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

Submitted to:

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PART A

**FOUNDATION INVESTIGATION REPORT
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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 Martindale Road underpass structure (MTO Structure Site No. 18-103).

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 QEW/Martindale Road underpass/interchange is located between the QEW/Highway 406 interchange and the bridge across the Twelve Mile Creek/Martindale Pond in St. Catharines, Ontario. The terrain is relatively flat with a gentle slope downwards to the southwest.

The existing Martindale Road grade is between approximately Elevation 94 m to 96 m in the immediate vicinity of the bridge. The QEW has been constructed in a cut in this area and is about 2.5 m to 3 m below the original ground surface, which is at about Elevation 92 m to 93 m. The Martindale Road embankments are about 2 m to 3 m in height relative to the original ground surface.

There are residential/agricultural properties on the north side of the QEW and industrial properties on the south side of the QEW in the immediate vicinity of the interchange.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the Martindale Road underpass site in November 2004, at which time eight boreholes (Boreholes 201 to 208) were advanced at the site using a track-mounted drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The borehole locations are shown on Drawing 1.

The boreholes were advanced using hollow stem augers, to depths ranging from 6.7 m to 27.6 m below the existing ground surface. Soil samples 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. The water level in the open boreholes was observed throughout the drilling operations, and a piezometer was installed in Borehole 203 and in Borehole 207 to permit monitoring of the groundwater level at the site. The piezometers consist of 50 mm diameter PVC pipe with a 1.5 m long slotted screen installed within a 3 m length sand filter pack. Upon completion, all boreholes were backfilled to ground surface using bentonite pellets.

The field work was supervised on a full-time basis by a member of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and testing. Index and classification tests consisting of water content determinations, Atterberg Limits testing and grain size distribution analyses were carried out on selected soil samples. Organic content tests were also carried out on selected samples.

The northings, eastings and elevations of the as-drilled borehole locations were measured in the field by a member of Golder's technical staff, relative to the locations staked by Morrison Hershfield. The borehole locations (including MTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

<i>Borehole Number</i>	<i>Borehole Locations</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
201	North Approach	4782038.3	322982.7	94.9
202	North Abutment	4782014.8	322076.9	95.4
203	North Abutment	4782020.1	322990.8	95.1
204	Centre Pier	4781982.5	322973.0	90.5
205	Centre Pier	4781995.3	323008.4	90.1
206	South Abutment	4781957.8	323000.6	95.2
207	South Abutment	4781962.5	323012.9	95.1
208	South Approach	4781939.3	323008.2	94.9

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 Subsurface Conditions at Martindale Road Underpass

Eight boreholes (Borehole 201 to 208) were advanced at the Martindale Road underpass at the locations shown on Drawing 1. Six boreholes were drilled in the vicinity of the existing abutments and approach embankments, and another two were drilled adjacent to the existing piers at the centre median on the QEW pavement level.

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 in situ and laboratory testing are summarized on the Record of Borehole sheets and Figures 1 to 8. 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 location.

In brief, the native subsoils underlying the fill materials at the site of Martindale Road underpass replacement consist of the following:

- A surficial clayey silt to silty sand deposit, the base of which was encountered at depths between 3.4 m and 5.5 m, with a thickness of between 0.4 m and 3.2 m, that has a firm to hard consistency/loose to very dense relative density.
- A clayey silt till deposit, found to be up to 18.5 m in thickness, which has a stiff to hard consistency.
- An interlayered silty sand, sandy silt to sand and gravel deposit, which was encountered at depths ranging from 11.6 m to 21.6 m, with a thickness between 1.3 m and 8.2 m and a loose to very dense relative density.
- A till/residual soil deposit, which was encountered at depths greater than 22 m, that has a hard consistency/very dense relative density.

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

4.2.1 Fill

Fill material was found in all of the boreholes at the site, ranging between 0.8 m and 3.1 m in thickness. In Boreholes 204 and 205, which were drilled at the QEW grade, the fill is comprised of the roadway granular and is about 0.8 m thick. Granular fill was also encountered in the remaining boreholes put down through the embankments; these fill materials range from 0.1 m to 0.8 m in thickness.

In Boreholes 201 to 203 and 206 to 208, the embankment fill underlying the granular fill consists mainly of clayey silt, with some sand, trace gravel and trace topsoil. The results of three grain size distribution tests on samples of this fill are shown on Figure 1.

The measured SPT “N” values of the clayey silt fill material were between 9 and 21 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency.

4.2.2 Surficial Clayey Silt to Silty Sand

In Boreholes 201 to 203 and Boreholes 206 to 208, the embankment fill is underlain by a surficial deposit that varies in composition from clayey silt, to sand and silt to silty sand. This deposit was encountered between Elevations 92.3 m and 92.9 m, and varied from 0.4 m to 3.2 m in thickness. This deposit is typically brownish-black in colour and some samples were noted to contain organics. The organic portion of the clayey silt to silty sand deposit was found in all of the six boreholes (Boreholes 201 to 203 and 206 to 208) located adjacent to the existing bridge abutments, and was 0.4 m to 1.4 m in thickness. Two samples were selected from the deposit for organic content testing. The results are shown on the Record of Borehole sheets and are summarised in the following table.

<i>Borehole Number</i>	<i>Sample Elevation (m)</i>	<i>Organic Content (%)</i>
202	94.0	5.3
208	92.2	2.2

The results of four grain size distribution tests carried out on samples of this deposit are shown on Figure 2. The results of three Atterberg limits tests conducted on cohesive portions of this surficial deposit are included on Figure 3. The Atterberg limits tests measured plastic limits of 15 to 19 per cent, liquid limits of 27 to 41 per cent, and corresponding plasticity indices of 12 to 22 per cent, which confirm that the cohesive portion of this deposit is a clayey silt of low plasticity. The measured water contents on samples of this deposit were between 15 and 20 per cent.

