

If permanent liners are used for construction of the caissons, the geotechnical resistances provided above would have to be reduced to neglect the component of shaft friction over the "socket" within the 100-blow soil.

6.4.2 Downdrag Load (Negative Skin Friction)

The estimated unfactored downdrag load acting on the caissons at the north and south abutments may be taken as shown in the table below:

Caisson Diameter	***Unfactored Downdrag Load***
0.9 m	1,250 kN
1.2 m	1,650 kN
1.5 m	2,250 kN

Other requirements for structural design with respect to downdrag load on the caissons are discussed in Section 6.3.2.

The downdrag loads provided above are relatively large and may render the use of caisson foundations impractical. If caisson foundations are considered advantageous for other reasons, it may be feasible to construct the caissons with a permanent lining and bentonite slurry "slip" layer; however, such construction may also prove costly. Recommendations for this type of construction can be developed if caisson foundations are determined to be the preferred option from a structural perspective.

6.4.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons (based on subgrade reaction theory), and the reductions due to group effects, may be determined as per Section 6.3.3.

A maximum factored lateral resistance of 400 kN at ULS, and a maximum lateral resistance of 250 kN at SLS (for 10 mm of horizontal deflection at pile cap level) are recommended for 0.9 m diameter caissons. Values for alternative caisson diameters can be provided if larger diameter caisson foundations are adopted at this site.

6.4.4 Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

6.5 Lateral Earth Pressures for Design

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

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular ‘A’ or Granular ‘B’ Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be placed and compacted in accordance with MTO’s Special Provision 105S10. 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 1.2 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC)*.

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

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.

6.6 Approach Embankment Design and Construction

The widening and realignment of Lake Street will require placement of up to about 6 m to 6.5 m of fill for the new eastern portion of the north and south approaches, and about 1.5 m of fill atop the existing embankments.

6.6.1 Subgrade Preparation and Embankment Construction

In order to minimize differential settlement between the new and widened portions of the approach embankments, it is recommended that all topsoil and softened / loosened soils be stripped from below the widened approach embankment areas. All subgrade soils should be proof-rolled prior to fill placement in accordance with OPSS 206. Embankment fill should be placed and compacted in accordance with MTO's Special Provision 105S10.

In order to minimize differential settlement between the widened portions of the approach embankments due to settlement of the fill itself, the use of granular fill is recommended over the use of cohesive fill, since the majority of settlement of granular fills will occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction. The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.6.2 Approach Embankment Stability

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the 6 m to 6.5 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.

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

Soil Type	*Bulk Unit Weight*	*Effective Angle of Friction*	*Undrained Shear Strength*
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m^3	32º to 35º	–
Surficial silty sand	20 kN/m^3	30º	–
Surficial clayey silt	20 kN/m^3	30º	50 kPa
Upper clayey silt till	21 kN/m^3	32º	100 kPa
Clayey silt to silty clay	19.5 kN/m^3	32º	40 kPa
Lower clayey silt till / sand and silt till	21 kN/m^3	35º	–

The undrained shear strengths used in the analyses, as summarized in the table above, are based on the design strength profiles provided on Figure 13. This figure plots the undrained shear strength data from in situ vane testing as well as shear strengths calculated from the oedometer test results based on the formula $s_u = 0.22 \times \sigma_p'$ (in kPa).

6.6.3 Approach Embankment Settlement

Settlement of the approach embankment foundation soils will occur due to consolidation of the glaciolacustrine clayey silt to silty clay stratum that is up to about 10 m thick as encountered at the borehole locations; some settlement of the upper clayey silt till will also occur under the approach embankment loading.

In order to estimate the magnitude and rate of settlement, analyses were carried out using the commercially-available computer program *Unisettle*, in conjunction with hand calculations. The settlement of the founding soils has been estimated using the consolidation parameters and elastic deformation moduli given in the table below, based on the oedometer test results and correlations with the undrained shear strength, Atterberg limits, and SPT "N" values:

Soil Unit	***Bulk Unit Weight***	***Preconsolidation Pressure***	***Cc***	***Cr***	***Elastic Modulus***
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m^3	–	–	–	–
Loose to compact surficial silty sand	19 kN/m^3	–	–	–	15 MPa
Upper clayey silt till	21.1 kN/m^3	See Figure 14	0.15	0.025	–
Clayey silt to silty clay	19.5 kN/m^3	See Figure 14	0.30	0.025	–
Hard / very dense lower till	22 kN/m^3	–	–	–	100 MPa

The total predicted settlement assuming the use of conventional earth fill or granular fill for the embankment widening is outlined below, along with the total predicted settlement for the use of two alternative lightweight fills: ultra-lightweight slag fill and EPS fill.

- **Conventional earth/granular fill for construction of approach embankment widening:** It is estimated that the embankment loading will exceed the preconsolidation pressure at the base of the upper clayey silt till and/or within the clayey silt to silty clay deposit, depending on the embankment load applied and the depth of the interface between these two clayey deposits. The settlement of the approach embankments as a consequence of the post-construction consolidation of the foundation soils is estimated to be up to about 100 mm and 125 mm under the new eastern portion of the north and south approaches, respectively. At the western shoulder of the Lake Street embankment, where about 1.5 m of fill will be placed on top of the existing embankment fill, less than 25 mm of consolidation settlement will occur. It is expected to take two to three years for 90 per cent of the consolidation settlement to occur under the widened embankment loading.

- **Ultra-lightweight slag fill for construction of approach embankment widening:** In order to reduce the magnitude of post-construction settlement under the new approach embankments, lightweight fill may be considered for the approach embankment widening. With the use of ultra-lightweight fill (having a bulk unit weight of about 11.5 kN/m^3) for the 6 m to 6.5 m high approach embankments, post-construction settlements of up to about 50 mm to 55 mm are predicted under the widened portions of both the north and south approaches. It is expected to take about one to two years for 90 per cent of the consolidation to occur under this widened embankment loading.

- **EPS fill for construction of approach embankment widening:** In order to further reduce the post-construction settlement under the widened approach embankments to about 25 mm, consideration could be given to the use of EPS fill, which has a bulk unit weight of about 1 kN/m^3. A thickness of 4 m of EPS (about 70 per cent of the embankment material) would be required to reduce the post-construction settlement to about 25 mm under the new portions of the north and south approaches. The EPS should be provided with a minimum of 1.2 m of conventional fill / pavement structure cover on the top of the embankment and side slopes, in order to reduce the chance of freezing/icing on the road surface. The EPS thickness should be tapered away from the abutments under the road, in order to minimize abrupt differential settlement.

It is understood that, based on the construction and staging schedule, a maximum period of about 2.5 months could be available for preloading of the widened approach embankment area prior to shifting traffic onto the newly widened embankments to allow for construction of the western portion of the new underpass. There is a possibility that the available period could be even shorter than this, which makes relying on surcharging during this limited period undesirable. However, there is flexibility in the contract with respect to when padding and final paving of Lake Street can take place; the final paving could occur up to about one year following the opening of the widened embankment to local traffic. The rate of settlement has been examined based on this timeline, and is summarized as follows:

- Where conventional earth or granular fill is used for the embankment widening, it is estimated that about 25 per cent of the consolidation settlement will occur after 2.5 months, and about 70 per cent of the consolidation settlement will be completed after an additional year. If final paving occurred at this point, post-paving settlements of more than 25 mm would remain. As noted previously, it is expected to take between two and three years for 90 per cent of the settlement to occur. Based on the predicted rate of settlement, then, the use of conventional earth or granular fill is not recommended for the widening of the Lake Street approach embankments.

- Where ultra-lightweight slag fill is adopted for the approach embankment widening, it is estimated that about 15 mm to 20 mm of settlement will occur after 2.5 months, and about 35 mm to 40 mm of settlement will be completed after one year. If final paving occurred at this point, the maximum post-paving settlement would be limited to about 15 mm.

Based on the construction schedule and the considerations outlined above, the use of ultra-lightweight slag fill has been selected for construction of the Lake Street approach embankment widenings. It is understood that MH will specify use of an asphalt curb and sidewalk instead of concrete for a short section north and south of the new structure; settlement monitoring will be carried out (as discussed further in Section 6.7); then prior to completion of the contract, sections of settled embankment will be padded and re-paved and the asphalt curbs and sidewalks will be removed and replaced with concrete curbs and sidewalks.

Based on consideration of the embankment geometry and settlement profile, the ultra-lightweight slag fill should be placed for a distance of 20 m behind the north and south abutments, over the full width of the approach embankment widening. The slag fill should then be transitioned at 5H:1V to conventional earth/granular fill. Slag fill is not required for the grade raise on top of the existing Lake Street approach embankments, where the predicted settlement using conventional earth/granular fill for the grade raise is on the order of 25 mm or less; this predicted settlement, together with the timing for placement of this fill, will be reasonably consistent with the magnitude and timing of settlement for the widened portion of the embankment.

6.7 Construction Considerations

6.7.1 Open-Cut Excavations

Excavations for the pile caps will extend through existing fill at the abutments (where it is assumed that the pile caps will be perched), and into the surficial silty sand to sand and silt deposit at the piers. Open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and the upper silty sand to sand and silt soils are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical.

6.7.2 Temporary Roadway Protection

Excavation support may be required at the site for temporary roadway protection (for example, to maintain traffic on Lake Street during construction, or where there is limited space along the QEW adjacent to the pier excavations). Where required, the temporary excavation support system should be designed and constructed in accordance with MTO’s Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 105S19.

6.7.3 Groundwater Control

It is noted that the surficial silty sand to sand and silt deposit may be water-bearing, particularly during wet periods of the year, with groundwater “perched” atop the underlying, less permeable clayey silt till deposit. It is anticipated that the groundwater within the pier pile cap excavations can be adequately controlled by pumping from properly filtered sumps.

6.7.4 Obstructions During Pile Driving

As discussed in Section 6.3, it is recommended that an NSSP be included in the Contract Documents to warn the contractor of the presence of boulders within the overburden soils, which are glacially derived, as such obstructions may affect the installation of steel H-piles. A draft NSSP is provided in Appendix A.

6.7.5 Settlement Monitoring for Approach Embankments

As discussed in Section 6.6.3, it is recommended that some settlement monitoring be carried out for the widened portion of the Lake Street embankments, prior to final paving and restoration of concrete curbs and sidewalks. A monitoring instrumentation drawing has been developed to show approximate locations for the proposed settlement monitoring plates, and this drawing is included in Appendix A together with an NSSP to address the supply, installation and monitoring of the settlement plates.

7.0 CLOSURE

This Foundation Design Report was prepared by Shannon Palmer, EIT, and reviewed by Lisa Coyne, P.Eng., an Associate and Senior Engineer of Golder, with technical input from Murty Devata, P.Eng., a Specialist Foundations Consultant to Golder. Fin Heffernan, P.Eng., a Designated MTO Contact for Golder, carried out an independent review of the report.

GOLDER ASSOCIATES LTD.

Shannon L. Palmer, EIT
Geotechnical Group

Lisa C. Coyne, P.Eng.
Associate

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

SLP/LCC/MSD/FJH/lcc
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TABLE 1
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
LAKE STREET UNDERPASS, G.W.P. 607-00-00

Foundation Option	***Feasibility***	***Advantages***	***Disadvantages***	***Relative Costs***
Spread footings	• Not feasible for support of abutments or piers	• Not applicable	• Low geotechnical resistance • Differential settlement across abutments and between foundation elements	• Not applicable
Steel H-piles driven to found within 100-blow lower till or bedrock	• Feasible for support of abutments and piers	• Minimize differential settlement across abutments and between foundation elements • Readily installed	• Piles may "hang up" on boulders within lower till deposit.	• Less expensive than caissons
Caissons bored to found within 100-blow lower till or bedrock	• Feasible for support of abutments and pier	• Minimize differential settlement across abutments and between foundation elements • Higher bearing resistances than for steel H-piles, though partially offset by higher downdrag loads than for steel H-piles.	• Possibility of basal heave. Liner required due to soil conditions. Permanent liner recommended over temporary liner, to avoid difficulties with withdrawal of temporary liner due to length of caissons and presence of hard/very dense soils near caisson base, and to avoid "necking" of the caissons. • Potential difficulty with boulders within lower till deposit.	• More expensive than steel H-piles, plus cost of permanent liner if adopted as recommended

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
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
l_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 = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 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
* density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

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	

Golder Associates

RECORD OF BOREHOLE No 301

 METRIC

PROJECT 04-1111-002 | LOCATION N 4782009.1 ;E 325032.6 | ORIGINATED BY PKS

W.P. 607-00-00

DIST Central HWY QEW | BOREHOLE TYPE 108 mm Solid Stem Augers | COMPILED BY SLP

DATUM Geodetic | DATE October 13, 2004 | CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
98.4	GROUND SURFACE						
0.0	TOPSOIL						
0.2	Silty sand (FILL) Loose Brown Moist	1	SS	7	98		
97.7 / 0.8	Silty SAND to SAND and SILT Compact Brown Moist to wet	2	SS	12			
96.9 / 1.5	CLAYEY SILT, trace to some sand, trace gravel and shale fragments (TILL) Stiff to very stiff Grey Moist to wet	3	SS	24	97		
		4	SS	20	96		2 14 54 30
		5	SS	16	95		
		6	SS	13			
		7	SS	9	94		
					93		
		8	SS	12	92		
91.3 / 7.2	END OF BOREHOLE						

Note:

1. Borehole dry on completion of drilling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3

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

Foundation Design

RECORD OF BOREHOLE No 302

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4781993.2 ;E 355020.0
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE November 30, 2004
CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
101.3	GROUND SURFACE						
0.0	Asphalt						
0.2	Sand and gravel (FILL) Compact to very dense Brown Moist	1	SS	29			
100.2 / 1.1	Sand (FILL) Dense to very dense Black Moist	2	SS	88		○	
		3	SS	47			
		4	SS	47		○	
98.2 / 3.1	Silty sand, containing rootlets, organics and clay pockets (FILL) Compact Brown Moist	5	SS	15			
		6	SS	13		○	
		7	SS	23			
96.1 / 5.2	Silty SAND to SAND and SILT, trace organics Loose Brown Wet	8	SS	6		○	
93.7 / 7.6	CLAYEY SILT, some sand, trace gravel and shale fragments (TILL) Stiff to very stiff Grey to red Wet	9	SS	25			
		10	SS	17	∇	w_P – w – w_L	
		11	SS	14			
		12	SS	12			
		13	SS	17		w_P – w – w_L	1 15 56 28

Elevation scale: 101, 100, 99, 98, 97, 96, 95, 94, 93, 92, 91, 90, 89, 88, 87

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

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

MIS-MTO 001 04111002AAMTO.GPJ GAL-MISS.GDT 16/10/06

RECORD OF BOREHOLE No 302

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4781993.2 ;E 355020.0
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE November 30, 2004
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE)	WATER CONTENT (%)
	CLAYEY SILT, some sand, trace gravel and shale fragments (TILL) Stiff to very stiff Grey to red Wet	14	SS	14	86		w ≈ 17
		15	SS	12	85–84		
83.5 / 17.8	CLAYEY SILT to SILTY CLAY, trace to some sand, trace shale fragments Firm to stiff Grey/red Wet	16	SS	9	83		wP ≈ 20, w ≈ 31, wL ≈ 37
					82	+ ≈ 66 (2.3)	
		17	SS	8	81		
		18	SS	8	80		wP ≈ 19, w ≈ 31, wL ≈ 31
					79	+ ≈ 43 (2.3)	
78.1 / 23.2	CLAYEY SILT, some sand, trace gravel and shale fragments, containing silty sand seams (TILL) Very stiff to hard Grey/red Wet	19	SS	28	78–76		
		20	SS	100/.25	74		w ≈ 8
73.3 / 28.0	SAND and SILT, trace gravel and shale pieces, trace clay (TILL) Very dense Red Wet				73–72		

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 302

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00 LOCATION N 4781993.2 ;E 355020.0 ORIGINATED BY PKS
DIST Central HWY QEW BOREHOLE TYPE 108 mm Solid Stem Augers COMPILED BY SLP
DATUM Geodetic DATE November 30, 2004 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND and SILT, trace gravel and shale pieces, trace clay (TILL) Very dense Red Wet						71				
70.4			21	SS	100/.15						
30.9	END OF BOREHOLE Note: 1. Water level in open borehole at 9.1 m depth upon completion of drilling operations										

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 303

 METRIC

PROJECT 04-1111-002			
W.P. 607-00-00	LOCATION N 4781989.3 ;E 325035.3		ORIGINATED BY PKS
DIST Central	HWY QEW	BOREHOLE TYPE 108 mm Solid Stem Augers	COMPILED BY SLP
DATUM Geodetic	DATE October 14, 2004		CHECKED BY LCC

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10, 20, 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
97.2	GROUND SURFACE							
0.0	TOPSOIL							
0.2	Silty sand, trace gravel and wood fragments (FILL) Compact Brown Moist	1	SS	12				
96.5 / 0.8	Silty SAND to SAND and SILT Compact Brown Wet	2	SS	13				
95.7 / 1.5	CLAYEY SILT to SILTY CLAY, trace sand Very stiff Grey Wet	3	SS	21				
		4	SS	16	0	1	79	20
94.2 / 3.1	CLAYEY SILT, trace to some sand, trace gravel (TILL) Stiff to very stiff Grey/red Wet	5	SS	12				
		6	SS	13				
		7	SS	13				
		8	SS	8	1	12	55	32
		9	SS	12				
		10	SS	12				
	Shale fragments below 10.7 m depth	11	SS	28				
		12	SS	22				
84.0 / 13.2	CLAYEY SILT to SILTY CLAY containing fine sand seams Firm to stiff Grey Wet	13	SS	7				

Elevation scale: 97, 96, 95, 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83

Field vane at approx. elev. 90.1: >100

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 303

 METRIC

PROJECT 04-1111-002			
W.P. 607-00-00	LOCATION N 4781989.3 ;E 325035.3	ORIGINATED BY PKS	
DIST Central HWY QEW	BOREHOLE TYPE 108 mm Solid Stem Augers	COMPILED BY SLP	
DATUM Geodetic	DATE October 14, 2004	CHECKED BY LCC	

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%) / SHEAR STRENGTH	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT to SILTY CLAY containing fine sand seams Firm to stiff Grey Wet	14	SS	9		
		15	SS	3		0 8 61 31
79.0 18.3	CLAYEY SILT, trace to some sand, trace gravel (TILL) Hard Grey Wet	16	SS	17		
		17	SS	24		
77.0 20.3	SAND and SILT, trace to some gravel (TILL) Dense to very dense Red Wet	18	SS	100/.23		
74.4 22.9	CLAYEY SILT, some sand, trace gravel, silty sand layers (TILL) Hard Red Wet	19	SS	111		
73.8 23.5	CLAYEY SILT, some sand, trace gravel, shale/limestone fragments (RESIDUAL SOIL) Hard Red Wet	20	SS	60/.07		
72.6 24.6	END OF BOREHOLE AUGER REFUSAL SPOON REFUSAL					

Elevation scale: 82, 81, 80, 79, 78, 77, 76, 75, 74, 73

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Note:

1. Water level in open borehole at 18.3 m depth upon completion of drilling operations.

2. Water level in piezometer at 20.6 m depth (Elevation 76.6 m) on October 18, 2004.

3. Water level in piezometer at 1.85 m depth (Elevation 95.4 m) on November 26, 2004.

4. Water level in piezometer at 1.6 m depth (Elevation 95.6 m) on May 13, 2005.

5. Water level in piezometer at 1.6 m depth (Elevation 95.6 m) on December 6, 2005.

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

Foundation Design

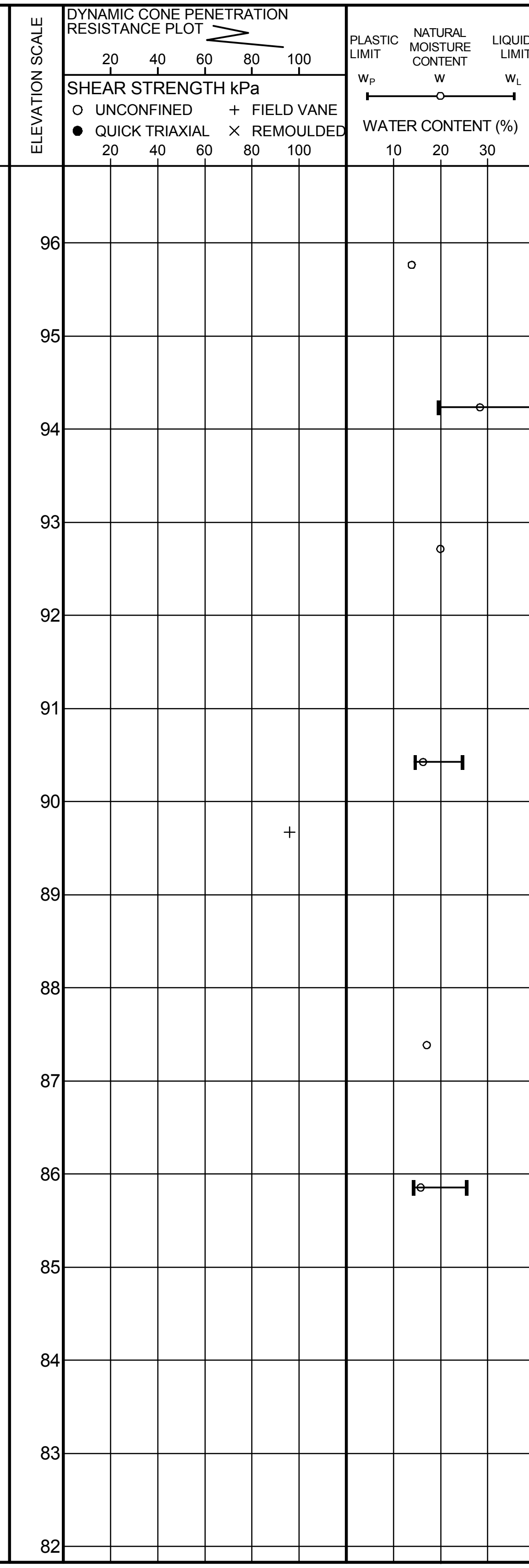
Golder Associates

RECORD OF BOREHOLE No 304

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4781963.0 ;E 325029.0
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE October 18, 2004
CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
96.8	GROUND SURFACE			
0.0	Silty sand, trace to some gravel, trace organics (FILL) Compact Brown/black Moist	1	SS	14
		2	SS	22
95.3 / 1.5	Silty SAND to SAND and SILT, trace gravel, trace organics Loose Brown Moist	3	SS	2
94.5 / 2.3	CLAYEY SILT to SILTY CLAY, trace sand Soft to very stiff Grey Moist to wet	4	SS	2
		5	SS	18
		6	SS	13
92.3 / 4.6	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Grey to reddish brown Moist to wet	7	SS	12
		8	SS	8
		9	SS	14
		10	SS	16
		11	SS	16
		12	SS	18
		13	SS	13



GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

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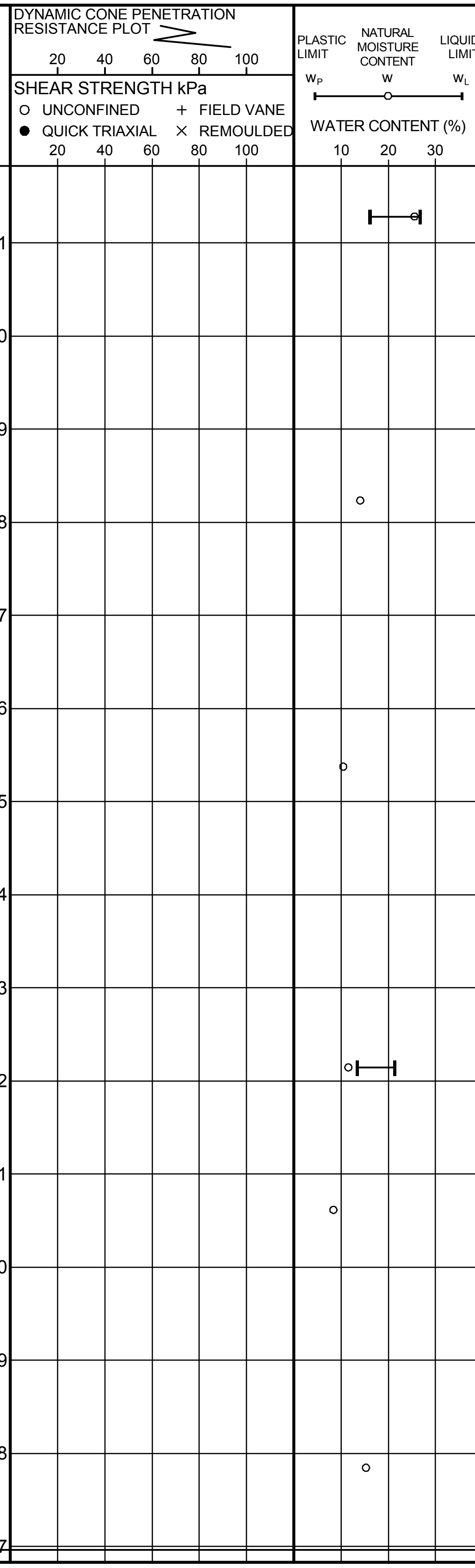
RECORD OF BOREHOLE No 304

 METRIC

PROJECT 04-1111-002			
W.P. 607-00-00	LOCATION N 4781963.0 ;E 325029.0	ORIGINATED BY PKS	
DIST Central	HWY QEW	BOREHOLE TYPE 108 mm Solid Stem Augers	COMPILED BY SLP
DATUM Geodetic	DATE October 18, 2004	CHECKED BY LCC	

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		14	SS	7		3 11 59 27
81.0 / 15.9	CLAYEY SILT to SILTY CLAY, some sand, trace gravel Firm to stiff Grey to reddish brown Moist to wet	15	SS	8		
79.0 / 17.8	CLAYEY SILT,some sand, trace gravel (TILL) Stiff to very stiff Grey to reddish brown Moist to wet	16	SS	11		
		17	SS	26		
76.1 / 20.7	SAND and SILT, trace to some gravel, trace clay (TILL) Very dense Reddish grey to reddish brown Wet	18	SS	100/.23		13 37 44 6
74.0 / 22.8	CLAYEY SILT, trace to some sand, trace gravel (TILL) Hard Reddish grey to grey Moist to wet	19	SS	100/.30		
		20	SS	59		
70.6	Silty sand seam at 26.2 m depth					
26.2	CLAYEY SILT, trace sand, trace gravel, shale and limestone fragments (RESIDUAL SOIL) Hard Red Moist/wet	21	SS	69		
69.4 / 27.4	SHALE (BEDROCK) Red to grey	22	SS	100/.10		
		23	SS	100/.05	∇	
67.0						



UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 304

 METRIC

PROJECT 04-1111-002

W.P. 607-00-00 LOCATION N 4781963.0 ;E 325029.0 ORIGINATED BY PKS

DIST Central HWY QEW BOREHOLE TYPE 108 mm Solid Stem Augers COMPILED BY SLP

DATUM Geodetic DATE October 18, 2004 CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
29.9	END OF BOREHOLE AUGER REFUSAL SPOON REFUSAL Note: 1. Water level in open borehole at 29.3 m depth upon completion of drilling operations										

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 305

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00 LOCATION N 4781963.0 ;E 325039.3 ORIGINATED BY PKS
DIST Central HWY QEW BOREHOLE TYPE 108 mm Solid Stem Augers COMPILED BY SLP
DATUM Geodetic DATE October 18, 2004 CHECKED BY LCC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa
96.2	GROUND SURFACE						
0.0	Silty SAND to SAND and SILT, trace gravel Loose to compact Brown Moist to wet	1	SS	9		96	
		2	SS	12		95	
94.4 1.8	CLAYEY SILT, trace to some sand, trace gravel (TILL) Stiff to very stiff Grey to reddish grey Moist to wet	3	SS	24		94	
		4	SS	17		93	
		5	SS	13			
		6	SS	10	∇	92	
		7	TO	PH		91	
							>100 +
		8	SS	8		90	
		9	SS	8		89, 88	
							× >100 +
		10	SS	10		87, 86	
		11	SS	12		85, 84	
		12	SS	22		83	
82.5 13.7	CLAYEY SILT to SILTY CLAY, containing sand seams Firm to soft Grey Wet	13	SS	8		82	

Column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No 305

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4781963.0 ;E 325039.3
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE October 18, 2004
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR SA SI CL
	CLAYEY SILT to SILTY CLAY, containing sand seams Firm to soft Grey Wet	14	SS	6	81			
					80			
		15	SS	2	79		wP–w	0 2 67 31
		16	TO	PH				
78.0 18.2	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Grey Wet				78	× + ; × >100 +		
					77			
76.1 20.1	SAND and SILT, trace gravel, trace clay (TILL) Very dense Red Wet	17	SS	53	76		o	
75.1 21.1	CLAYEY SILT, some sand, trace gravel and shale fragments (TILL) Hard Red Wet	18	SS	100/.23	75			
					74			
		19	SS	108	73		o	
					72			
71.8 24.4	Silty SAND, trace gravel Very dense Red Wet	20	SS	79			o	
71.4 24.8	CLAYEY SILT, trace gravel and shale fragments, containing sand lenses (RESIDUAL SOIL) Hard Grey Wet				71			
		21	SS	48	70			3 13 70 15
					69			
68.8 27.4	SHALE (BEDROCK) Bedrock cored from 27.4 m to 30.5 m depth For bedrock coring details refer to Drillhole Record No. 305				68			
					67			

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 305

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4781963.0 ;E 325039.3
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE October 18, 2004
CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
65.7							66				
30.5	END OF BOREHOLE Note: 1. Water level in open borehole at 4.6 m depth upon completion of drilling operation										

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 306

METRIC

PROJECT 04-1111-002			
W.P. 607-00-00	LOCATION N 4781946.9 ;E 325034.1		ORIGINATED BY PKS
DIST Central	HWY QEW	BOREHOLE TYPE 108 mm Solid Stem Augers	COMPILED BY SLP
DATUM Geodetic	DATE December 5, 2004		CHECKED BY LCC

SOIL PROFILE / SAMPLES

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.8	GROUND SURFACE					
0.0	Sand and gravel (FILL) Compact Brown Moist	1	SS	20		
96.0 / 0.8	Silty SAND to SAND and SILT, trace gravel Loose to compact Brown Wet	2	SS	9		
		3	SS	22		
94.5 / 2.3	CLAYEY SILT, some sand, trace gravel and shale fragments(TILL) Stiff to very stiff Grey Wet	4	SS	17	∇	
		5	SS	15		
		6	SS	12		
		7	SS	8		
		8	SS	14		
		9	SS	14		0 14 55 31
		10	SS	16		
		11	SS	10		
84.6 / 12.2	CLAYEY SILT to SILTY CLAY, trace sand, trace gravel and shale fragments Firm Grey Wet	12	SS	6		
		13	SS	5		

ELEVATION SCALE: 96, 95, 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82

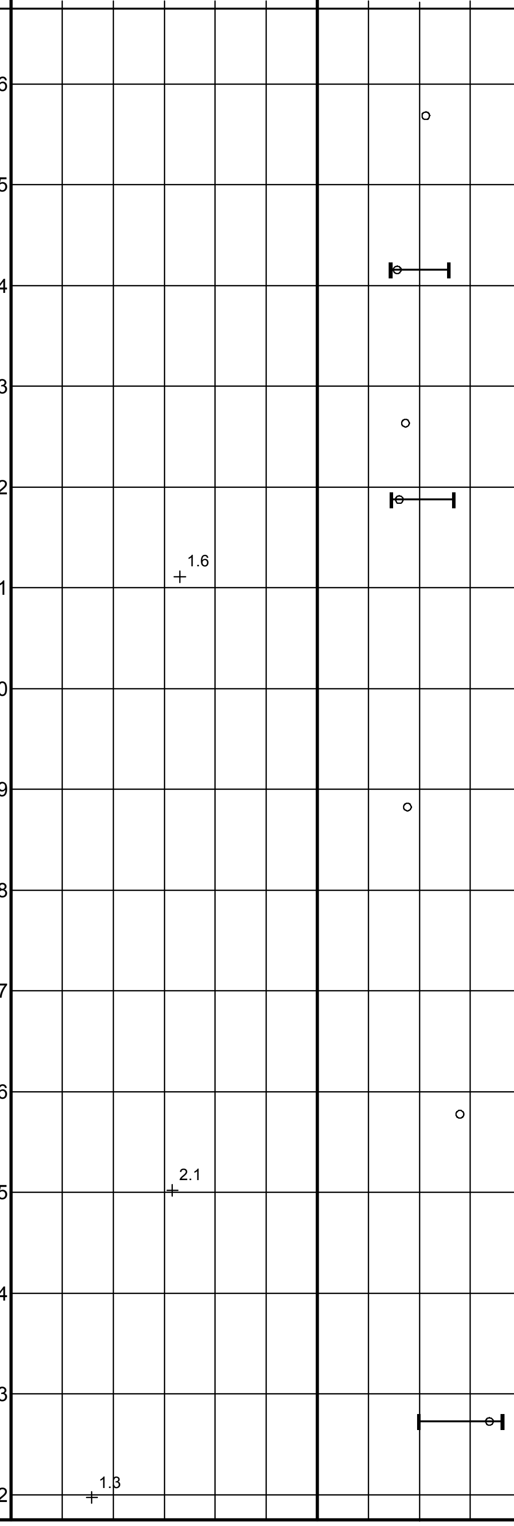
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³



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

MIS-MTO 001 04111002AAMTO.GPJ GAL-MISS.GDT 16/10/06

Golder Associates

RECORD OF BOREHOLE No 306

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4781946.9 ;E 325034.1
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE December 5, 2004
CHECKED BY LCC

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
81.5 15.3	CLAYEY SILT, some sand, trace gravel, shale fragments (TILL) Stiff Grey Wet	14	SS	11					
80.5 16.3	SAND and SILT, some gravel, trace to some clay (TILL) Compact Grey Wet	15	SS	14		11	44	35	10
79.0 17.8	CLAYEY SILT, trace sand and gravel, containing silty sand interlayers (TILL) Very stiff to hard Red Wet	16	SS	16	~15				
		17	SS	17					
		18	SS	100/.20	~9				
		19	SS	100/.07					
72.3 24.5	END OF BOREHOLE Note: 1. Water level in open borehole at 3.1 m depth upon completion of drilling operations								

Elevation scale: 81, 80, 79, 78, 77, 76, 75, 74, 73

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

Golder Associates

RECORD OF BOREHOLE No 307

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4789924.6 ;E 325033.3
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE October 26, 2004
CHECKED BY LCC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
97.2	GROUND SURFACE				
0.0	Silty SAND, trace clay, trace gravel (FILL) Compact Brown Moist	1	SS	14	
96.5 / 0.8	SAND and SILT, trace gravel, trace clay Compact to loose Moist to wet	2	SS	16	
		3	SS	7	2 47 47 4
		4	SS	22	
94.5 / 2.7	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Grey Wet	5	SS	18	
		6	SS	9	
		7	SS	12	
		8	SS	7	
		9	SS	9	
		10	SS	14	
87.0 / 10.2	CLAYEY SILT to SILTY CLAY, trace sand Soft to stiff Grey Wet	11	SS	6	
		12	SS	5	0 1 37 62
		13	SS	6	

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (97 to 83); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

47

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 307

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4789924.6 ;E 325033.3
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE October 26, 2004
CHECKED BY LCC

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20 40 60 80 100) | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	CLAYEY SILT to SILTY CLAY, trace sand Soft to stiff Grey Wet	14	SS	4
		15	SS	8
79.4 17.8	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Grey Wet	16	SS	18
77.1 20.1	SAND and SILT, trace gravel, trace clay (TILL) Very dense Red Wet	17	SS	54
		18	SS	100/.23
74.4 22.9	CLAYEY SILT, some sand, trace gravel and shale fragments (TILL) Hard Red Wet	19	SS	100/.10
72.8	Sandy SILT	20	SS	100/.2
24.6	END OF BOREHOLE Note: 1. Water level in borehole upon completion at 4.6 m depth			

Elevation scale: 82, 81, 80, 79, 78, 77, 76, 75, 74, 73

41

$+^3, \times^3$: Numbers refer to Sensitivity
$O^{3\%}$ STRAIN AT FAILURE

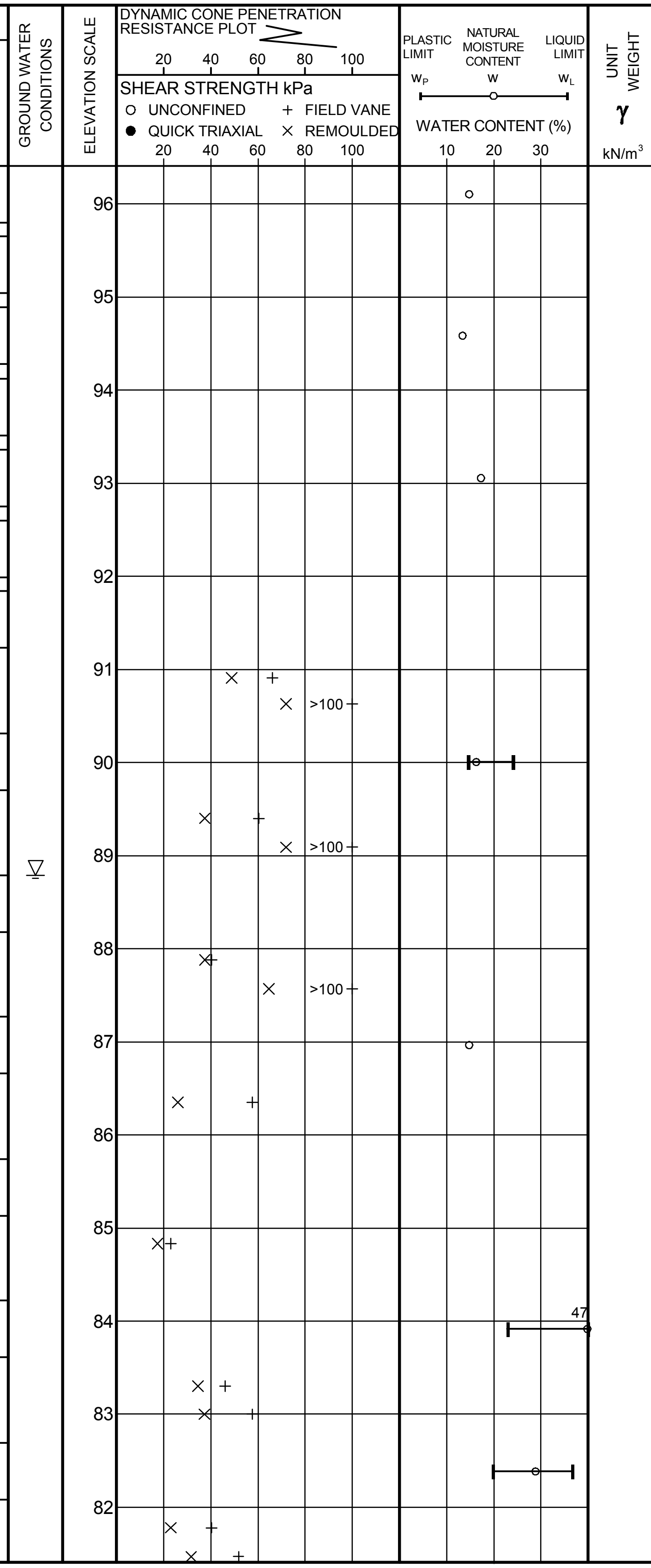
Golder Associates

RECORD OF BOREHOLE No 308

 METRIC

PROJECT 04-1111-002			
W.P. 607-00-00	LOCATION N 4781919.0 ;E 325040.5		ORIGINATED BY PKS
DIST Central HWY QEW	BOREHOLE TYPE 108 mm Solid Stem Augers		COMPILED BY SLP
DATUM Geodetic	DATE October 27, 28, 2004		CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.4	GROUND SURFACE				
0.0	Silty sand, trace gravel (FILL) Loose Brown Moist to wet	1	SS	9	
95.7 / 0.8	Silty SAND to SAND and SILT Compact Brown Moist to wet	2	SS	11	
		3	SS	25	
94.4 / 2.0	CLAYEY SILT, some sand, trace gravel (TILL) Firm to very stiff Grey Wet	4	SS	14	
		5	SS	11	
		6	SS	7	
		7	TO	PH	
		8	SS	8	
		9	SS	7	
		10	SS	9	
86.0 / 10.4	CLAYEY SILT to SILTY CLAY, trace sand Firm to stiff Grey Wet	11	SS	6	0 1 35 64
		12	SS	2	
		13	TO	PH	