The measured SPT “N” values within the clayey silt to silty sand deposit range from 5 to 63 blows per 0.3 m of penetration, indicating a firm to hard consistency/loose to very dense relative density.

4.2.3 Clayey Silt Till

The granular fills in Boreholes 204 and 205 (in the centre median) and the surficial deposit in the remaining boreholes are underlain by an extensive till deposit. The base of the till deposit was encountered between Elevations 73.3 m and 74.1 m. The till consists typically of clayey silt with sand to trace sand, and trace gravel. Seams of sand and/or silt were noted within some of the recovered samples, and an approximately 2.1 m thick lens or interlayer of silt was encountered within the till in Borehole 205. The result of five grain size distribution tests carried out on selected clayey silt till samples are shown on Figure 4.

The measured SPT “N” values within the glacial till deposit range from 7 to 55 blows per 0.3 m of penetration, but typically range from about 20 to 55 blows indicating a very stiff to hard consistency. The till was found to become softer in the lower parts of the deposit at the locations

of Boreholes 202 to 207; the measured SPT “N” values within this softer portion ranged from 7 to 15 blows per 0.3 m of penetration, indicating a firm to stiff consistency.

Two field vane tests were conducted within the softer portion of the till deposit, and the results are as follows:

<i>Borehole Number</i>	<i>Sample Elevation (m)</i>	<i>In Situ (kPa)</i>	<i>Remoulded (kPa)</i>
202	78.0	90	70
204	75.3	109	60

The results of twenty-three Atterberg limits tests are shown on Figures 5A and 5B. The measured plastic limits are between 12 and 18 per cent and the liquid limits are between 18 and 34 per cent, with corresponding plasticity indices of 6 to 15 per cent. These results confirm that the soil is a clayey silt of low plasticity. The measured water contents on samples of the till were between approximately 11 and 29 per cent.

4.2.4 Lower Sands, Silts and Gravels

In the six boreholes where the till deposit was fully penetrated (Boreholes 202 to 207), a 1.5 m to 5.2 m thick interlayered deposit consisting of silty sand to sandy silt to sand and gravel was encountered below the till deposit and above the lower till/residual soil deposit. The surface of this interlayered deposit was encountered between Elevations 73.3 m and 74.1 m.

The results of two grain size distribution tests carried out on samples of this deposit are shown on Figure 6. The measured water contents on selected samples were between approximately 9 and 25 per cent.

The measured SPT “N” values range from 6 to greater than 100 blows per 0.3 m of penetration, indicating a loose to very dense relative density.

4.2.5 Till/Residual Soil

Boreholes 202 to 207 were terminated within a clayey to sandy silt deposit which has been classified as till/residual soil. The surface of this lower till was encountered between approximately Elevation 70.3 m and 72.2 m. This deposit is red in colour and consists mainly of clayey silt with some sand and trace gravel (limestone and shale fragments), which is considered indicative of weathered shale. At a few locations, the deposit consists of sandy silt till which is likely indicative of weathered sandstone or siltstone.

The results of two grain size distribution tests carried out on samples of the till are shown on Figure 7. Atterberg limits tests were also carried out on two samples and the results are shown on

Figure 8. The measured plastic limits are 12 and 15 per cent, the liquid limits are 18 and 22 per cent, and the corresponding plasticity indices are 6 and 7 per cent.

Based on the measured SPT “N” values which range from 51 to greater than 100 blows per 0.3 m of penetration, the till/residual soil deposits have a hard consistency/very dense relative density.

4.2.6 Groundwater Conditions

The water level observed in the open boreholes on completion of drilling operations was at depths ranging from 12.8 m to 25.9 m below ground surface, which corresponds to elevations ranging from 69.2 m to 77.3 m.

Two piezometers were installed, one each in Boreholes 203 and 207, which are in the vicinity of the existing abutments of the Martindale bridge structure. The following table summarizes the water level measurements for these piezometers:


<i>Borehole Number</i>	<i>Ground Surface Elevation</i>	<i>Piezometer Tip Elevation</i>	<i>Measured Groundwater Elevation</i>		
			<i>26 Nov 2004</i>	<i>13 May 2005</i>	<i>06 Dec 2005</i>
203	95.1 m	72.2 m	81.9 m	82.2 m	82.1 m
207	95.1 m	67.7 m	81.3 m	81.9 m	81.7 m

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year. Perched groundwater conditions should be anticipated within the surficial deposits above the fine-grained till deposit, especially during the wetter months of the year.

5.0 CLOSURE


This Foundation Investigation Report was prepared by Ms. Beng Lay Teh and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fintan Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

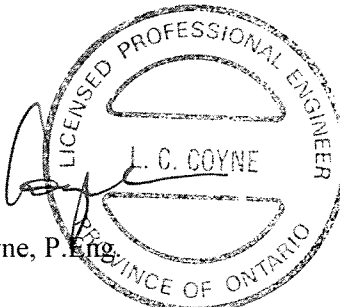
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PART B

**FOUNDATION DESIGN REPORT
MARTINDALE ROAD 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 design recommendations for the design of the proposed replacement of the Martindale Road bridge structure over QEW in the City of St Catharines. 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 to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out design of the proposed structure foundations. Where comments are made on construction they are provided in order to highlight those aspects which could affect the design and construction of the project, and for which special provisions or operational constraints may be required in the Contract Documents. 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 Martindale Road underpass will be a single-span structure, with a total structure length of approximately 56 m. During the development of the design, a two-span structure was also considered, and so geotechnical recommendations pertaining to both a single-span and a two-span structure are presented in this report.

The existing natural ground surface is about Elevation 92 m to 93 m in the general area and the QEW, which has been constructed in a cut at this site, is at about Elevation 90 m. The existing Martindale Road approach embankments are approximately 2 m to 3 m high relative to the original ground. The new structure will require widening of the existing approach embankments, primarily on the west side of the embankments. An embankment grade raise will also be required: for the single-span structure that has been adopted the new approach embankment grade raise will be about 2.2 m higher than the existing grade; if a two-span underpass was adopted, the new approach embankment grade would be about 1.7 m higher than the existing grade.

The subsoils encountered at the site consist of about 2.3 m to 3.1 m of embankment fill, overlying a firm to hard /loose to compact surficial clayey silt to silty sand deposit, which is underlain by an extensive stiff to hard clayey silt till deposit. This till is underlain by an interlayered sand/silt/clayey silt deposit, which is in turn underlain by a till/residual soil deposit.

The surficial clayey silt to silty sand deposit, which has variable organic content, is not suitable for the support of shallow foundations. Spread footings could be founded on the underlying clayey silt till; however, the consistency of the upper portion of the till deposit is variable and the geotechnical resistance will likely be too low to make the use of shallow foundations feasible.

It is therefore recommended that the foundation elements be supported on driven steel H-piles or drilled caissons founded at depth within the “100-blow” clayey silt till or till/residual soil deposit. From a foundations perspective, driven steel H-piles are considered to be the most practicable option, and these can be used in either a conventional or integral abutment configuration. Driven steel H-piles will also be more economical than caissons, since if caissons are adopted then the use of a relatively expensive permanent liner is recommended based on the soil conditions at the site. However, for the single-span structure that will be adopted at this site, the higher loads may require the use of heavier H-pile sections: in order to achieve higher axial resistances than those recommended for HP 310 x 110 piles, consideration could be given to the use of HP 310 x 132 piles for support of the new abutments; in order to achieve higher lateral resistances, consideration could be given to the use of HP 360 x 108 piles; and if both higher axial and lateral resistances are required, consideration could be given to the use of HP 360 x 132 piles. Based on discussions with piling contractors and the MTO Bridge Office, HP 310 x 132 piles are generally commercially available without any special order; HP 360 x 132 piles are in shorter supply, though a sufficient quantity of this section should be readily available for a project the size of the Martindale Road underpass structure.

Deep foundations supporting the abutments will be subjected to downdrag loads due to settlement of the founding soils under the widened and raised approach embankment loading, unless settlement mitigation measures (preloading/surcharging or the use of lightweight fill) are adopted.

A summary comparison of the advantages, disadvantages and relative costs associated with each of the feasible foundation options is presented in Table 1, following the text of this report.

6.3 Spread Footings

In order to limit the total settlement under shallow foundations to 25 mm or less, the footings would have to be extended below the firm to stiff surficial clayey silt deposit, to be founded within the clayey silt till deposit at or below Elevation 91 m for the abutments and at a minimum depth of 1.2 m below lowest surrounding grade at the centre pier (if a two-span structure is adopted). This will require excavations that are up to about 4.5 m in depth where they are extended through the existing approach embankments at this site.

For spread footings founded at or below the design elevation given above, the following factored geotechnical resistance at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS) can be used for design:

<i>Footing Width</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS</i>
3 m	450 kPa	275 kPa
4 m	500 kPa	225 kPa

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*, using the curves for cohesive soils.

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \phi'$, between cast-in-place concrete footings and the undisturbed, properly prepared clayey silt till may be taken as 0.45. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.4 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 site soils are suitable for construction of a conventional, integral or semi-integral abutment structure.

The surface of the “100-blow” till was encountered between Elevations 70.5 m and 72 m in the boreholes put down at this site, 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.

<i>Foundation Element</i>	<i>Relevant Boreholes</i>	<i>Estimated Elevation Of “100-Blow” Till</i>	<i>Estimated Pile Tip Elevation</i>
North Abutment	202, 203	71.5 m to 72 m	70 m
Centre Pier (if two-span structure adopted)	204, 205	70 m to 70.5 m	69 m
South Abutment	206, 207	70.5 m to 71 m	69 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 (including the heavier pile sections, if adopted) should be stiffened with MTO flange plates for protection during driving, in accordance with OPSS 903.07.05.04.

6.4.1 Axial Geotechnical Resistance

The following table provides recommended values for the factored axial resistance at ULS and geotechnical resistance at SLS for various H-pile sections. In order to achieve higher axial resistances than those recommended for the HP 310 x 110 piles, consideration could be given to the use of HP 310 x 132 piles for support of the new Martindale Road underpass abutments. In order to achieve higher lateral resistances, consideration could be given to the use of HP 360 x 108 piles. If both higher axial resistances and lateral resistances are required, consideration should be given to the use of HP 360 x 132 piles.