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$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

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RECORD OF BOREHOLE No 308

 METRIC

PROJECT	04-1111-002		
W.P.	607-00-00	LOCATION	N 4781919.0 ;E 325040.5
DIST	Central HWY QEW	BOREHOLE TYPE	108 mm Solid Stem Augers
DATUM	Geodetic	DATE	October 27, 28, 2004

ORIGINATED BY PKS
COMPILED BY SLP
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
	CLAYEY SILT to SILTY CLAY, trace sand Firm to stiff Grey Wet	14	SS	4
79.5 16.9	CLAYEY SILT, some sand, trace gravel (TILL) Firm to very stiff Grey Wet	15	SS	4
		16	SS	18
76.6 19.8	SAND and SILT, trace gravel, trace clay (TILL) Very dense Red Wet	17	SS	100/.23
75.1 21.3	CLAYEY SILT, some sand, trace gravel and limestone fragments (TILL) Hard Red Wet	18	SS	100/.25
		19	SS	100/.25
73.3 23.1	END OF BOREHOLE Note: 1. Water level in open borehole at 7.6 m depth upon completion of drilling operations			

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (81, 80, 79, 78, 77, 76, 75, 74); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED — 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+³, ×³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 309

 METRIC

PROJECT	04-1111-002		
W.P.	607-00-00	LOCATION	N 4781899.9 ;E 325024.3
DIST	Central	HWY	QEW
BOREHOLE TYPE	108 mm Solid Stem Augers		
DATUM	Geodetic	DATE	November 29, 2004
ORIGINATED BY	PKS	COMPILED BY	SLP
CHECKED BY	LCC		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10, 20, 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)
101.3	GROUND SURFACE				
0.0	Asphalt				
0.2	Sand and gravel (FILL) Compact to dense Brown Moist	1	SS	40	
100.2 / 1.1	Sand (FILL) Dense to very dense Black Moist	2	SS	36	
		3	SS	43	
		4	SS	56	
98.2 / 3.1	Sand, some silt (FILL) Compact to dense Brown Moist to wet	5	SS	16	
		6	SS	31	
		7	SS	32	
96.1 / 5.2	SAND and SILT, trace clay, trace gravel Compact Brown Wet	8	SS	25	2 45 44 4
93.7 / 7.6	CLAYEY SILT, some sand, trace gravel and limestone fragments (TILL) Stiff to very stiff Grey/red Wet	9	SS	26	
		10	SS	13	
		11	SS	14	
		12	SS	13	
88.0 / 13.3	CLAYEY SILT to SILTY CLAY, trace sand, trace gravel and shale fragments Firm to stiff Grey/red Wet	13	SS	7	

ELEVATION SCALE: 101, 100, 99, 98, 97, 96, 95, 94, 93, 92, 91, 90, 89, 88, 87

Continued Next Page

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

RECORD OF BOREHOLE No 309

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4781899.9 ;E 325024.3
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE November 29, 2004
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT to SILTY CLAY, trace sand, trace gravel and shale fragments Firm to stiff Grey/red Wet	14	SS	5	
		15	SS	7	
		16	SS	5	
		17	SS	8	
		18	SS	3	0 4 54 42
78.1 23.2	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Grey Wet	19	SS	18	
		20	SS	100/.23	
72.5 28.8	SAND and SILT, trace gravel and shale pieces (TILL) Very dense Red Wet				

Elevation scale: 86, 85, 84, 83, 82, 81, 80, 79, 78, 77, 76, 75, 74, 73, 72

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 309

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4781899.9 ;E 325024.3
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE November 29, 2004
CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
70.7			21	SS	100/ 10		71				
30.6	END OF BOREHOLE Note: 1. Water level in open borehole at 12.2 m depth upon completion of drilling operations										

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 16/10/06

Foundation Design

Golder Associates

RECORD OF BOREHOLE No 310

 METRIC

PROJECT 04-1111-002

W.P. 607-00-00 LOCATION N 4781895.4 ;E 325041.3 ORIGINATED BY PKS

DIST Central HWY QEW BOREHOLE TYPE 108 mm Solid Stem Augers COMPILED BY SLP

DATUM Geodetic DATE November 1, 2004 CHECKED BY LCC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
96.4	GROUND SURFACE			
0.0	TOPSOIL			
0.1	Silty sand, trace gravel (FILL) Compact Brown Moist to wet	1	SS	9
95.7 / 0.8	Silty SAND to SAND and SILT, trace gravel Compact Brown Moist to wet	2	SS	13
94.8 / 1.6	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Brown to grey Moist to wet	3	SS	22
		4	SS	17
		5	SS	12
		6	SS	11
		7	SS	10
		8	SS	13
88.4 / 8.0	CLAYEY SILT to SILTY CLAY, trace sand Firm to stiff Grey/red Wet	9	SS	8
		10	SS	5
		11	SS	6
		12	SS	5
		13	SS	10

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (96 to 82); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Field vane sensitivity values plotted: 1.5, 1.7, 1.5. Liquid limit value plotted: 44.

Continued Next Page

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

RECORD OF BOREHOLE No 310

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00
LOCATION N 4781895.4 ;E 325041.3
ORIGINATED BY PKS
DIST Central HWY QEW
BOREHOLE TYPE 108 mm Solid Stem Augers
COMPILED BY SLP
DATUM Geodetic
DATE November 1, 2004
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	CLAYEY SILT to SILTY CLAY, trace sand Firm to stiff Grey/red Wet	14	SS	6	81
					80
		15	SS	6	79
78.5 17.9	CLAYEY SILT, trace to some sand, trace gravel (TILL) Very stiff Grey/red Wet	16	SS	15	78
					77
76.6 19.8	SAND and SILT, trace clay, trace gravel, shale and limestone fragments (TILL) Very dense Red Wet	17	SS	132	76
75.1 21.3	CLAYEY SILT, some sand, trace gravel, shale and limestone fragments (TILL) Hard Red Wet	18	SS	100/.15	75
					74
73.4 23.0	END OF BOREHOLE	19	SS	100/.18	

Shear strength: field vane 1.6 (sensitivity) at approx. elev. 80.1.

Note:

1. Water level in piezometer at 2.1 m depth (Elevation 94.3 m) on November 26, 2004.

2. Water level in piezometer at 1.7 m depth (Elevation 94.7 m) on May 13, 2005, and at 1.9 m depth (Elevation 94.5 m) on Dec. 6, 2005.

4. Water level in piezometer at 1.9 m depth (Elevation 94.5 m) on December 6, 2005.

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE No 311

 METRIC

PROJECT 04-1111-002
W.P. 607-00-00 LOCATION N 4781875.5 ;E 325039.1 ORIGINATED BY PKS
DIST Central HWY QEW BOREHOLE TYPE 108 mm Solid Stem Augers COMPILED BY SLP
DATUM Geodetic DATE October 29, 2004 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
99.8	GROUND SURFACE					
0.0 / 0.1	TOPSOIL Silty sand (FILL)	1	SS	11		
99.0 / 0.8	Silty SAND, trace gravel Compact Brown Moist to wet	2	SS	12		
97.9 / 1.8	CLAYEY SILT, some sand, trace gravel and shale fragments (TILL) Stiff to very stiff Brown to grey Wet	3	SS	20		
		4	SS	13		
		5	SS	14		1 11 55 33
		6	SS	13		
		7	SS	10		
					∇	
		8	SS	10		
93.1 / 6.7	END OF BOREHOLE Note: 1. Water level in open borehole at 6.1 m depth upon completion of drilling operations					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (99, 98, 97, 96, 95, 94); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

MINISTRY OF TRANSPORTATION, ONTARIO

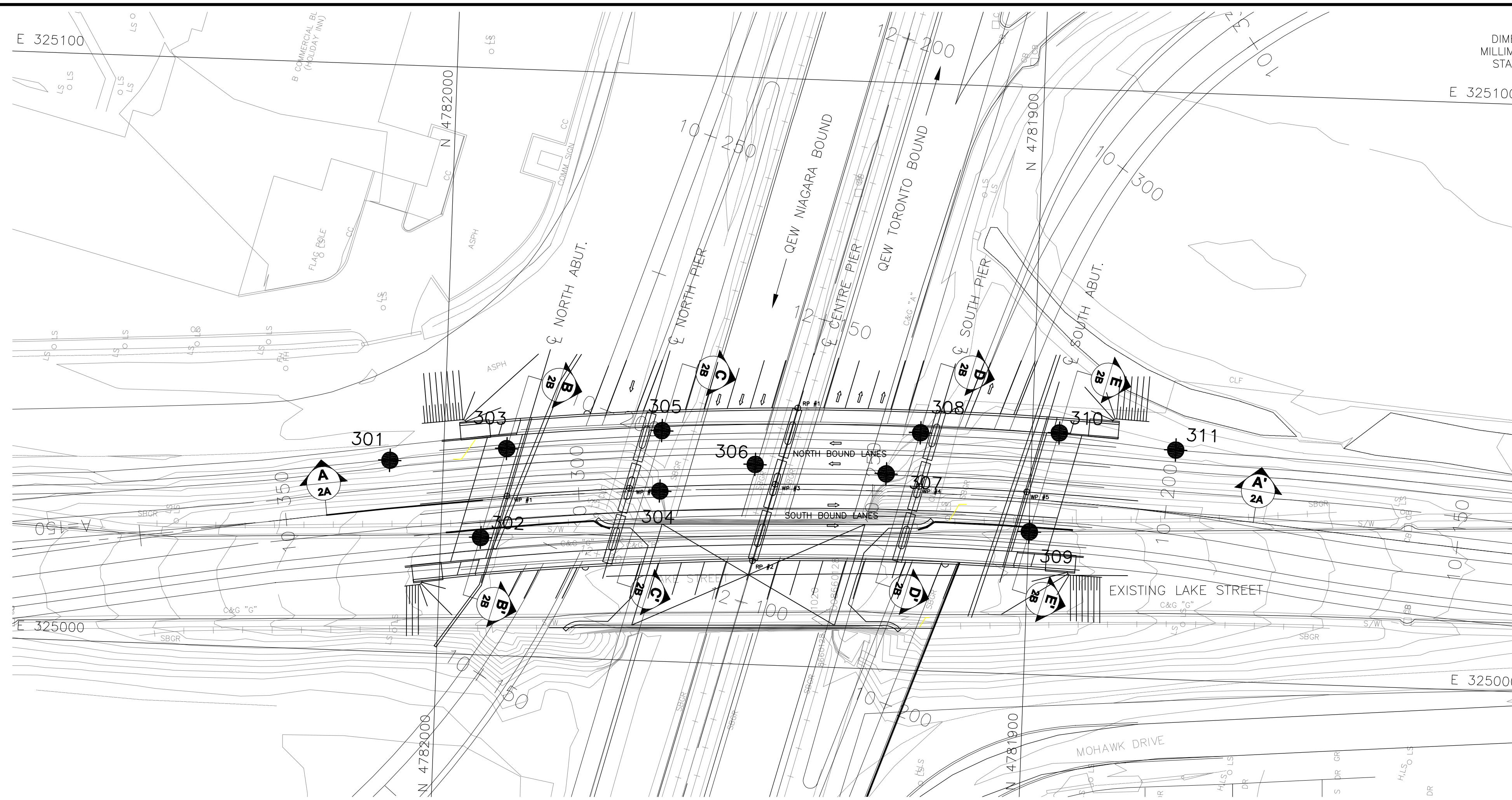
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 607-00-00

SHEET

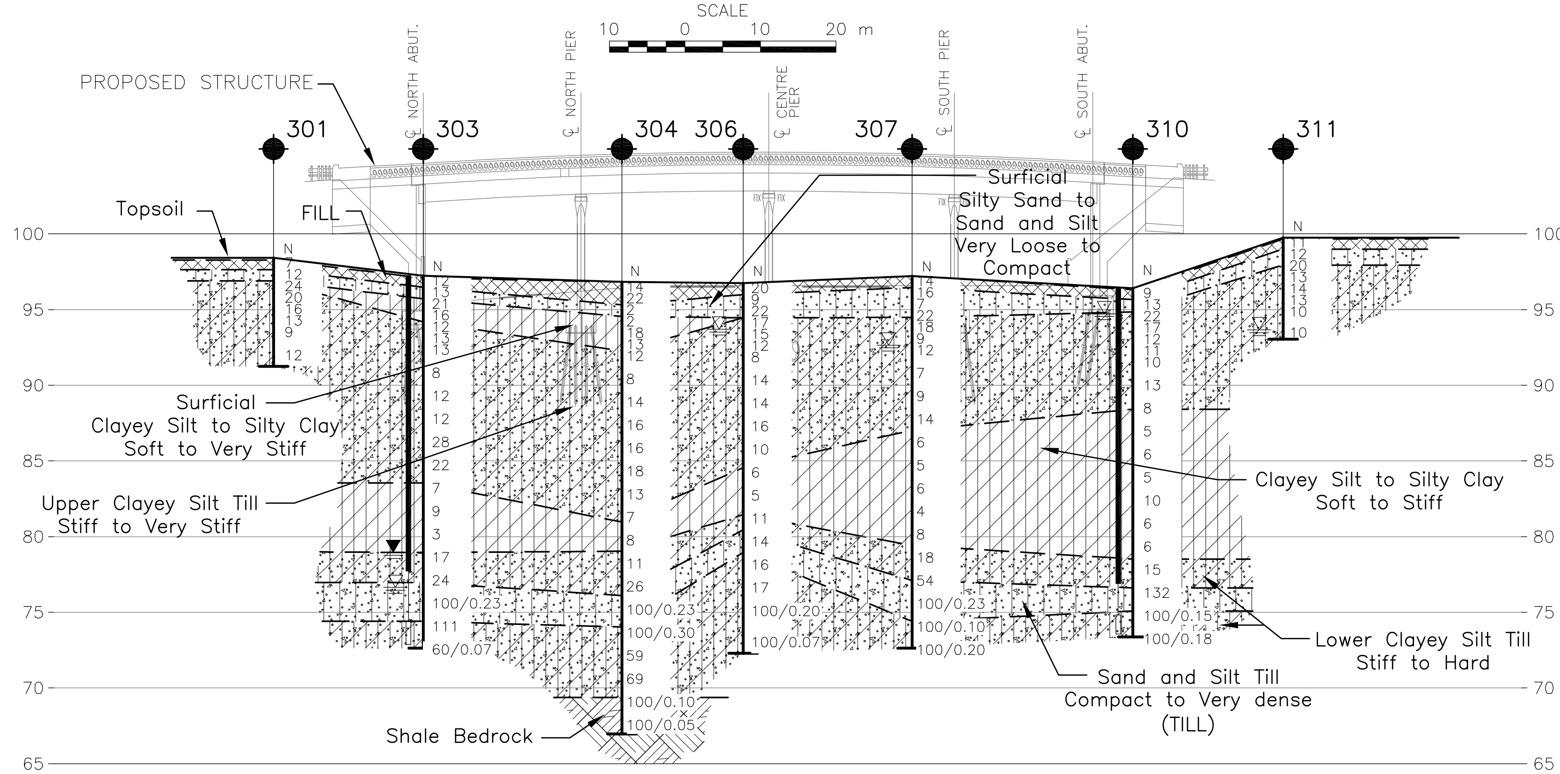
LAKE STREET UNDERPASS BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

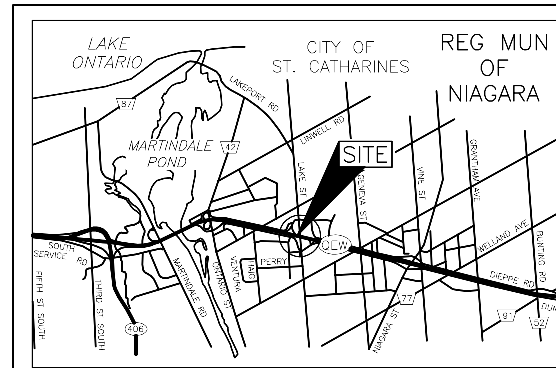
SCALE 10 0 10 20 m



A-A' ℄ PROFILE ALONG LAKE STREET

HORIZ. SCALE 10 0 10 20 m

VERT. SCALE 5 0 5 10 m



KEY PLAN

SCALE 1 0 1 km

LEGEND

- Borehole – Current 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 in piezometer, measured on November 1 and October 14, 2004
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
301	96.4	4782009.1	325032.6
302	101.3	4781993.2	325020.0
303	97.2	4781989.3	325035.3
304	96.8	4781963.0	325029.0
305	96.2	4781963.0	325039.3
306	96.8	4781946.9	325034.1
307	97.2	4781924.6	325033.3
308	96.4	4781919.0	325040.5
309	101.3	4781899.9	325024.3
310	96.4	4781895.4	325041.3
311	99.8	4781875.5	325039.1

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by Morrison Hershfield Limited, drawing file nos. 18105-01.dwg received Oct 16, 2006, qew final.dwg, m402.dwg and hm_elec_design.dwg, received June 1, 2003.

REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No.