<i>Pile Section</i>	<i>Axial Resistance</i>	
	<i>Factored ULS</i>	<i>SLS</i>
HP 310 x 110	1,800 kN	1,600 kN
HP 310 x 132	2,150 kN	1,900 kN
HP 360 x 108	1,800 kN	1,600 kN
HP 360 x 132	2,150 kN	1,900 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 H-piles driven into the "100-blow" till, the following note should be shown on the contract drawings, assuming a resistance factor of 0.5 is applied to the use of the Hiley formula:

For HP 310 x 110 or 360 x 108 piles: *"Piles to be driven in accordance with SS 103-11, using an ultimate capacity of 3,600 kN per pile."*

For HP 310 x 132 or 360 x 132 piles: *"Piles to be driven in accordance with SS 103-11, using an ultimate capacity of 4,300 kN per pile."*

6.4.2 Downdrag Load (Negative Skin Friction)

Assuming the use of conventional earth or granular fill, the embankment loading will cause settlement of the surficial clayey silt and the less stiff portions of the clayey silt till deposit at this site (as discussed further in Section 6.7). As indicated, the magnitude of consolidation settlement as a result of the grade raise is expected to be between 10 mm and 20 mm under the existing embankment; however, there will be up to approximately 50 mm of settlement at the outer limits of the abutments, where the approach embankments will be both widened and raised. Negative skin friction or downdrag loads will, therefore, need to be taken into account in the design of the piles supporting the abutments.

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 relationship between the predicted settlement of the till deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the base of the till 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 ultra-lightweight or EPS fill as backfill behind the abutments; further discussion on the use of lightweight fill is provided in Section 6.7. 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.

6.4.3 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:

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

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:

$$k_h = \frac{6z s_u}{B} \quad \text{where}$$

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; 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.

<i>Soil Unit</i>	<i>Elevation</i>	<i>n_h</i>	<i>s_u</i>
Fill and surficial silts and sand	Above 92 m	10 MPa/m	–
Stiff to very stiff clayey silt till	92 m to 73 m	–	125 kPa
Loose to very dense lower sand and silts	73 m to 70 m	15 MPa/m	–
Hard/very dense (“100-blow”) lower till	Below 70 m	–	500 kPa

The following table provides recommended values for the maximum factored lateral resistance at ULS and maximum lateral resistance at SLS (for 10 mm of horizontal deflection at the pile cap level) for various pile sections, 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*.

<i>Pile Section</i>	<i>Lateral Resistance</i>	
	<i>Factored ULS</i>	<i>SLS</i>
HP 310 x 110	200 kN	110 kN
HP 310 x 132	200 kN	110 kN
HP 360 x 108	240 kN	140 kN
HP 360 x 132	240 kN	140 kN

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:

<i>Pile Spacing in Direction of Loading (d = Pile Diameter)</i>	<i>Subgrade Reaction Reduction Factor</i>
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.

6.4.4 Frost Protection

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

6.5 Caissons

Consideration could be given to the use of caissons founded within the “100-blow” till for support of the new Martindale Road 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:

<i>Foundation Element</i>	<i>Relevant Boreholes</i>	<i>Estimated Elevation Of 100-Blow Till</i>	<i>Estimated Caisson Base Elevation</i>
North Abutment	202, 203	71.5 m to 72 m	69 m
Centre Pier	204, 205	70 m to 70.5 m	68 m
South Abutment	206, 207	70.5 m to 71 m	68.5 m

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

Running or flowing of water-bearing cohesionless strata could occur during or after drilling of the caissons, so 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. 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 for the construction of the caissons in these soil 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.

6.5.1 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:

<i>Caisson Diameter</i>	<i>Axial Geotechnical Resistance</i>	
	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.

6.5.2 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:

<i>Caisson Diameter</i>	<i>Unfactored Downdrag Load</i>
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 6.4.2.

The downdrag loads provided above are relatively large and may render the use of caisson foundations impractical. The downdrag loads could be eliminated with the use of ultra-lightweight or EPS fill as backfill behind the abutments, and this measure has the advantage of reducing the settlement (and therefore future roadway maintenance) due to the embankment loading; further discussion on the use of lightweight fill is provided in Section 6.7. Alternatively, 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.

6.5.3 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 6.4.3.

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.

6.5.4 Frost Protection

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

6.6 Lateral Earth Pressures for Design

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

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II but with less than 5 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(I) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(I) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the 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

6.7 Approach Embankments

Construction of the new underpass will require widening of the existing approach embankments. An embankment grade raise will also be required: if a two-span underpass is adopted, the new approach embankment grade will be about 1.7 m higher than the existing; and if a single-span underpass is adopted, the new approach embankment grade will be about 2.2 m higher than the existing Martindale Road embankment grade.

6.7.1 Subgrade Preparation and Embankment Construction

Since the embankments at this site do not exceed a height of 8 m, construction of mid-height berms is not required.

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.

6.7.2 Approach Embankment Stability

Static slope stability analyses for the approach embankments have been 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.

<i>Soil Type</i>	<i>Unit Weight (kN/m³)</i>	<i>Effective Angle of Friction</i>	<i>Undrained Shear Strength</i>
Embankment fill (earth fill assumed)	20	30°	—
Surficial clayey silt to silty sand	20	30° - 32°	—
Clayey silt till (very stiff to hard)	21	35°	—
Clayey silt till (stiff)	20-21	30° - 35°	80 kPa
Lower sands and silts	20-21	30°	—
Till/Residual soil	21-22	35°	—

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the approximately 4 m to 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.

6.7.3 Approach Embankment Settlement

Settlement of the approach embankments at the site will occur due to compression of the new embankment fill itself, as well as consolidation of the surficial clayey silt and silty sand soils and the underlying clayey silt till under the new raised and widened embankment loading. In order to estimate the magnitude and rate of settlement, analyses were carried out using the commercially

available computer program Unisettle (Version 3.2), in conjunction with hand calculations. The settlement of the founding soils has been estimated using the elastic deformation moduli given in the table below, based on correlations with the relevant Atterberg limits and SPT “N” values as well as the oedometer test results from other structure sites on this project:

<i>Soil Type</i>	<i>Bulk Unit Weight (kN/m³)</i>	<i>Elastic Modulus (MPa)</i>
Existing embankment fill (earth fill)	20 – 22	10 - 15
Surficial clayey silt to silty sand	20	5 - 15
Clayey silt till – very stiff to hard	20	40 - 50
Clayey silt till – stiff	21	5 - 10
Lower sands and silts	20	30
Till/Residual soil	21	100

For the proposed Martindale Road embankment, there will be differential settlement across the width of the embankment as a result of placement of about 1.7 m to 2.2 m of new fill atop the existing embankments, as compared with the new full-height embankment construction in the widening areas. Based on the settlement analyses, and assuming the use of conventional earth or granular embankment fill, it is expected that about 10 mm to 20 mm of settlement will occur under the footprint of the *existing* embankments due to the grade raise (with the higher magnitude occurring under the west shoulder of the existing embankment, closest to the widening area); up to about 50 mm of settlement is predicted to occur under the new widened areas, where up to about 5 m of new fill will be placed. The majority of the settlement is expected to occur within three to six months after completion of the approach embankment widening and grade raise; following this period, it is expected that there will be less than about 5 mm to 10 mm of settlement remaining.

It is understood that preloading/surcharging is not an option at this site due to space and time restrictions. Since the settlements will be predominantly elastic and will occur relatively quickly, it is suggested that the construction of the approach slab and the final paving be delayed as long as possible if conventional earth/granular fill is used for the approach embankment widening and grade raise. It is understood that padding and final paving could be delayed by at least six months following completion of the approach embankment widening, which would allow for reduction of the post-paving settlements to less than about 5 mm to 10 mm, as noted above.

In order to reduce the magnitude of post-construction settlement under the widened/raised approach embankments, potentially eliminating the downdrag loads on deep foundation elements or permitting the use of shallow foundations at the site, ultra-lightweight or EPS fill may be considered for embankment construction, as outlined below. The additional cost of these materials is probably not warranted as a settlement mitigation measure in itself, since the settlement can be accommodated by the delay in paving; however, the use of this material to eliminate the downdrag loads could result in reduced costs for the piles.

- Use ultra-lightweight fill (unit weight of about 11.5 kN/m^3) for construction of the widened portion of the approach embankments. This would reduce the magnitude of settlement under the widened area from 50 mm to about 25 mm. This reduced settlement would be compatible with the approximately 10 mm to 20 mm of settlement that is predicted under the existing embankment where a 1.7 m to 2.2 m grade raise (with conventional earth fill) is required.
- Use approximately 3 m of EPS fill (i.e. similar to the height of the existing approach embankments) for the widened portions of the embankments, in conjunction with conventional earth and granular fill to make up the full embankment height, and use earth/granular fill for the grade raise in the existing embankment area. This would reduce the post-construction settlement under the widened area from 50 mm to between 10 mm and 20 mm, which is similar to that predicted under the footprint of the existing embankments following their grade raise.

If EPS fill is adopted, 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 in order to minimize abrupt differential settlement.

6.8 Construction Considerations

6.8.1 Open-Cut Excavations

Excavations for the pile caps will extend mainly through existing embankment fill and into the surficial sand/silt deposits at the abutments and will extend into the clayey silt till deposit at the center pier. 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 sand/silt soils are classified as Type 3 soil and the till is classified as Type 2 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.

6.8.2 Temporary Roadway Protection

Excavation support may be required at the site for temporary roadway protection (for example, to maintain traffic on Martindale Road during construction, or where there is limited space along the QEW adjacent to the pier excavation). 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.

6.8.3 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 seepage into the pile cap excavations can be adequately controlled by pumping from properly filtered sumps.


6.8.4 Obstructions During Pile Driving

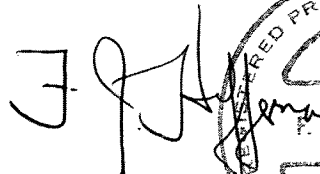
As discussed in Section 6.4, it is recommended that an NSSP be included in the Contract Documents to warn the contractor of the presence of boulders within the overburden soils, which are glacially derived, as such obstructions may affect the installation of steel H-piles. A draft NSSP is provided in Appendix A.


7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Beng Lay Teh and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer, with technical input from Ms. Anne Poschmann, P.Eng., a Principal with Golder. Mr. Fin Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality review of this report.

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TABLE 1
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
MARTINDALE ROAD UNDERPASS, G.W.P. 607-00-00