HWY. QEW		PROJECT NO. 04-1111-002	DIST.
SUBM'D. SLP	CHKD. LCC	DATE: OCT 2006	SITE: 18-105
DRAWN: MSM	CHKD. SLP	APPD. LCC	DWG. 1

PLOT DATE: October 16, 2006
FILENAME: F:\Projects\2004\04-1111-002 (MH, StCatherines)\...\04111002BD001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

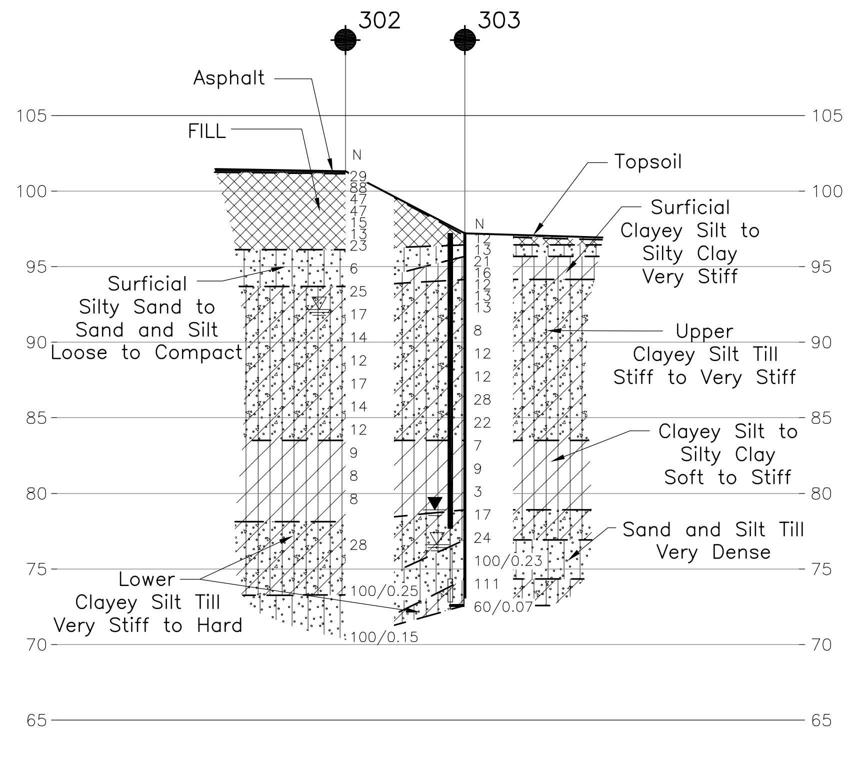
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 607–00–00

LAKE STREET UNDERPASS
SOIL STRATA

SHEET

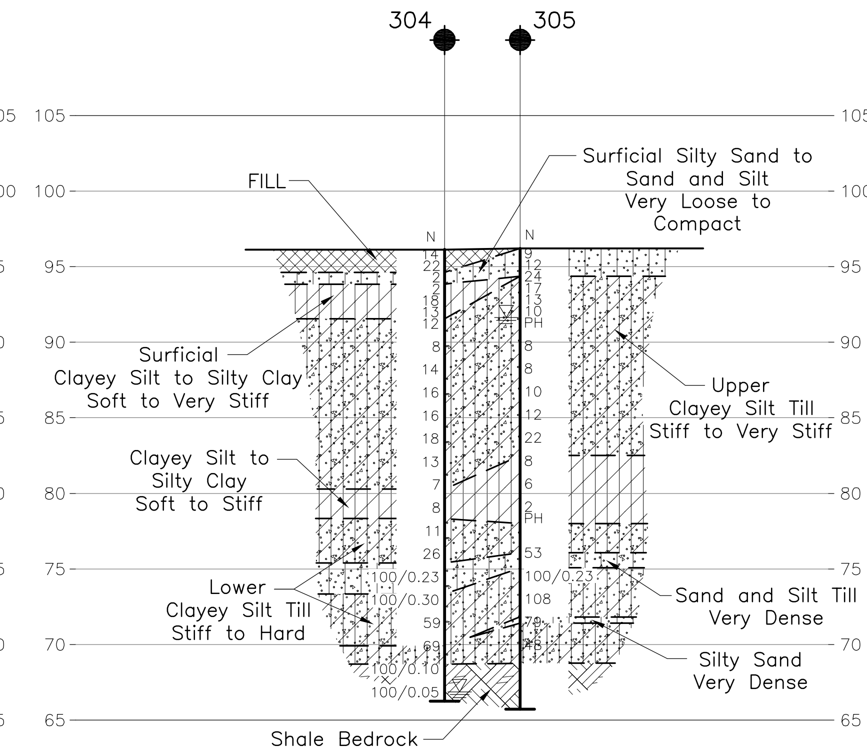
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



B-B' CROSS SECTION NORTH ABUTMENT

HORIZ. SCALE 10 0 10 20 m

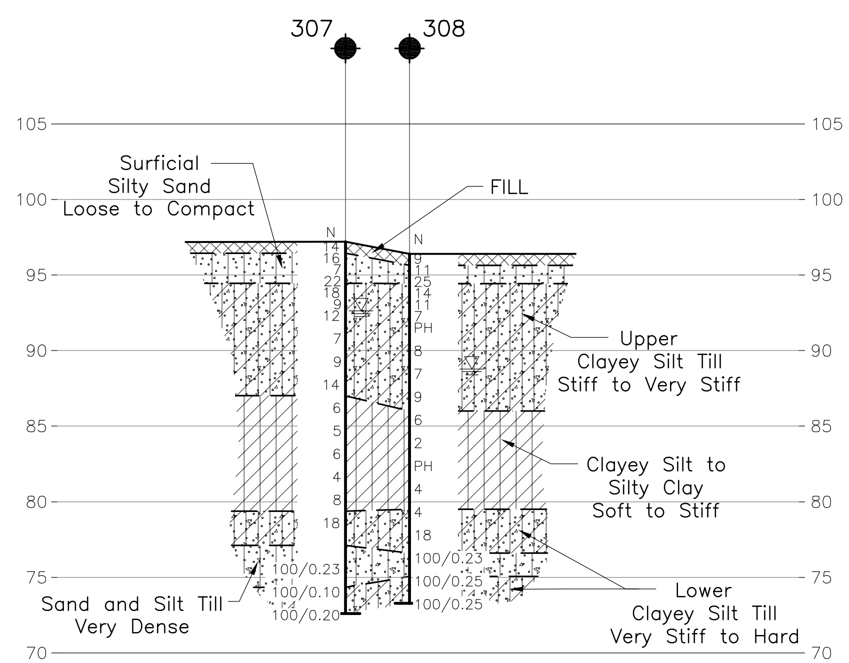
VERT. SCALE 5 0 5 10 m



C-C' CROSS SECTION NORTH PIER

HORIZ. SCALE 10 0 10 20 m

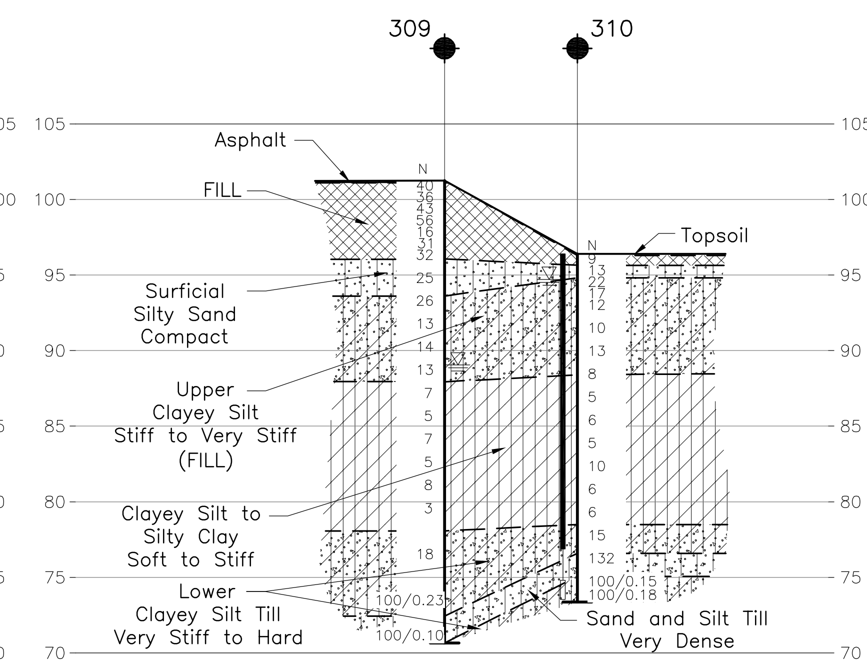
VERT. SCALE 5 0 5 10 m



B-B' CROSS SECTION SOUTH PIER

HORIZ. SCALE 10 0 10 20 m

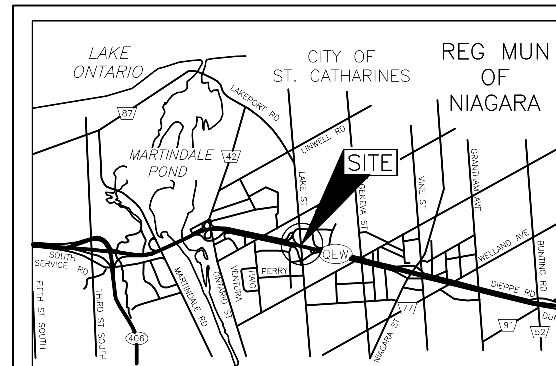
VERT. SCALE 5 0 5 10 m



D-D' CROSS SECTION SOUTH ABUTMENT

HORIZ. SCALE 10 0 10 20 m

VERT. SCALE 5 0 5 10 m



KEY PLAN
SCALE 1 0 1 km

LEGEND

●	Borehole – Current 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 in piezometer, measured on November 1 and October 14, 2004
▽	WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
301	96.4	4782009.1	325032.6
302	101.3	4781993.2	325020.0
303	97.2	4781989.3	325035.3
304	96.8	4781963.0	325029.0
305	96.2	4781963.0	325039.3
306	96.8	4781946.9	325034.1
307	97.2	4781924.6	325033.3
308	96.4	4781919.0	325040.5
309	101.3	4781899.9	325024.3
310	96.4	4781895.4	325041.3
311	99.8	4781875.5	325039.1

NOTES

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

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F. J. HEFFERNAN – REGISTERED PROFESSIONAL ENGINEER – PROVINCE OF ONTARIO

L.C. COYNE – LICENSED PROFESSIONAL ENGINEER – PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

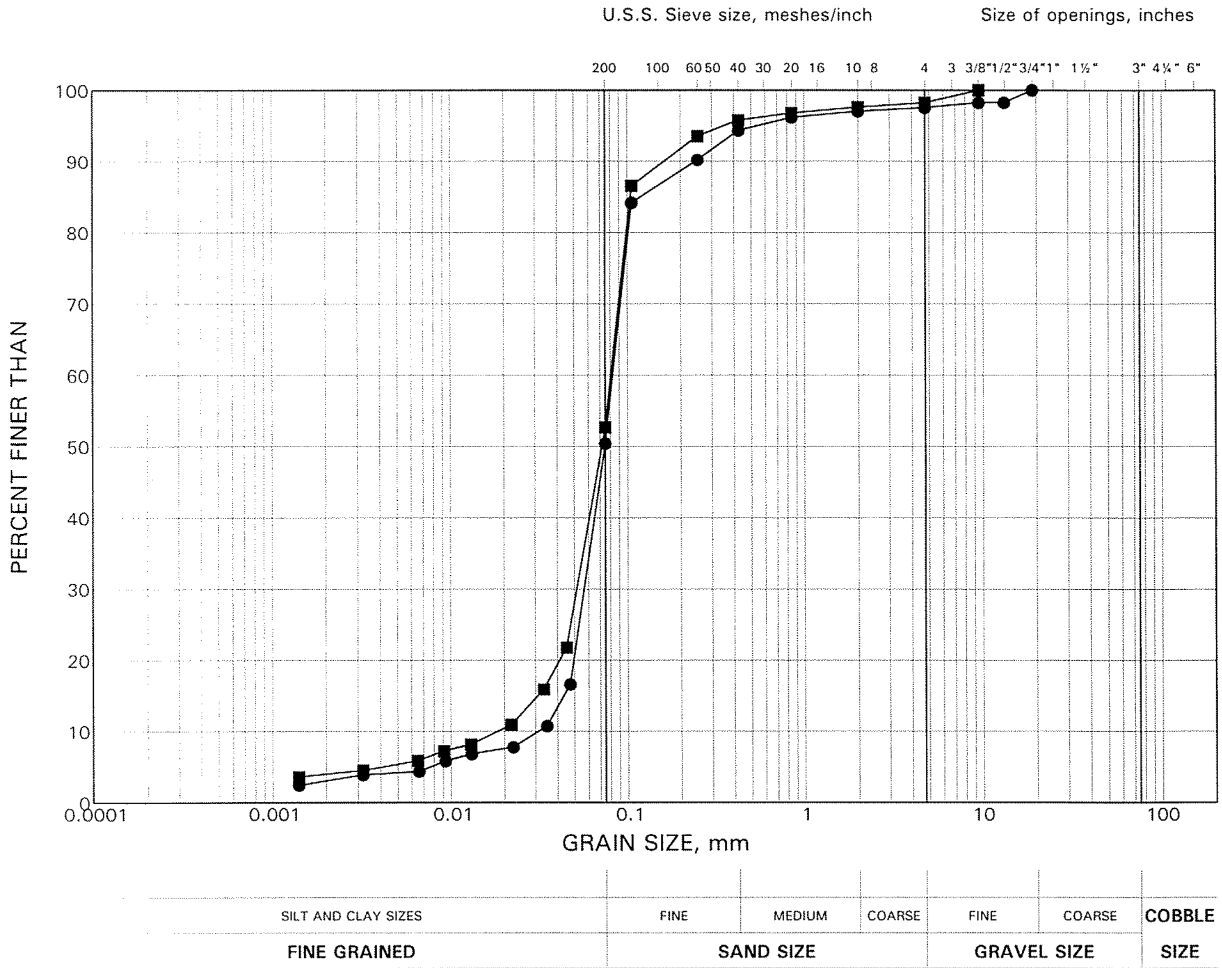
Geocres No.

HWY. QEW		PROJECT NO. 04–1111–002	DIST.
SUBM'D. SLP	CHKD. LCC	DATE: OCT 2006	SITE: 18–105
DRAWN: MSM	CHKD. SLP	APPD. LCC	DWG. 2

PLOT DATE: October 16, 2006
FILENAME: F:\Projects\2004\04-1111-002 (MH, StCatharines)\--BD-- (LAKE STREET GAL BP7)\04111002BD002.dwg

GRAIN SIZE DISTRIBUTION TEST RESULTS
Surficial Silty Sand to Sand and Silt

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	307	3	95.4
■	309	8	95.1

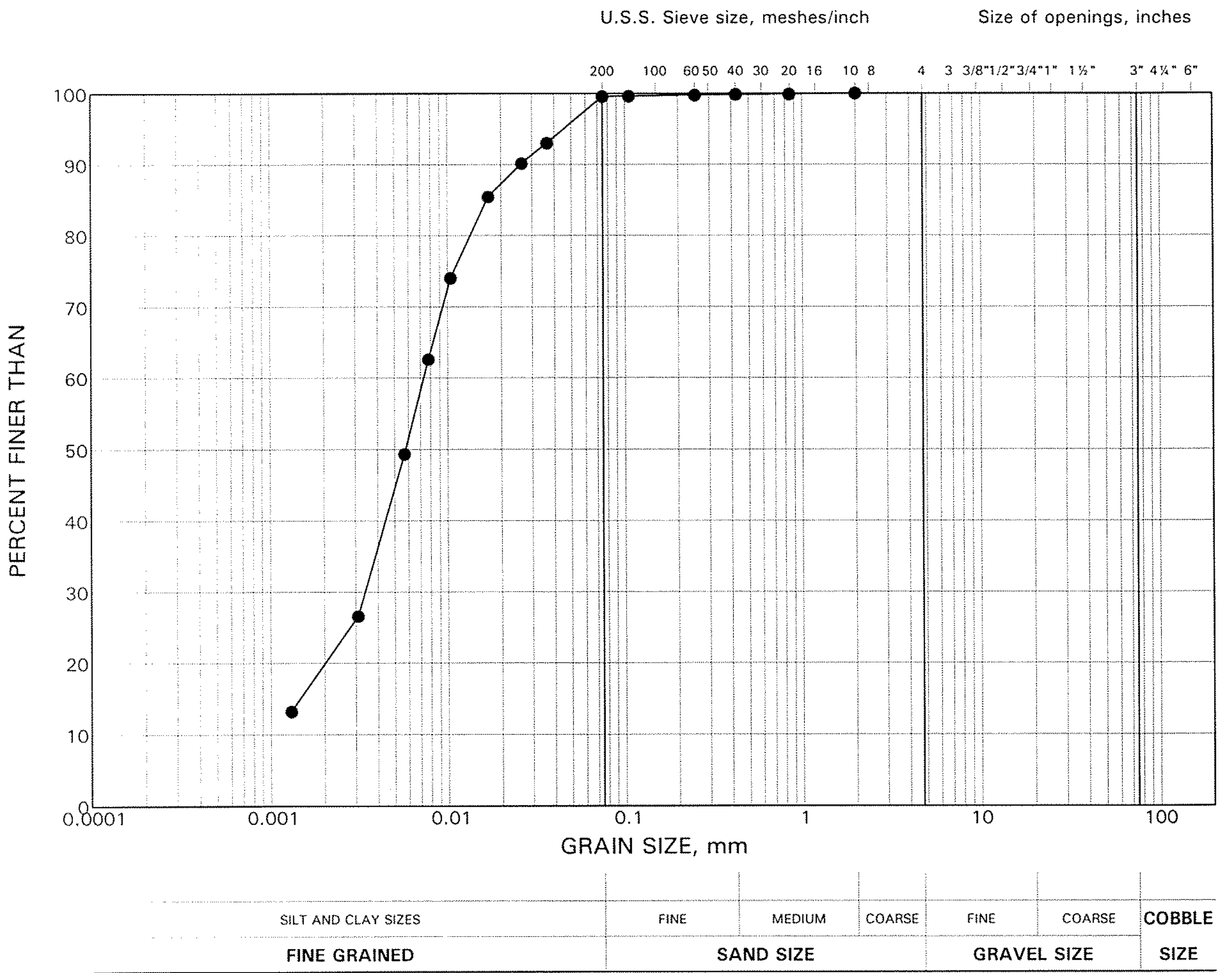
Date August, 2005
Project 04-1111-002-3

Golder Associates

Prepared by LG
Checked by

GRAIN SIZE DISTRIBUTION TEST RESULT
Surficial Clayey Silt

FIGURE 2



LEGEND

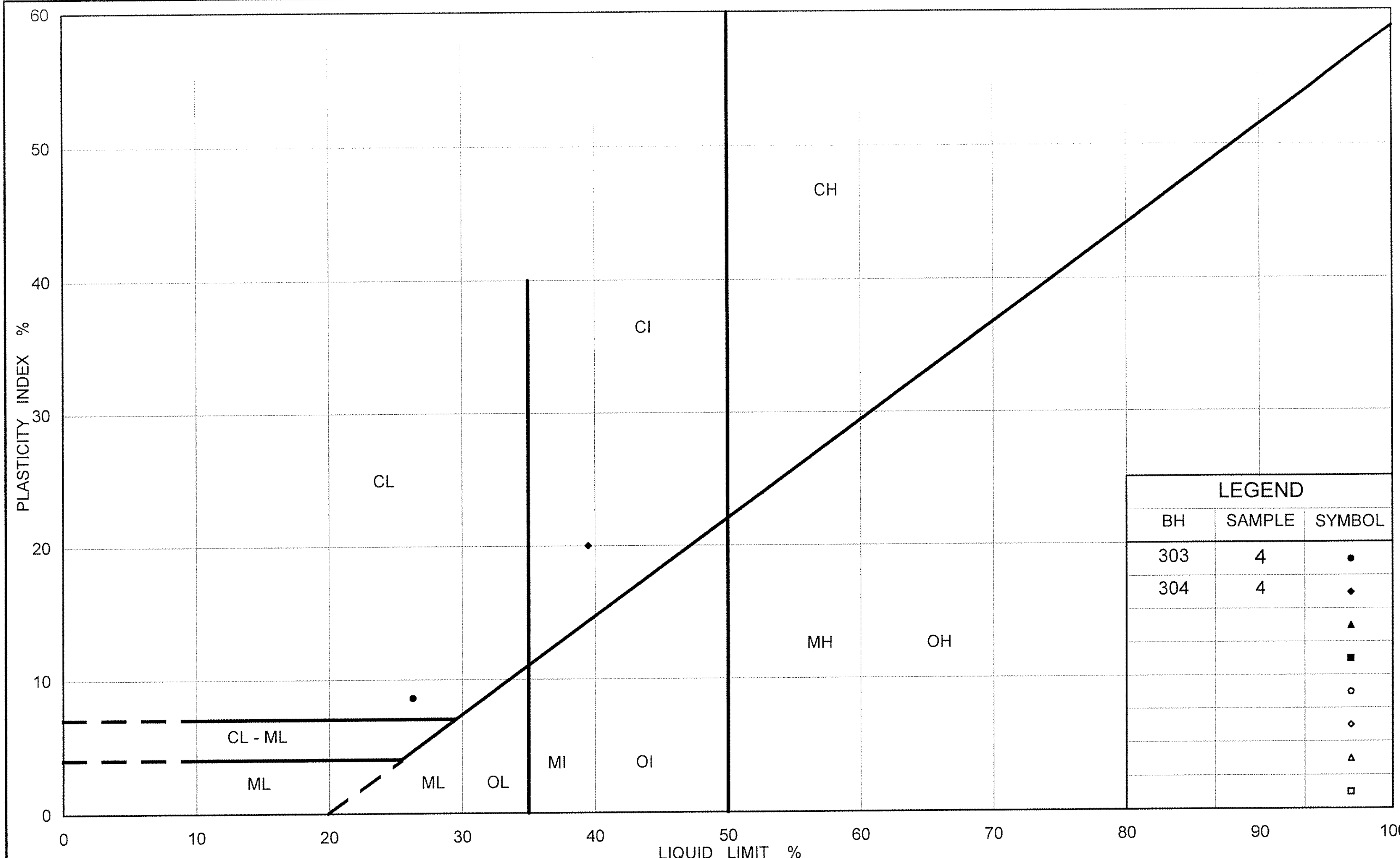
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	303	4	94.9

Date August, 2005
Project 04-1111-002-3

Golder Associates

Prepared by LG
Checked by [signature]