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>
Spread footings “perched” on granular pad in approach embankment fill	<ul style="list-style-type: none"> Not feasible due to construction staging schedule 	<ul style="list-style-type: none"> Minimizes excavation requirements Lower foundation costs 	<ul style="list-style-type: none"> Differential settlement between abutments and pier. Schedule would have to accommodate six months of preloading time before construction of the abutments in order to minimize potential for differential settlement. 	
Spread footings founded below surficial clayey silt, on upper clayey silt till	<ul style="list-style-type: none"> Not feasible if conventional earth/granular fill used for embankments; could be considered if EPS fill used behind abutments 	<ul style="list-style-type: none"> Lower foundation costs than deep foundation elements 	<ul style="list-style-type: none"> Significant depth of excavation required with temporary excavation support. Settlement of abutments governed by embankment loading. Differential settlement along length of abutment footings due to variable embankment loading. Cost of EPS fill. 	<ul style="list-style-type: none"> Higher subexcavation costs than for perched footings Cost of EPS fill must be considered if spread footings adopted for abutments
Steel H-pile foundations driven to found within 100-blow clayey silt till or till/residual soil	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> Avoids differential settlement between foundation elements Allows for integral abutment design 	<ul style="list-style-type: none"> Higher loads for single-span structure may require use of heavier H-pile sections; however, HP 310 x 132 are generally available without special order, and it is anticipated that sufficient HP 360 x 132 piles will be available for a project the size of the Martindale Road underpass structure. Must take downdrag loads into account at abutments. If a two-span structure adopted: Traffic constraints for working at the centre pier could make battered pile installation difficult. Potential difficulty with boulders within lower till deposit. 	<ul style="list-style-type: none"> Higher cost than spread footings
Caisson foundations bearing within 100-blow till/residual soil	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> Avoids differential settlement between foundation elements Potential for reduced “footprint” at centre pier, where there will be space restrictions during construction 	<ul style="list-style-type: none"> Must take downdrag loads into account at abutments. 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. 	<ul style="list-style-type: none"> Higher cost 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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Consistency

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

(b) Cohesive Soils

c_u, s_u

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength


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

- Notes:** 1 $\tau = c' + \sigma' \tan \phi'$
 2 shear strength = (compressive strength)/2
 * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 201				1 OF 1 METRIC										
W.P. <u>607-00-00</u>		LOCATION <u>N 4782038.3 ; E 322982.7</u>				ORIGINATED BY <u>PKS</u>										
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>				COMPILED BY <u>BLT</u>										
DATUM <u>Geodetic</u>		DATE <u>November 10, 2004</u>				CHECKED BY <u>ASP</u>										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
94.9	GROUND SURFACE															
0.0	Sand and gravel (FILL)															
0.2	Compact Brown		1	SS	20											
	Clayey silt, some sand, trace gravel, trace organics (FILL)															
	Stiff to very stiff Brown Moist		2	SS	9											
92.9			3	SS	10											
2.0	CLAYEY SILT, some sand, trace to some gravel, trace organics and rootlets															
	Firm to stiff Blackish grey Moist		4	SS	5											
91.5			5	SS	11											
3.4	CLAYEY SILT with sand to some sand, trace gravel (TILL)															
	Stiff to hard Brown Moist		6	SS	39											
			7	SS	29											
			8	SS	27											
88.2	END OF BOREHOLE															
6.6	Note: 1. Borehole dry on completion of drilling.															

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

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

PROJECT 04-1111-002			RECORD OF BOREHOLE No 202			2 OF 2 METRIC												
W.P. 607-00-00			LOCATION N 4782014.8 ; E 322976.9			ORIGINATED BY PKS												
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY BLT												
DATUM Geodetic			DATE November 18, 2004			CHECKED BY ASP												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m ³	GR SA SI CL	
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30						
--- CONTINUED FROM PREVIOUS PAGE ---																		
	CLAYEY SILT, trace to some sand, trace gravel, containing sand/silt seams (TILL) Stiff to hard Brown to grey Moist to wet		14	SS	31	▽	80											
								79										
								78										
								77										
								76										
			15	SS	20													
			16	SS	15													
			17	SS	9													
74.1																		
21.3	Silty SAND, containing clay seams/pockets Compact to very dense Grey Wet		18	SS	18													
			19	SS	100													
71.6																		
23.8	CLAYEY SILT, some sand, trace to some gravel, containing pockets of sandy silt (TILL/RESIDUAL SOIL) Very dense Grey to red Wet		20	SS	100/13													
			21	SS	100													
69.2																		
26.2	END OF BOREHOLE Note: 1. Water encountered at 18.9 m depth (Elevation 76.5 m) during drilling.																	

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

PROJECT 04-1111-002			RECORD OF BOREHOLE No 203			1 OF 2 METRIC		
W.P. 607-00-00			LOCATION N 4782020.1 ; E 322990.8			ORIGINATED BY PKS		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY BLT		
DATUM Geodetic			DATE November 18, 2004			CHECKED BY ASP		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)
95.1	GROUND SURFACE							
0.0	Crushed limestone (FILL) Compact Grey Moist		1	SS	23		95	
94.3								
0.8	Clayey silt, some sand, trace gravel, trace organics (FILL) Firm Blackish brown Moist		2	SS	7		94	
			3	SS	7			
92.8							93	
2.3	SAND and SILT, trace clay, trace gravel, trace organics Loose Mottled brown and grey Moist		4	SS	6			
92.0							92	
3.1	CLAYEY SILT, some sand, trace gravel Stiff Mottled brown and grey Moist		5	SS	14			
91.3							91	
3.8	Sandy SILT, trace clay Dense to very dense Brown Wet		6	SS	45			
			7	SS	63		90	
89.6								
5.5	CLAYEY SILT, some sand, trace gravel, containing sand/silt seams (TILL) Very stiff to hard Brownish grey to grey Moist		8	SS	24		89	
							88	
			9	SS	27		87	
							86	
			10	SS	27			
							85	
			11	SS	31		84	
							83	
			12	SS	29		82	
							81	
			13	SS	27			