Oct 75, FF-S-21



Ministry of Transportation

Ontario

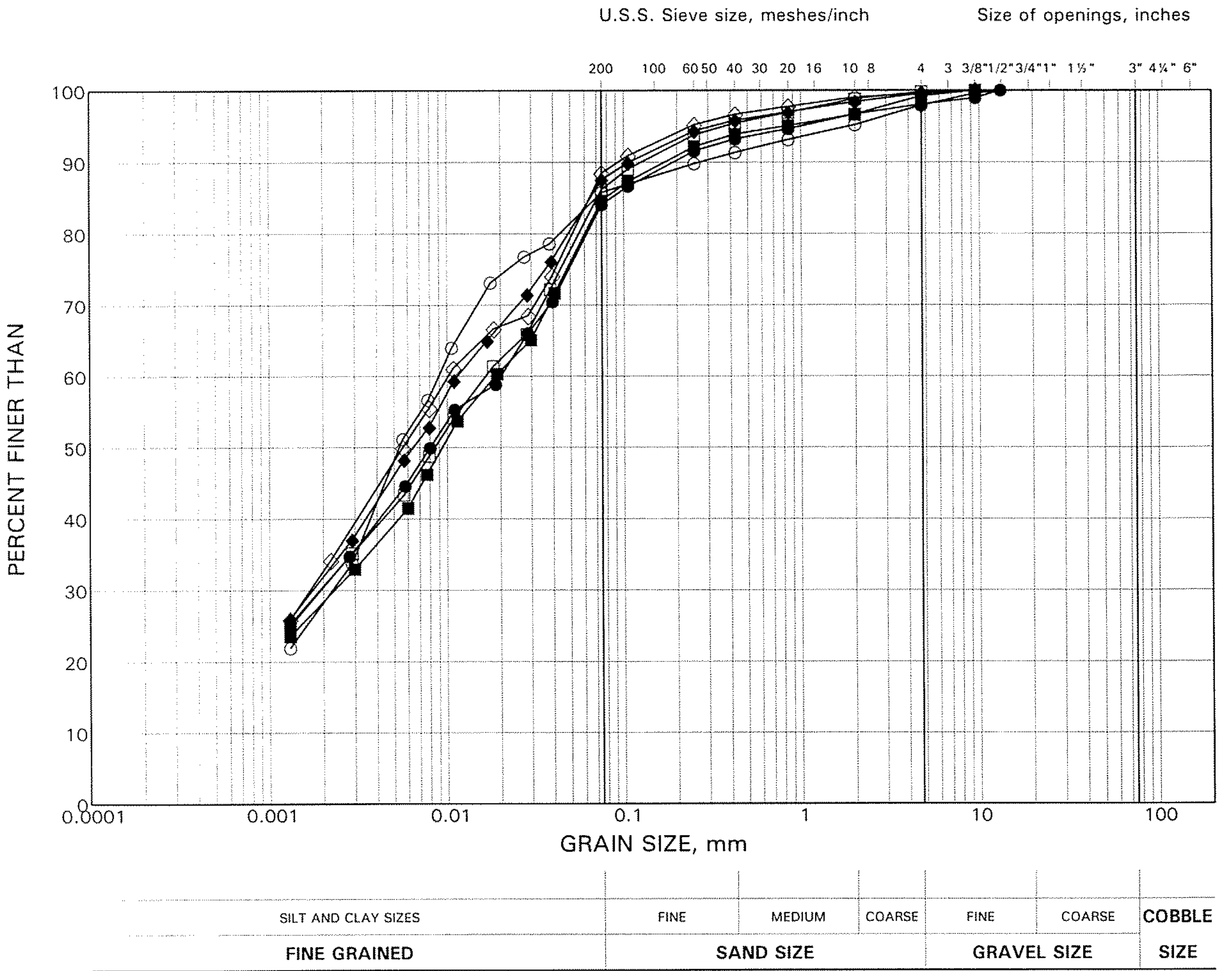
PLASTICITY CHART
Surficial Clayey Silt to Silty Clay

FIG No. 3

Project No. 04-1111-002-3

GRAIN SIZE DISTRIBUTION TEST RESULTS
Upper Clayey Silt Till

FIGURE 4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	301	4	95.8
■	302	13	87.3
◆	303	8	90.8
○	304	14	81.3
□	306	9	88.9
◇	311	5	96.4

Date August, 2005
Project 04-1111-002-3

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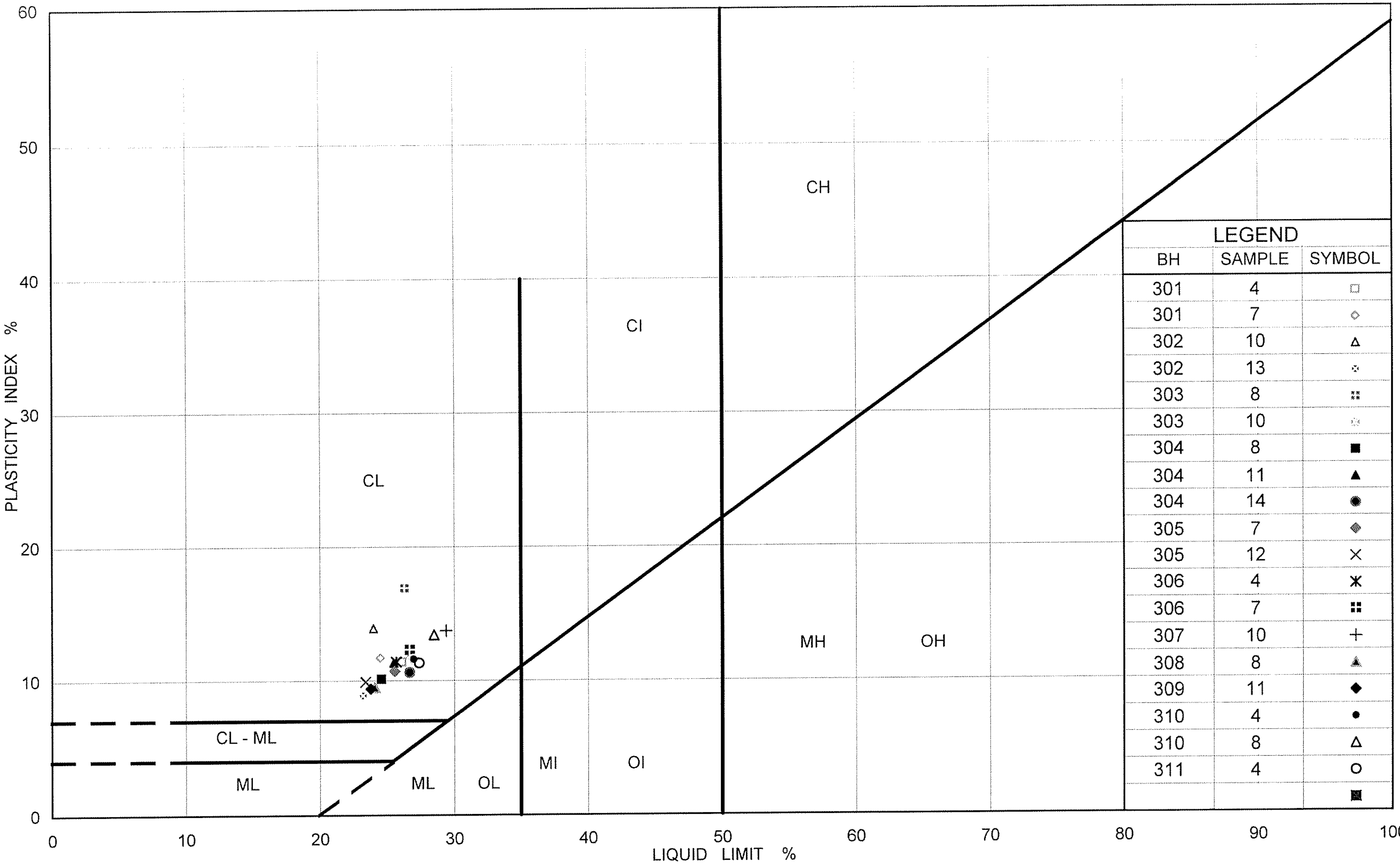
Checked by [signature]

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Upper Clayey Silt Till

FIG No. 5

Project No. 04-1111-002-3

CONSOLIDATION TEST SUMMARY
UPPER CLAYEY SILT TILL

FIGURE 6A

SAMPLE IDENTIFICATION

Project Number	04-1111-002	Sample Number	7
Borehole Number	305	Sample Depth, m	4.6-5.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	12/23/2004		
Date Completed	01/11/2005		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m^3	21.10
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	17.82
Area, cm^2	31.61	Specific Gravity, measured	2.75
Volume, cm^3	60.53	Solids Height, cm	1.265
Water Content, %	18.40	Volume of Solids, cm^3	39.99
Wet Mass, g	130.22	Volume of Voids, cm^3	20.54
Dry Mass, g	109.98	Degree of Saturation, %	98.5

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.915	0.514	1.915				
4.86	1.913	0.512	1.914	45	1.73E-02	2.15E-04	3.63E-07
9.52	1.909	0.509	1.911	135	5.73E-03	4.48E-04	2.52E-07
19.44	1.903	0.504	1.906	146	5.28E-03	3.16E-04	1.63E-07
38.71	1.893	0.496	1.898	103	7.41E-03	2.71E-04	1.97E-07
77.58	1.875	0.482	1.884	184	4.09E-03	2.42E-04	9.69E-08
154.95	1.849	0.461	1.862	240	3.06E-03	1.75E-04	5.27E-08
38.71	1.859	0.469	1.854			4.49E-05	
9.52	1.873	0.480	1.866			2.50E-04	
4.86	1.880	0.486	1.877			7.84E-04	
9.52	1.878	0.484	1.879	53	1.41E-02	2.24E-04	3.10E-07
19.44	1.874	0.481	1.876	146	5.11E-03	2.11E-04	1.05E-07
38.71	1.867	0.476	1.871	204	3.64E-03	1.90E-04	6.76E-08
77.58	1.858	0.469	1.863	135	5.45E-03	1.21E-04	6.45E-08
154.95	1.843	0.457	1.851	240	3.02E-03	1.01E-04	3.00E-08
309.27	1.812	0.432	1.828	171	4.14E-03	1.05E-04	4.26E-08
619.02	1.768	0.397	1.790	171	3.97E-03	7.42E-05	2.89E-08
1238.75	1.719	0.359	1.744	197	3.27E-03	4.13E-05	1.32E-08
2478.49	1.665	0.316	1.692	184	3.30E-03	2.27E-05	7.35E-09
1238.75	1.673	0.322					
309.27	1.695	0.340					
77.57	1.725	0.363					
19.44	1.755	0.387					
4.86	1.779	0.406					

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

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

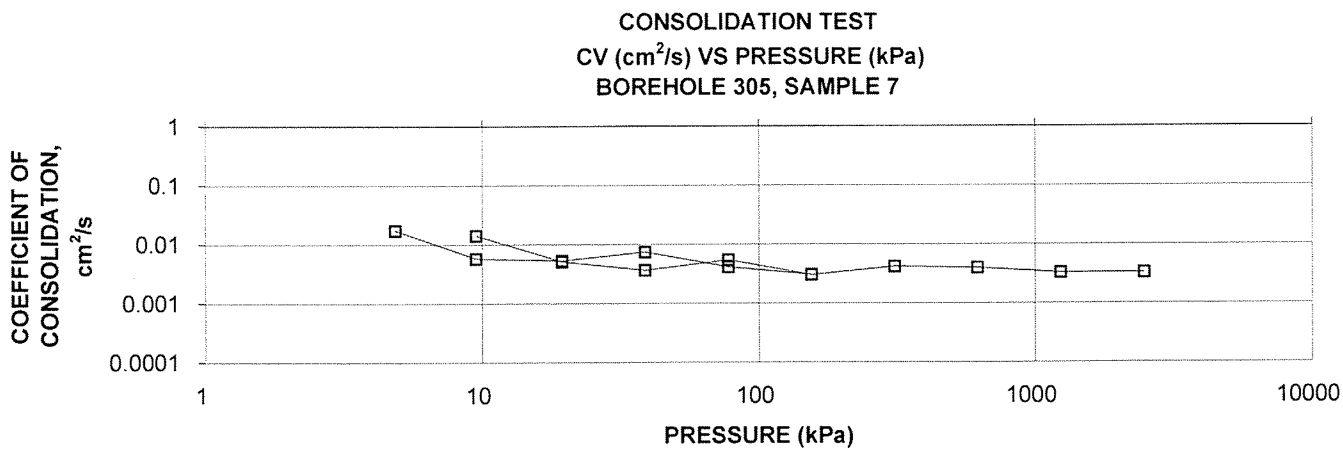
Sample Height, cm	1.76	Unit Weight, kN/m^3	22.61
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	19.44
Area, cm^2	31.61	Specific Gravity, measured	2.75
Volume, cm^3	55.47	Solids Height, cm	1.265
Water Content, %	16.28	Volume of Solids, cm^3	39.99
Wet Mass, g	127.89	Volume of Voids, cm^3	15.48
Dry Mass, g	109.98		