Continued Next Page

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



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

PROJECT		04-1111-002		RECORD OF BOREHOLE No 203		2 OF 2 METRIC																	
W.P.		607-00-00		LOCATION		N 4782020.1 ; E 322990.8																	
DIST		Central HWY QEW		BOREHOLE TYPE		108 mm Diameter Solid Stem Augers																	
DATUM		Geodetic		DATE		November 18, 2004																	
				ORIGINATED BY		PKS																	
				COMPILED BY		BLT																	
				CHECKED BY		ASP																	
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)										
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20						40	60	80	100	20	40	60	80	100	10
--- CONTINUED FROM PREVIOUS PAGE ---																							
	CLAYEY SILT, some sand, trace gravel, containing sand/silt seams (TILL) Very stiff to hard Brownish grey to grey Moist		14	SS	33																		
			15	SS	23																		
			16	SS	20																		
76.0																							
19.1	CLAYEY SILT, some sand, trace gravel (TILL) Stiff Grey-brown to grey Moist																						
			17	SS	11																		
73.8																							
21.3	Sandy SILT, containing clayey seams Loose Grey Wet																						
			18	SS	6																		
72.2																							
22.9	SAND and SILT, trace clay, trace gravel (TILL/RESIDUAL SOIL) Very dense Grey, becoming red below Elevation 71.9 m Wet																						
			19	SS	100/25																		
			20	SS	100/15																		
69.2																							
25.9	CLAYEY SILT, some sand, trace gravel, containing shale fragments (TILL/RESIDUAL SOIL) Hard Red Wet																						
			21	SS	100/18																		
67.5																							
27.6	END OF BOREHOLE Notes: 1. Water level at 25.9 m depth (Elev. 69.2 m) on completion of drilling. 2. Water level measured at 13.2 m depth (Elevation 81.9 m) on Nov. 26, 2004, at 12.9 m depth (Elev. 82.2 m) on May 13, 2005 and at 13.0 m depth (Elev. 82.1 m) on Dec. 6, 2005.																						
			22	SS	100/13																		




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

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

PROJECT 04-1111-002			RECORD OF BOREHOLE No 204			2 OF 2 METRIC											
W.P. 607-00-00			LOCATION N 4781982.5 ; E 322973.0			ORIGINATED BY PKS											
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY BLT											
DATUM Geodetic			DATE December 8, 2004			CHECKED BY ASP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
73.7	CLAYEY SILT, trace to some sand, trace gravel (TILL) Firm to stiff Brown to grey Moist		14	SS	9		75										
74																	
73																	
72.2	Sandy SILT, trace clay, trace gravel Loose Grey Wet		15	SS	9		72										
72							71										
71							70										
70							69										
69	CLAYEY SILT, some sand, trace gravel, containing shale and limestone pieces (TILL/RESIDUAL SOIL) Hard Red Wet		16	SS	64		68										
68							67										
67																	
66.0			17	SS	77												
65																	
64			18	SS	100/13												
63																	
62																	
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MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 16/10/06

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 205		2 OF 2 METRIC													
W.P. <u>607-00-00</u>		LOCATION <u>N 4781995.3 ; E 323008.4</u>		ORIGINATED BY <u>PKS</u>													
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>		COMPILED BY <u>BLT</u>													
DATUM <u>Geodetic</u>		DATE <u>December 7, 2004</u>		CHECKED BY <u>ASP</u>													
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---																
73.3	CLAYEY SILT, some sand, trace gravel, containing sandy silt pockets/seams (TILL) Stiff Grey Wet		14	SS	9												
74																	
16.8	SAND and GRAVEL, some silt Compact to very dense Red/grey Wet		15	SS	19												
73																	
72																	
70.3			16	SS	101												
71																	
19.8	CLAYEY SILT, some sand, containing shale pieces (TILL/RESIDUAL SOIL) Hard Red Wet		17	SS	100/25												
70																	
68.6																	
21.5	END OF BOREHOLE Note: 1. Water encountered at 12.8 m depth (Elevation 77.3 m) during drilling.		18	SS	100/2												

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

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 206		2 OF 2 METRIC	
W.P. <u>607-00-00</u>		LOCATION <u>N 4781957.8 ; E 323000.6</u>		ORIGINATED BY <u>PKS</u>	
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>		COMPILED BY <u>BLT</u>	
DATUM <u>Geodetic</u>		DATE <u>November 16, 17, 2004</u>		CHECKED BY <u>ASP</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
						20	40	60	80	100				
	--- CONTINUED FROM PREVIOUS PAGE ---													
	CLAYEY SILT, trace to some sand, trace gravel, containing sandy silt seams (TILL) Stiff to hard Brownish grey to grey Moist		14	SS	37									
			15	SS	24									
			16	SS	12									
			17	SS	12									
73.6			18	SS	16									
21.6	SILT, some sand, trace clay, trace to some gravel Compact to very dense Grey Wet		19	SS	51									
71.7			20	SS	101									
23.5	CLAYEY SILT, some sand, trace gravel (TILL/RESIDUAL SOIL) Hard Red Wet		21	SS	100/13									
			22	SS	100/13									
67.6														
27.7	END OF BOREHOLE													
	Note: 1. Water level at 25.9 m depth (Elev. 69.3 m) on completion of drilling.													