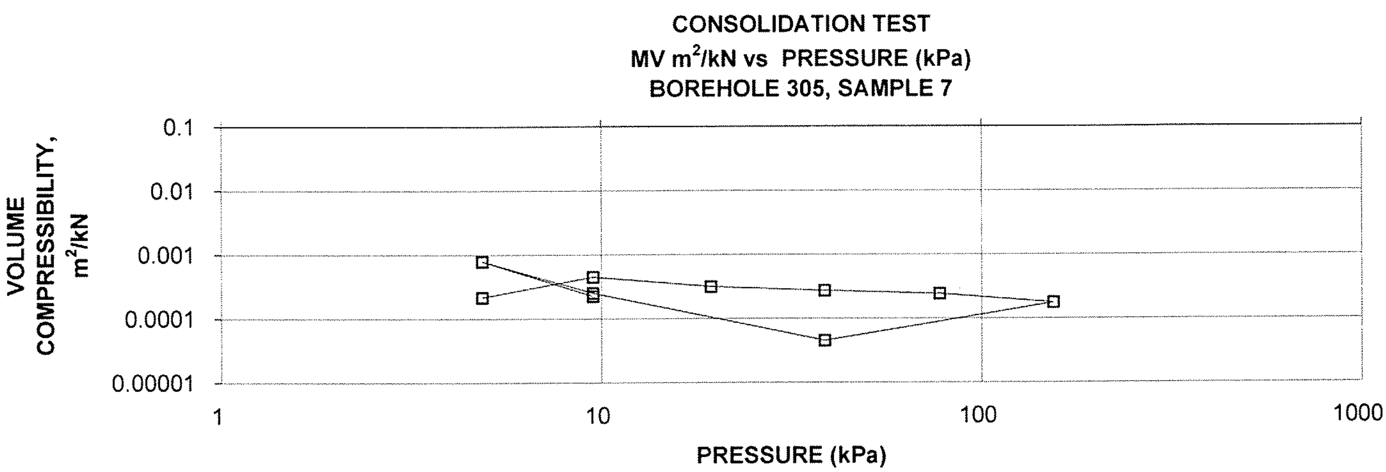
Prepared By: LFG

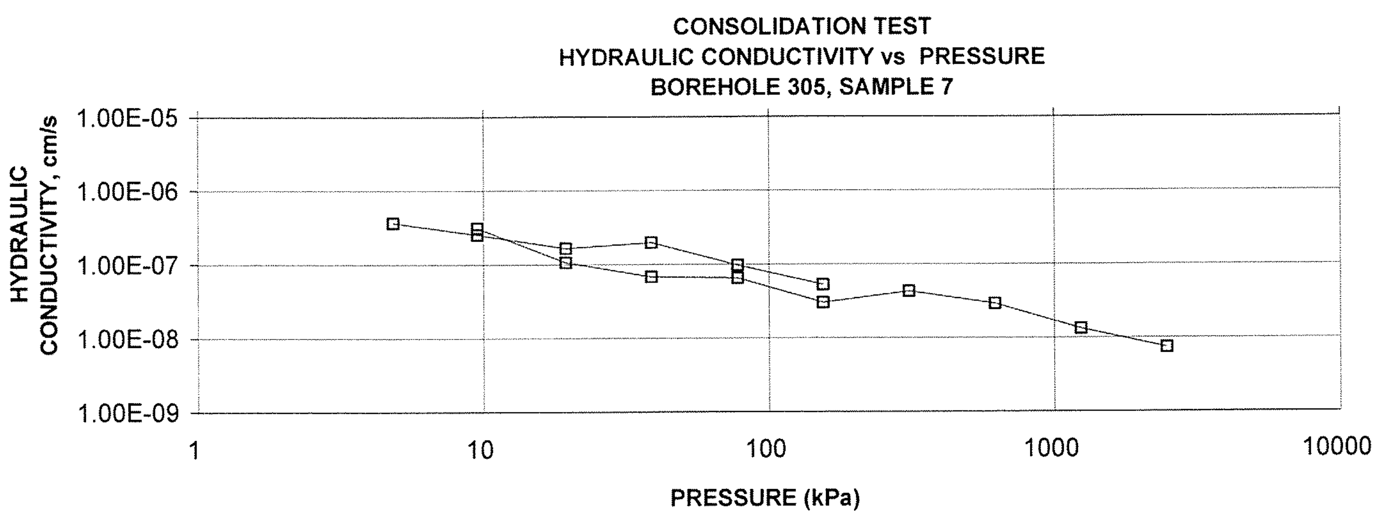
Golder Associates

Checked By: MM

CONSOLIDATION TEST SUMMARY UPPER CLAYEY SILT TILL	FIGURE 6B





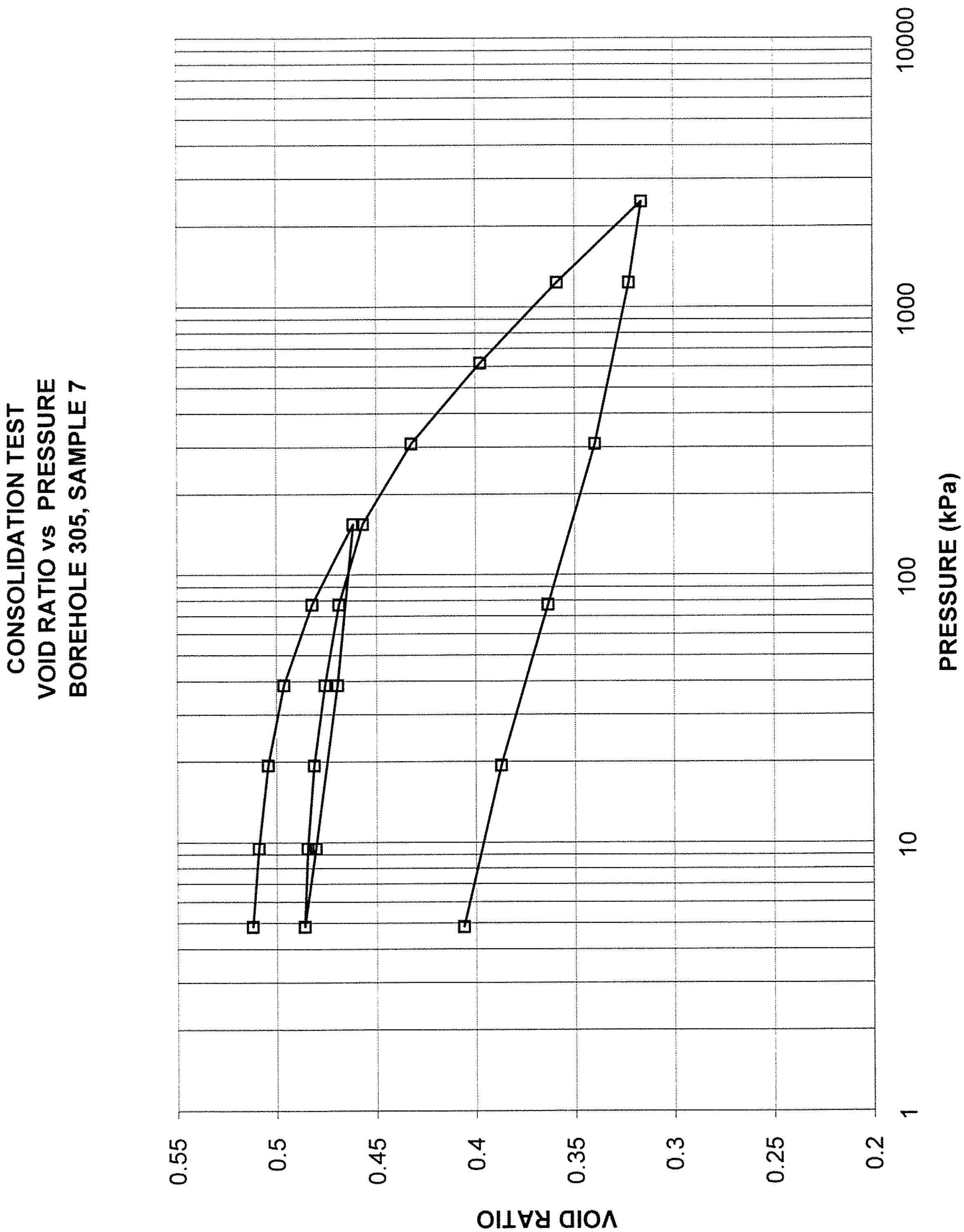


Project No. 04-1111-002
Prepared By: LFG

Golder Associates

Checkey By: MM

CONSOLIDATION TEST RESULTS UPPER CLAYEY SILT TILL	FIGURE 6C



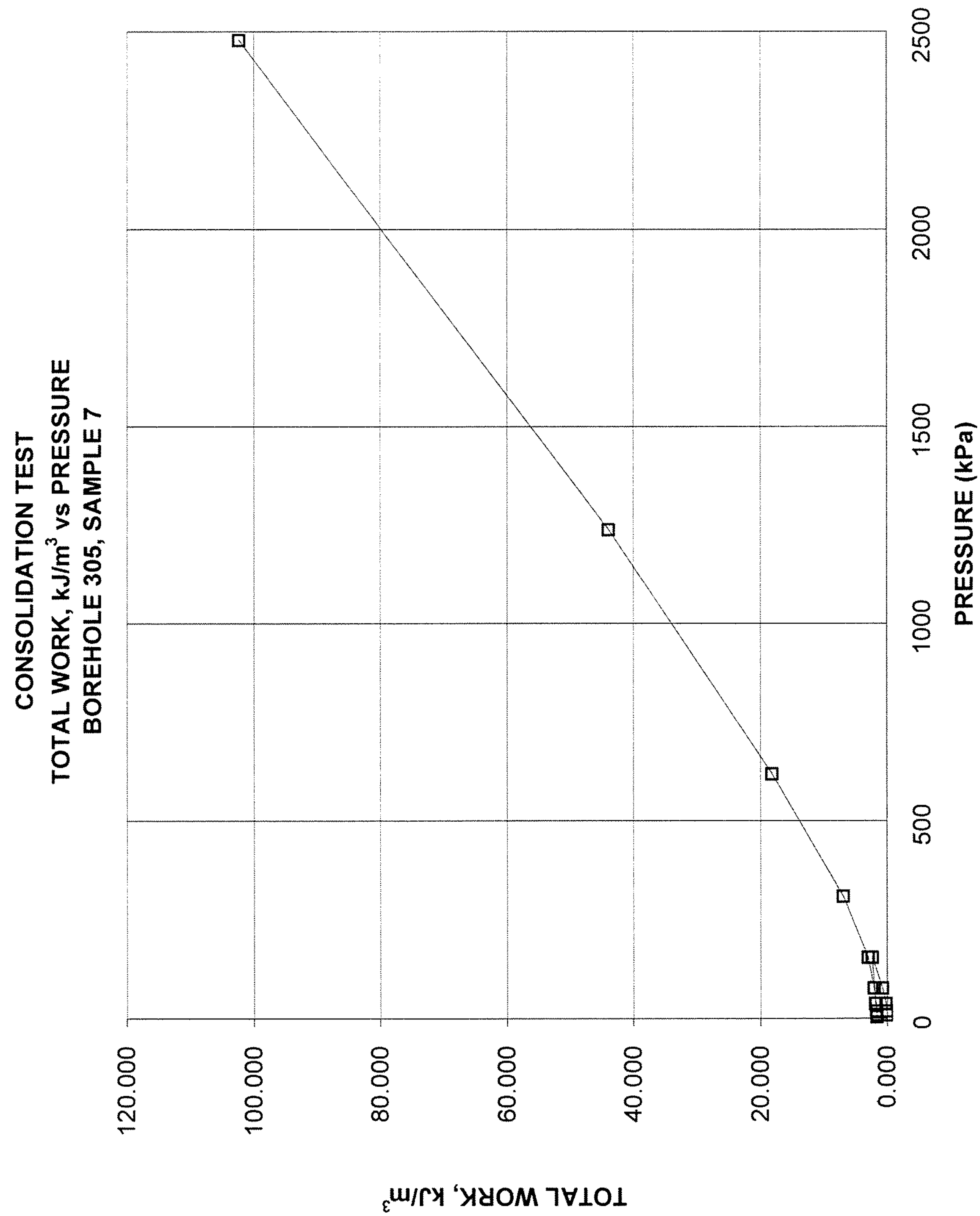
Project No. 04-1111-002

Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST RESULTS UPPER CLAYEY SILT TILL	FIGURE 6D



Project No. 04-1111-002

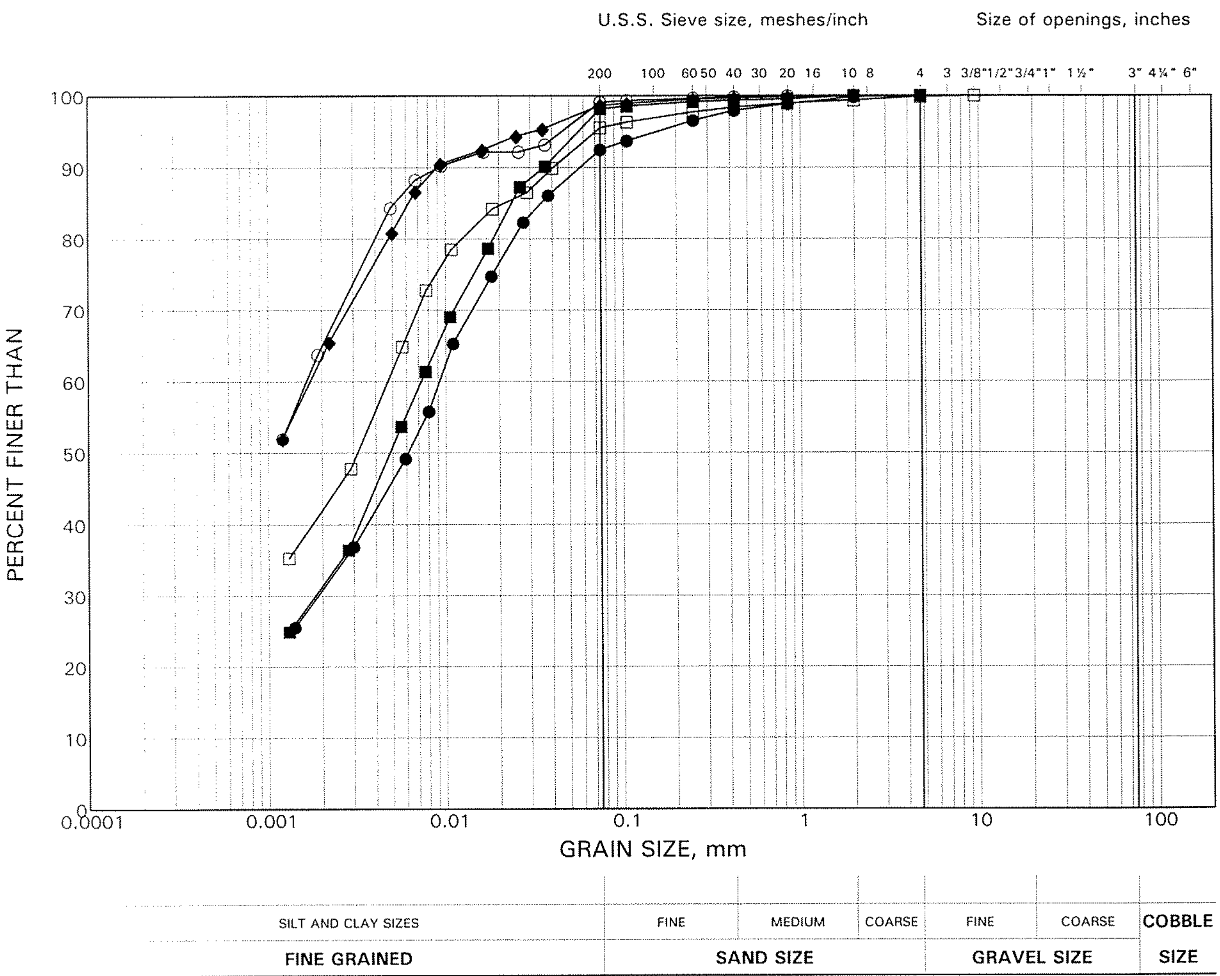
Golder Associates

Prepared By: LFG

Checked By: MM

GRAIN SIZE DISTRIBUTION TEST RESULTS
Clayey Silt to Silty Clay

FIGURE 7



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	303	15	80.1
■	305	15	79.1
◆	307	12	84.7
○	308	11	85.4
□	309	18	79.7

Date August, 2005

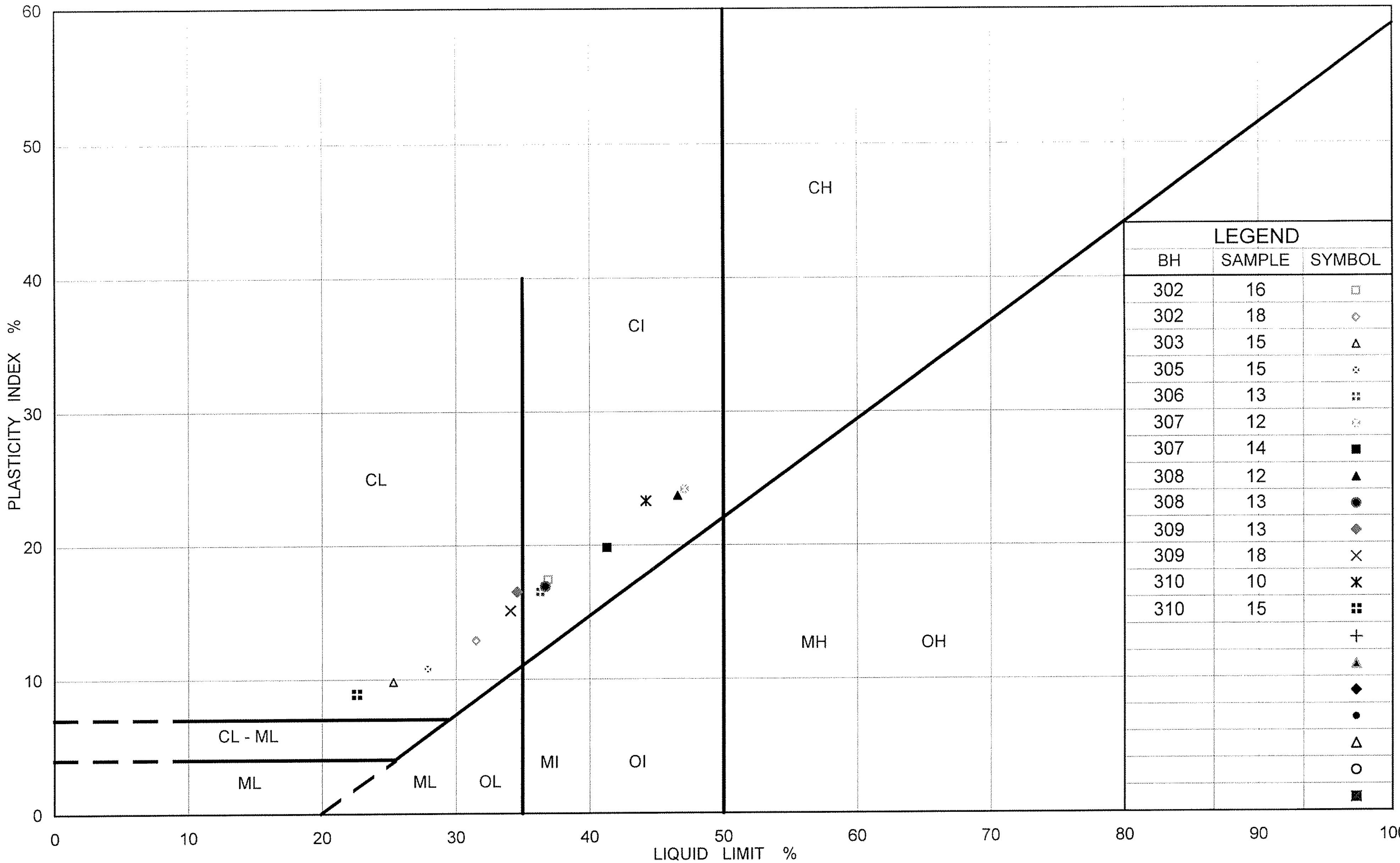
Project 04-1111-002-3

Golder Associates

Prepared by LG

Checked by [signature]

Oct 75. FF-S-21



Ministry of Transportation Ontario	**PLASTICITY CHART** Clayey Silt to Silty Clay	FIG No. 8 Project No. 04-1111-002-3

CONSOLIDATION TEST SUMMARY
CLAYEY SILT TO SILTY CLAY

FIGURE 9A

SAMPLE IDENTIFICATION

Project Number	04-1111-002	Sample Number	13
Borehole Number	308	Sample Depth, m	13.7-14.3

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	01/12/2005		
Date Completed	01/28/2005		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m^3	19.52
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	15.15
Area, cm^2	31.61	Specific Gravity, measured	2.70
Volume, cm^3	60.53	Solids Height, cm	1.095
Water Content, %	28.90	Volume of Solids, cm^3	34.63
Wet Mass, g	120.51	Volume of Voids, cm^3	25.91
Dry Mass, g	93.49	Degree of Saturation, %	104.3

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.915	0.748	1.915				
4.86	1.912	0.745	1.914	10	7.76E-02	3.22E-04	2.45E-06
9.52	1.909	0.743	1.911	85	9.10E-03	3.36E-04	3.00E-07
19.44	1.901	0.735	1.905	271	2.84E-03	4.21E-04	1.17E-07
38.71	1.888	0.724	1.895	321	2.37E-03	3.52E-04	8.18E-08
77.58	1.870	0.707	1.879	171	4.38E-03	2.42E-04	1.04E-07
155.07	1.845	0.684	1.858	146	5.01E-03	1.68E-04	8.27E-08
38.71	1.858	0.696	1.852			5.83E-05	
9.52	1.876	0.713	1.867			3.22E-04	
4.86	1.885	0.721	1.881			1.01E-03	
9.52	1.883	0.719	1.884	146	5.15E-03	2.24E-04	1.13E-07
19.44	1.878	0.714	1.881	263	2.85E-03	2.63E-04	7.35E-08
38.71	1.869	0.706	1.874	279	2.67E-03	2.44E-04	6.37E-08
77.58	1.856	0.694	1.863	158	4.65E-03	1.75E-04	7.97E-08
155.07	1.839	0.679	1.848	99	7.31E-03	1.15E-04	8.21E-08
310.05	1.789	0.633	1.814	177	3.94E-03	1.68E-04	6.51E-08
620.19	1.702	0.554	1.746	475	1.36E-03	1.46E-04	1.95E-08
1238.92	1.610	0.470	1.656	375	1.55E-03	7.76E-05	1.18E-08
2478.66	1.520	0.388	1.565	240	2.16E-03	3.79E-05	8.04E-09
1238.92	1.532	0.399					
310.05	1.564	0.428					
77.58	1.610	0.470					
19.44	1.662	0.517					
4.86	1.698	0.550					

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

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

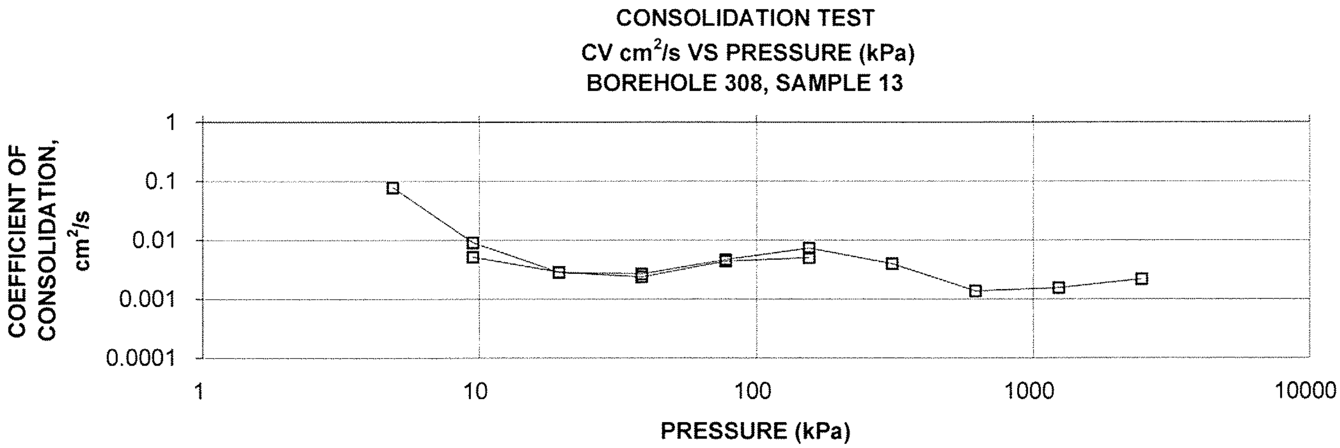
Sample Height, cm	1.70	Unit Weight, kN/m^3	21.54
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	17.08
Area, cm^2	31.61	Specific Gravity, measured	2.70
Volume, cm^3	53.67	Solids Height, cm	1.095
Water Content, %	26.10	Volume of Solids, cm^3	34.63
Wet Mass, g	117.89	Volume of Voids, cm^3	19.05
Dry Mass, g	93.49		

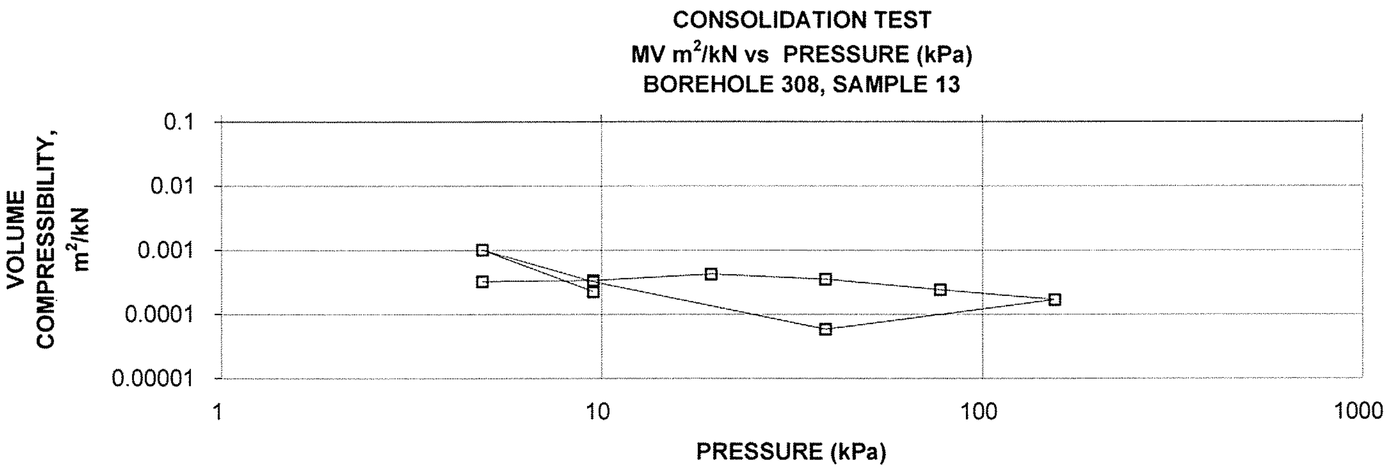
Prepared By: LFG

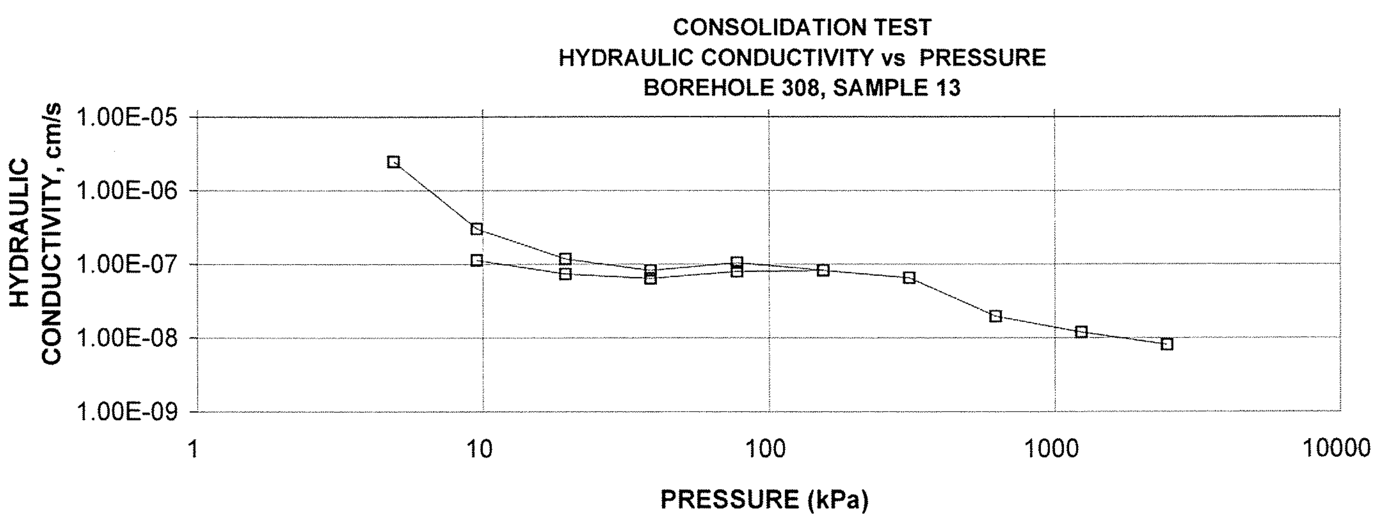
Golder Associates

Checked By: MM

CONSOLIDATION TEST SUMMARY CLAYEY SILT TO SILTY CLAY	FIGURE 9B





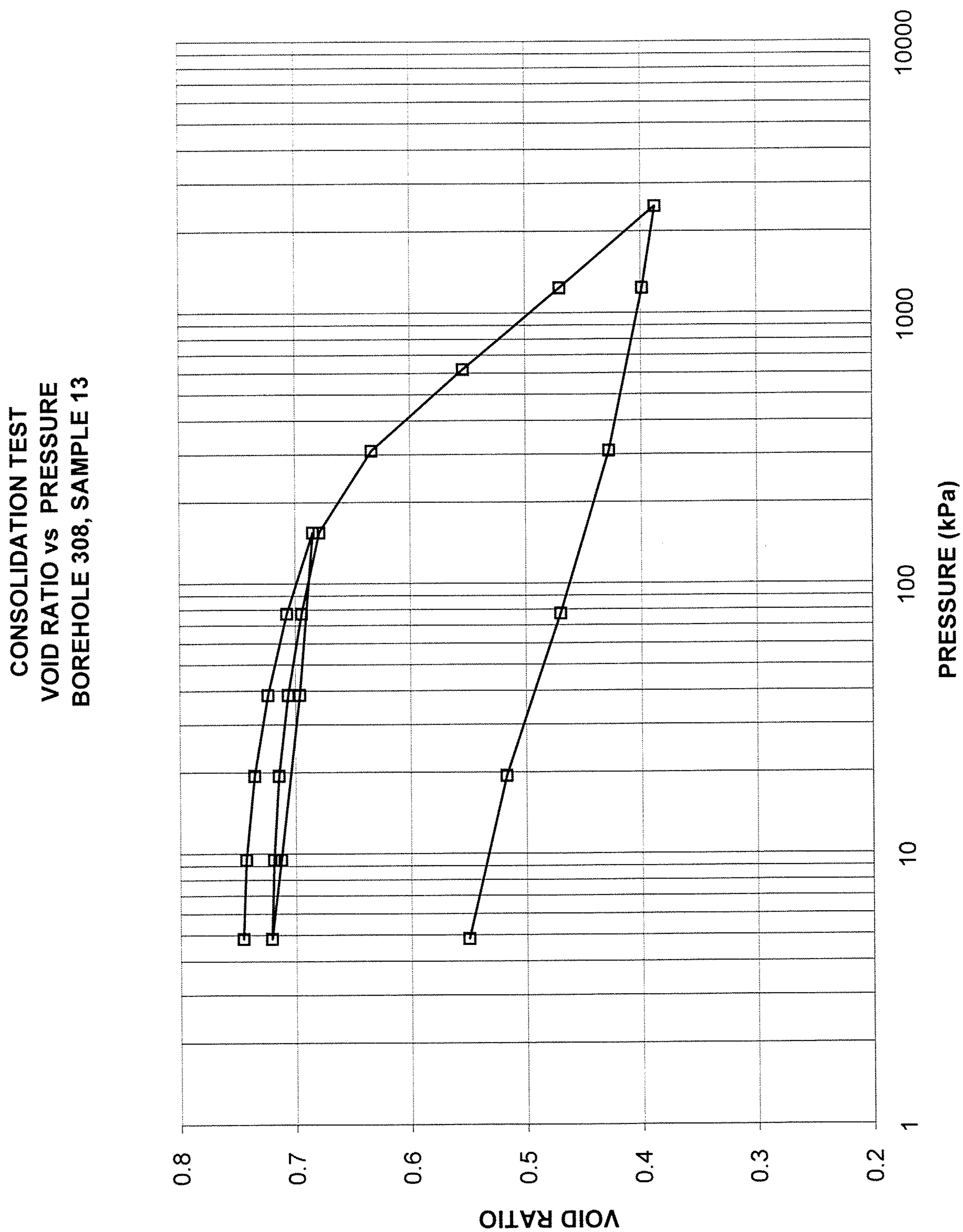


Project No. 04-1111-002
Prepared By: LFG

Golder Associates

Checkey By: MM

CONSOLIDATION TEST RESULTS CLAYEY SILT TO SILTY CLAY	FIGURE 9C



Project No. 04-1111-002

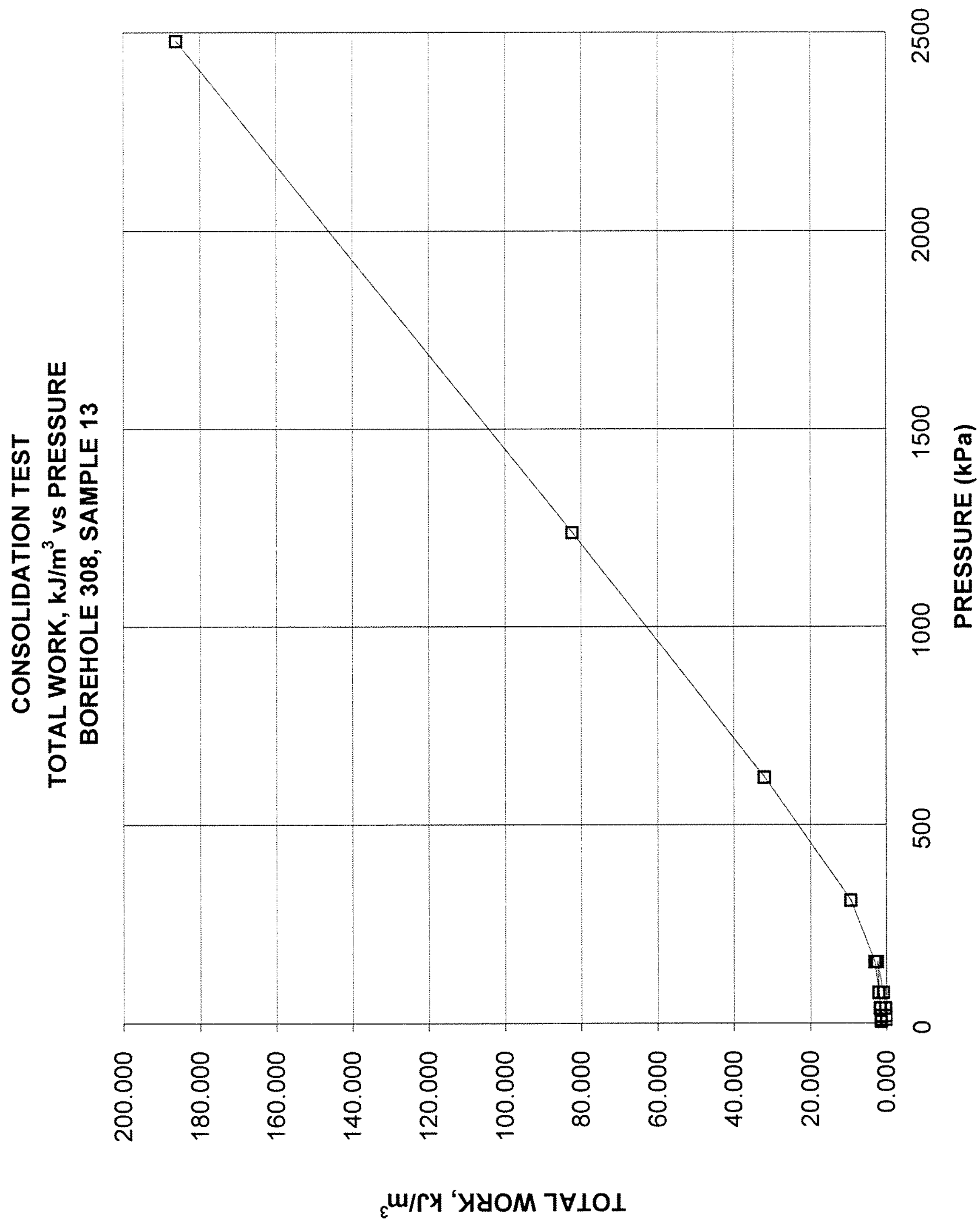
Prepared By: LFG

Golder Associates

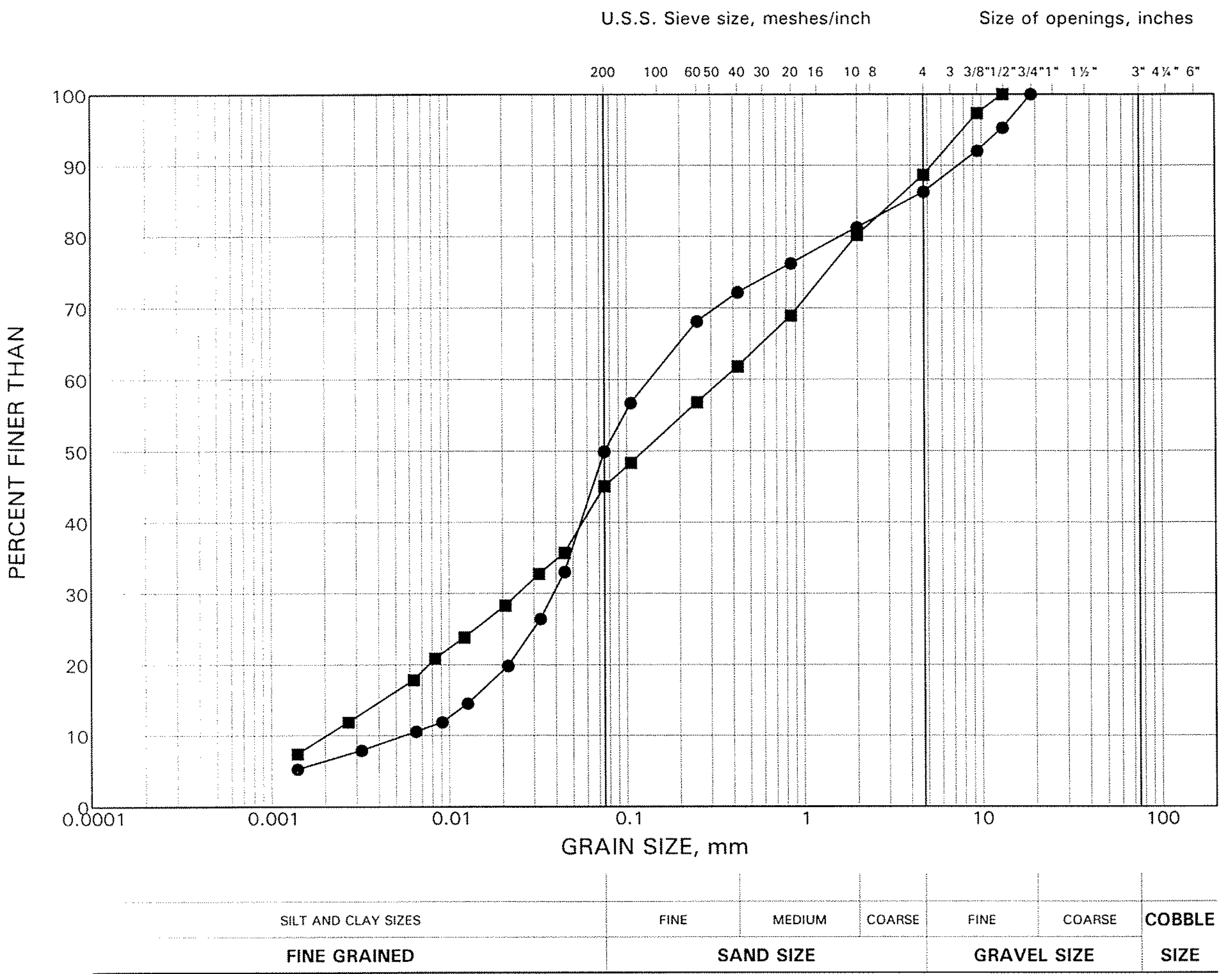
Checked By: MM

CONSOLIDATION TEST RESULTS
CLAYEY SILT TO SILTY CLAY

FIGURE 9D



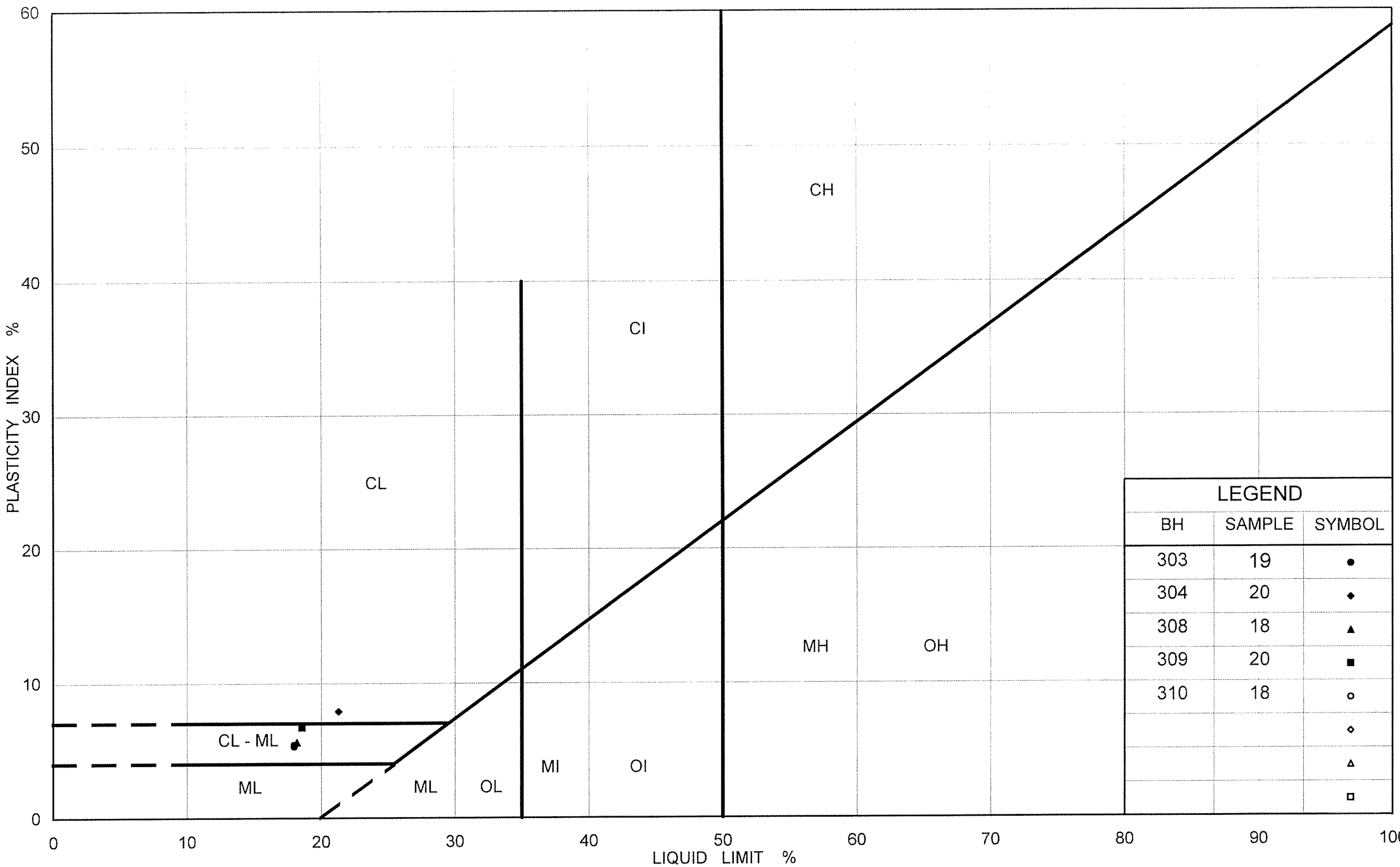
GRAIN SIZE DISTRIBUTION TEST RESULTS Lower Sand and Silt Till	FIGURE 10



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	304	18	75.4
■	306	15	79.7

Oct 75. FF-S-21



BH	SAMPLE	SYMBOL
303	19	●
304	20	◆
308	18	▲
309	20	■
310	18	○
		◇
		△
		□

Ministry of Transportation

Ontario

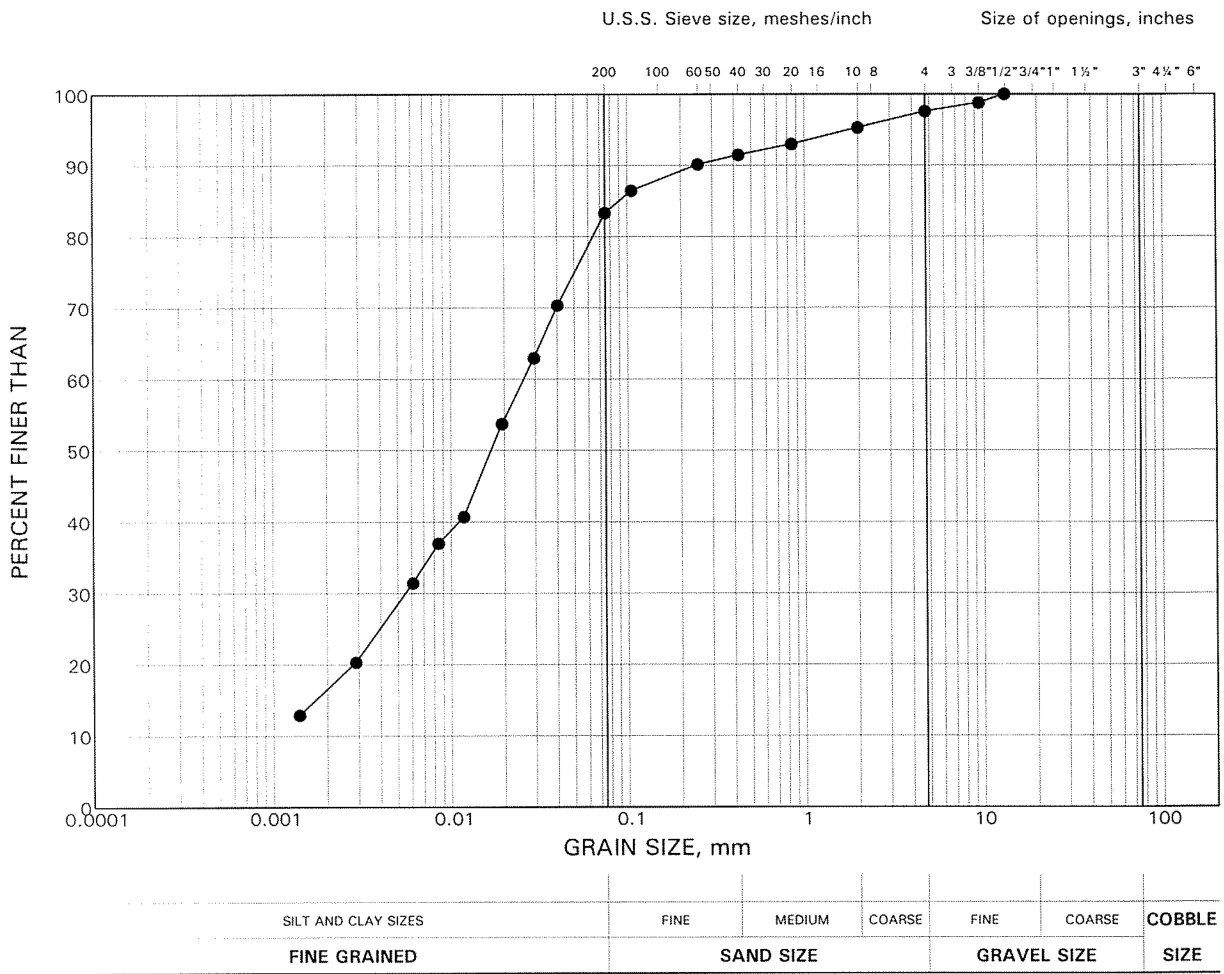
PLASTICITY CHART
Lower Clayey Silt Till

FIG No. 11

Project No. 04-1111-002-3

GRAIN SIZE DISTRIBUTION TEST RESULT
Clayey Silt (Residual Soil)

FIGURE 12



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	305	21	70.0

Date August, 2005

Project 04-1111-002-3

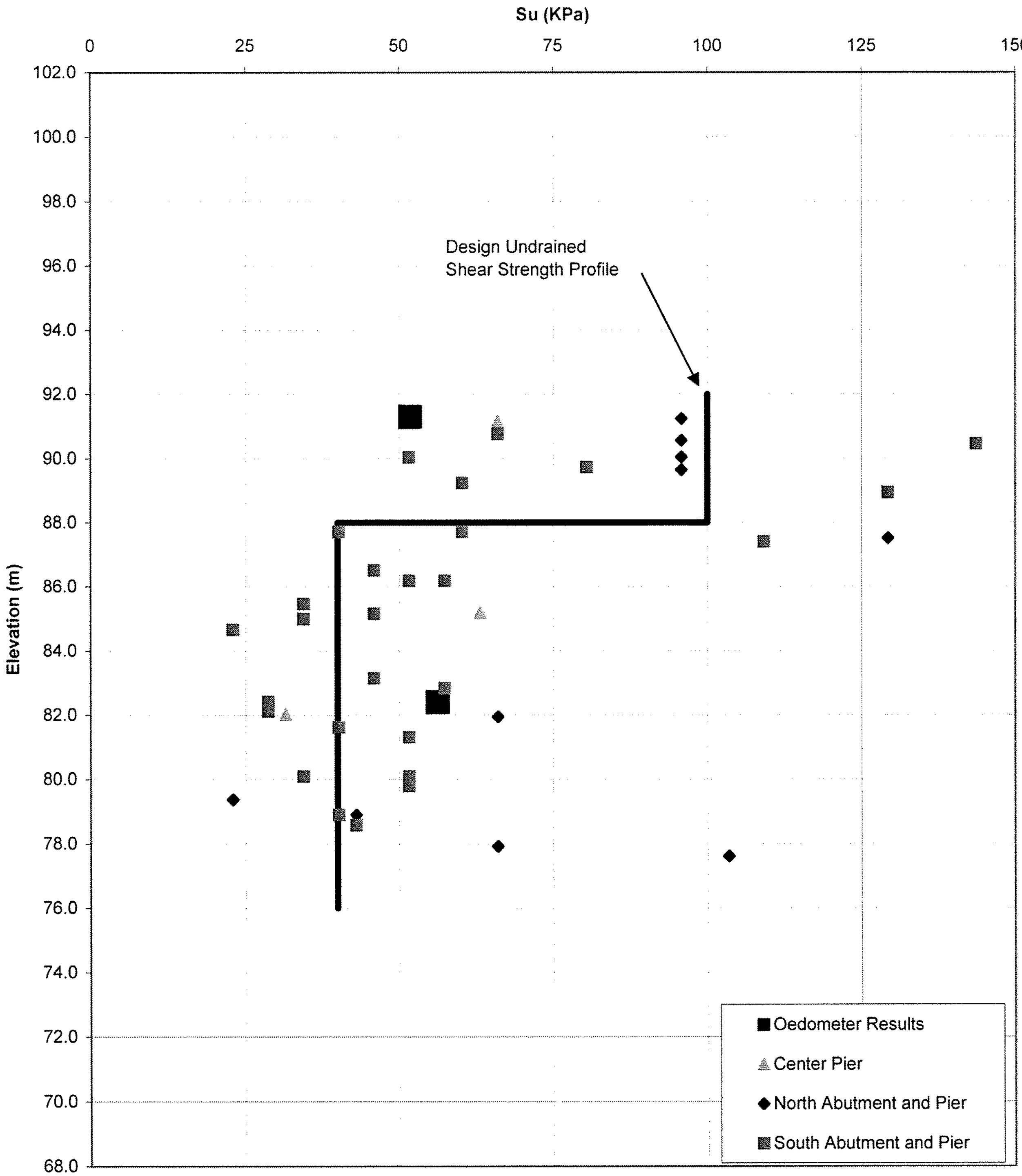
Golder Associates

Prepared by LG

Checked by

UNDRAINED SHEAR STRENGTH PROFILE
LAKE STREET UNDERPASS

FIGURE 13



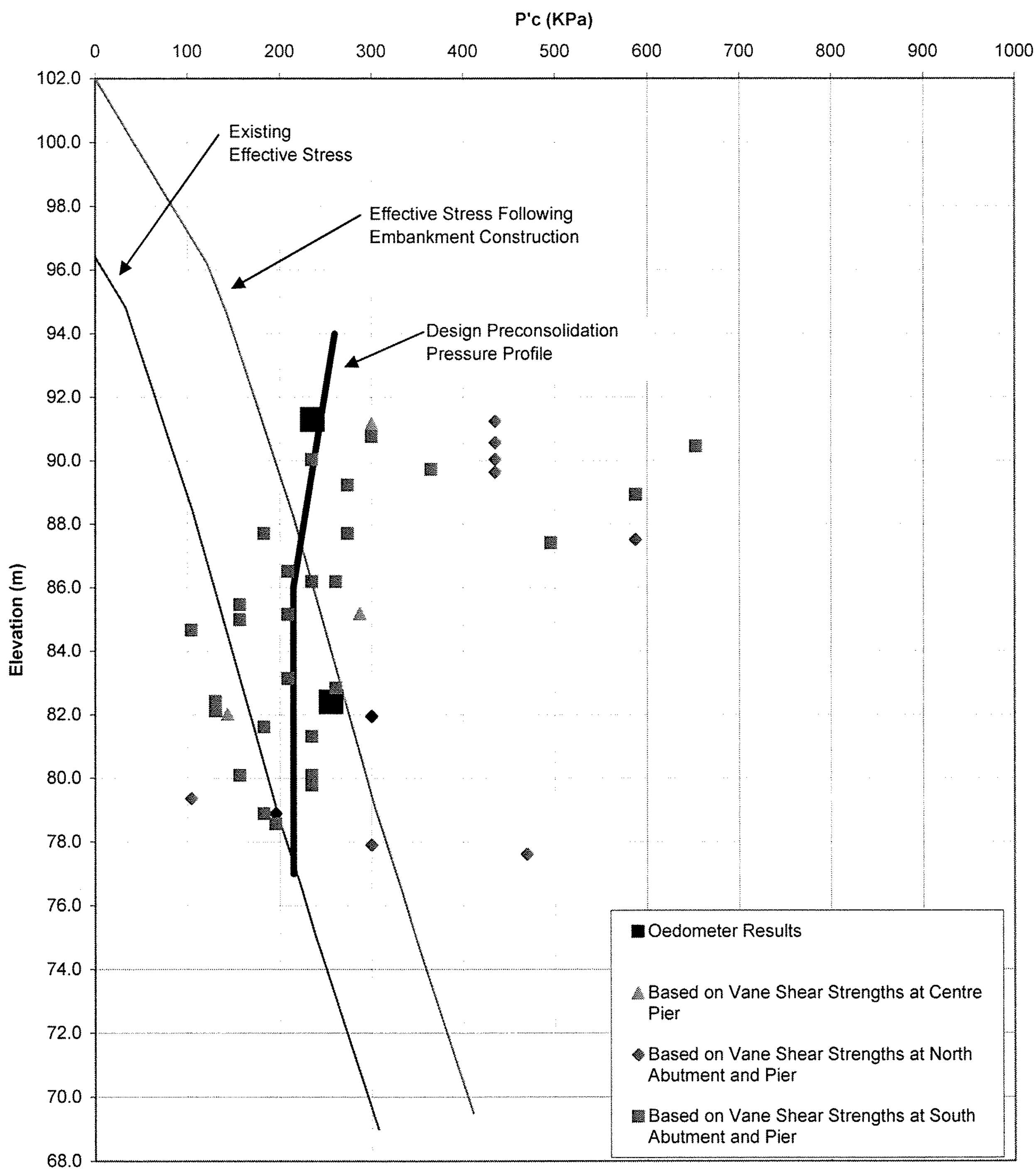
Date: September 2005
Project: 04-1111-002

Golder Associates

Drawn: SP
Checked: LCC

PRECONSOLIDATION PRESSURE PROFILE
LAKE STREET UNDERPASS

FIGURE 14



Date: September 2005
Project: 04-1111-002

Golder Associates

Drawn: SP
Checked: LCC

APPENDIX A

NON-STANDARD SPECIAL PROVISIONS

BOULDERS/OBSTRUCTIONS DURING PILE INSTALLATION - Item No.

Special Provision

The soils at the site are glacially-derived and should be expected to contain cobbles and boulders. Appropriate equipment and procedures will be required to penetrate obstructions (cobbles and boulders) that are encountered during pile driving.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

SETTLEMENT PLATES - Item No.

Non-Standard Special Provision **September 2006**

1.0 GENERAL

1.1 Scope

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

1.1.2 The purpose of the settlement plates is to monitor the progress of settlement under the widened Lake Street embankment. Settlement is measured by survey of the top of the rod with reference to stable, non-settling benchmarks.

1.1.3 The timing for final paving of the Lake Street approach embankments and construction of the final concrete curb and sidewalk will be controlled by the instrumentation readings. The settlement monitoring shall be carried out for a minimum period of twelve months, and up to eighteen months depending on the results of the settlement monitoring.

1.2 General Procedure

1.2.1 Rods shall be attached to a settlement plate at existing ground level following any necessary subexcavation/subgrade preparation. As the Lake Street embankment construction proceeds, the rods shall be extended above the new ground level.

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

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

1.3 Locations

The Contractor shall install the settlement plates at the approximate locations shown on the “Monitoring Instrument Location Plan and Typical Instrument Installation Detail” drawing contained elsewhere in this Contract. In general, the settlement plates are to be installed at approximately 20 m spacing along the east side of the embankment, out of the traffic path.

1.4 Notification

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

2.0 MATERIALS

2.1 General

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

2.2 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.

2.3 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.

2.4 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.

2.5 Friction Reducing Sleeve

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

2.6 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.

2.7 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.

3.0 INSTALLATIONS

3.1 Survey Benchmarks

3.1.1 The Contractor shall provide non-yielding, deep-seated survey benchmarks outside of the Lake Street embankment areas, and shall establish the geodetic elevation of each such benchmark.

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

3.2 Underground Utilities

3.2.1 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.

3.3 Settlement Plate

3.3.1 The settlement plate shall be installed horizontally on a 150 mm thick granular leveling pad placed on the ground surface following completion of any necessary subexcavation or subgrade preparation works.

3.3.2 The elevation of the base of the plate shall be surveyed by the Contractor before the placement of fill for the Lake Street embankment widening.

3.4 Rod

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

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

3.4.3 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.

3.5 Friction Reducing Sleeve

3.5.1 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.

3.6 Protective Surround

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

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

3.6.3 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.

3.7 Installation Details

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

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

3.7.3 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.

3.8 Marking and Labelling

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

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

3.9 Protection of Instruments

3.9.1 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.

4.0 MONITORING

4.1 Personnel / Access

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

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

4.2 Monitoring Program

4.2.1 The Contractor shall meet with the Contract Administrator and staff responsible for the ongoing monitoring immediately after installation of all of the instruments and before the start of embankment construction. 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.

4.2.2 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.

4.2.3 Monitoring by others for the baseline readings shall commence on the day following completion of installation of the instruments, and shall continue on a schedule to be determined by the Contract Administrator following construction of the embankments.

5.0 PAYMENT

5.1 Measurement for Payment

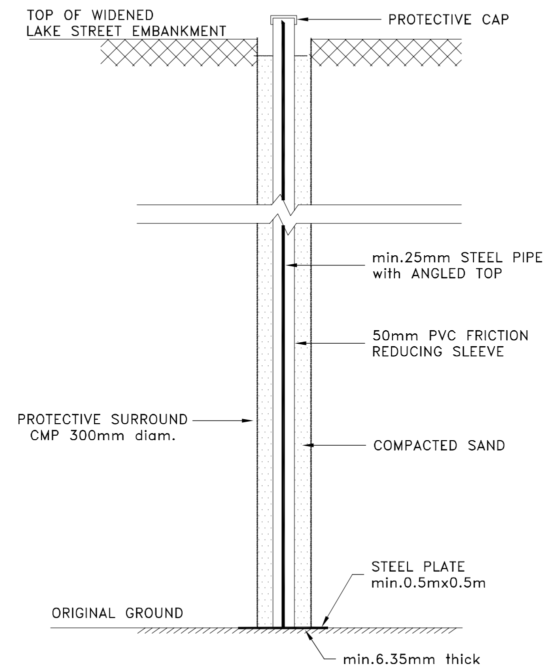
Measurement of the item, “Settlement Plates”, including all appurtenances, is by quantity. The unit of measurement is each.

5.2 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 plate and the required reporting.

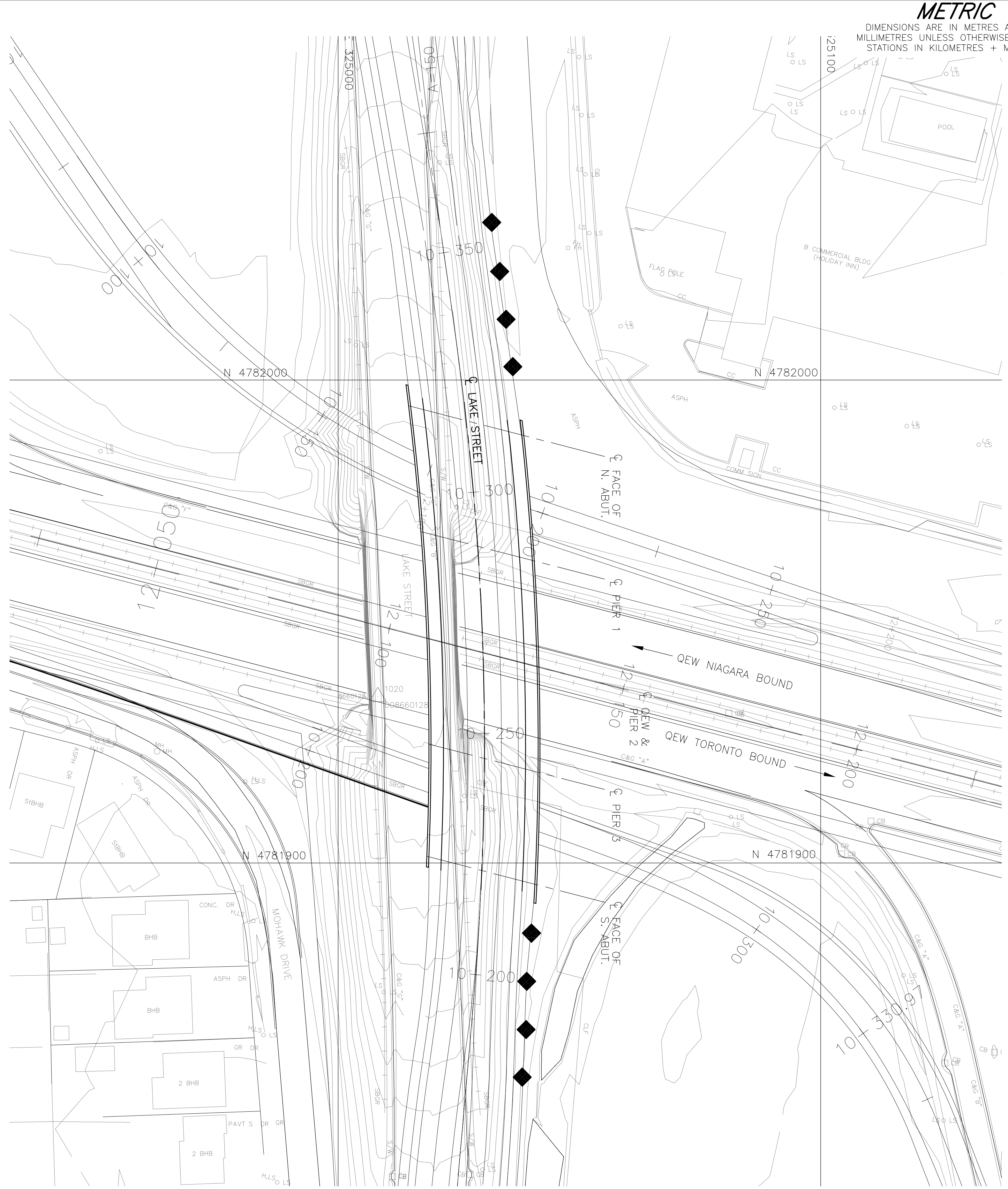
END OF SECTION

MINISTRY OF TRANSPORTATION, ONTARIO



SETTLEMENT PLATE (SP)

NOT TO SCALE



PLAN

SCALE

10 0 10 20 m

METRIC

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

CONT No.

WP No. 607-00-00

SHEET

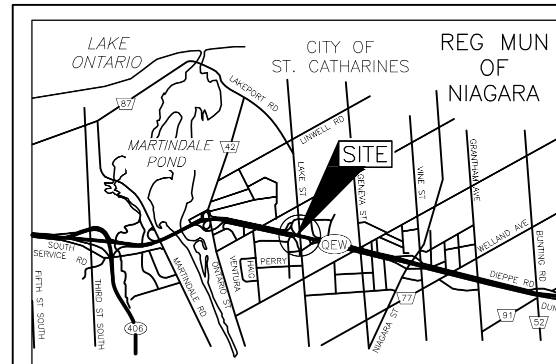
LAKE STREET EMBANKMENT

MONITORING INSTRUMENT LOCATION PLAN AND TYPICAL INSTRUMENT INSTALLATION DETAIL

Golder Associates

Golder Associates Ltd.

MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE

1 0 1 km

LEGEND

◆ Approximate Settlement Plate Locations

REFERENCE

Base plans provided in digital format by Morrison Hershfield Limited, drawing file nos. 18105-01.dwg received May 19, 2005, qew final.dwg, m402.dwg and hm_elec_design.dwg, received June 1, 2003.

NO.	DATE	BY	REVISION

Geocres No.

HWY. QEW	PROJECT NO. 04-1111-002		DIST.
SUBM'D. LCC	CHKD. LCC	DATE: SEPT 2006	SITE: 18-105
DRAWN: MSM	CHKD. LCC	APPD. MSD	DWG. 1

PLOT DATE: September 29, 2006
FILENAME: T:\Projects\2004\04-1111-002 (MH, StCatherines)\-XA- (LAKE STREET MONITORING)\0411110020XA001.dwg