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

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

PROJECT <u>04-1111-002</u>			RECORD OF BOREHOLE No 207			2 OF 3 METRIC																							
W.P. <u>607-00-00</u>			LOCATION <u>N 4781962.5 ; E 323012.9</u>			ORIGINATED BY <u>PKS</u>																							
DIST <u>Central</u> HWY <u>QEW</u>			BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>			COMPILED BY <u>BLT</u>																							
DATUM <u>Geodetic</u>			DATE <u>November 11, 12, 2004</u>			CHECKED BY <u>ASP</u>																							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	20 40 60 80 100			20 40 60 80 100			W _p W W _L			10 20 30			γ			GR SA SI CL							
--- CONTINUED FROM PREVIOUS PAGE ---																													
	CLAYEY SILT, some sand, trace to some gravel, containing sand/silt seams (TILL) Stiff to hard Greyish brown to grey Moist to wet		14	SS	29																								
			15	SS	30																								
			16	SS	12																								
			17	SS	10																								
73.8																													
21.2	Sandy SILT Compact Grey Wet		18	SS	18																								
72.2																													
22.9	Silty SAND, some gravel Compact to dense Grey/red Wet		19	SS	30																								
70.8																													
24.5	CLAYEY SILT, some sand, trace gravel Hard Grey Silty SAND, trace gravel (TILL/RESIDUAL SOIL) Very dense Red Wet		20	SS	100/18																								
69.2																													
25.9	CLAYEY SILT, some sand, trace gravel and shale pieces (TILL/RESIDUAL SOIL) Hard Red Moist		21	SS	100/13																								
67.7																													
27.6	Silty SAND, trace gravel, containing shale pieces (TILL/RESIDUAL SOIL) Very dense Red Wet END OF BOREHOLE		22	SS	100/15																								

Continued Next Page

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

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

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 207				3 OF 3 METRIC											
W.P. <u>607-00-00</u>		LOCATION <u>N 4781962.5 ; E 323012.9</u>				ORIGINATED BY <u>PKS</u>											
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>				COMPILED BY <u>BLT</u>											
DATUM <u>Geodetic</u>		DATE <u>November 11, 12, 2004</u>				CHECKED BY <u>ASP</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			W _L
	--- CONTINUED FROM PREVIOUS PAGE ---																
	Notes: 1. Water level at 25.9 m depth (Elev. 69.2 m) on completion of drilling. 2. Water level measured at 13.85 m depth (Elevation 81.3 m) on Nov. 26, 2004, at 13.2 m depth (Elev. 81.9 m) on May 13, 2005, and at 13.4 m depth (Elev. 87.1 m) on December 6, 2005.																

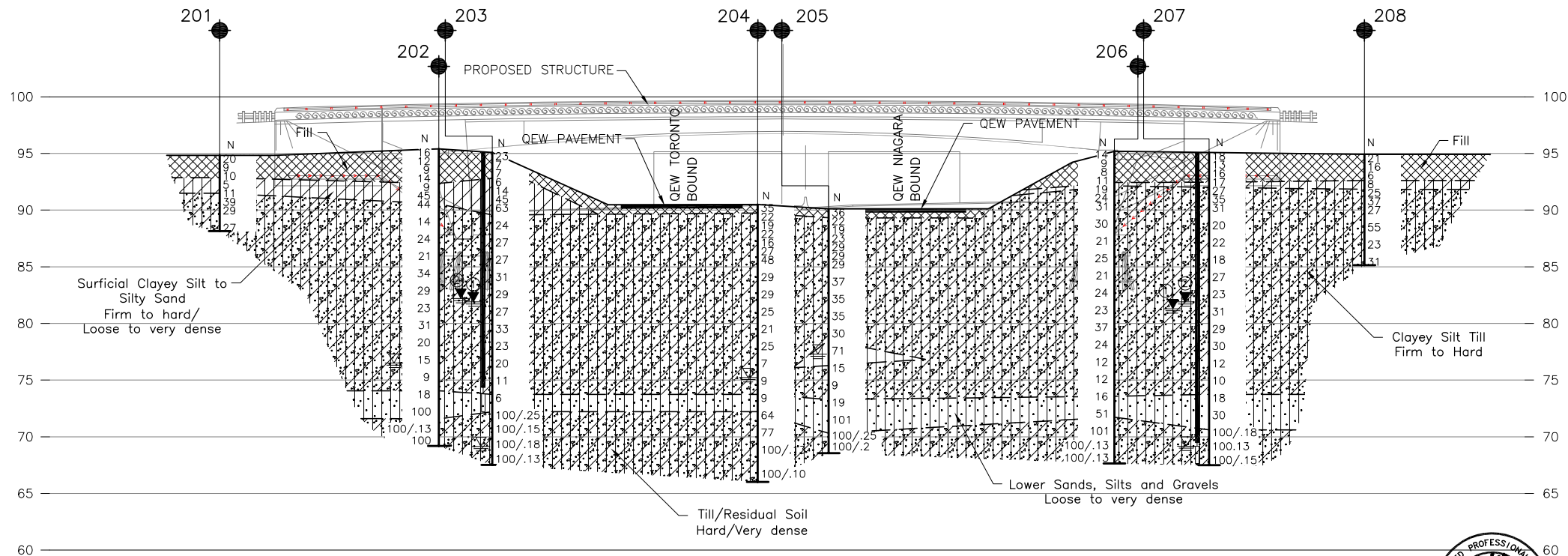
MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 16/10/06

PROJECT 04-1111-002			RECORD OF BOREHOLE No 208			1 OF 1 METRIC		
W.P. 607-00-00			LOCATION N 4781939.3 ; E 323008.2			ORIGINATED BY PKS		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY BLT		
DATUM Geodetic			DATE November 16, 2004			CHECKED BY ASP		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
94.9	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
8.0	Sand and gravel (FILL) Compact Moist		1	SS	21		94	
	CLAYEY SILT, some sand, trace gravel, trace organics (FILL) Firm to very stiff Brown Moist		2	SS	16		93	
			3	SS	6			
92.3			4	SS	8		92	41
2.6	SILTY CLAY, some sand, trace gravel, containing organics and rootlets Blackish brown		5	SS	25		91	
91.8	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Brownish grey to grey Moist		6	SS	37		90	
3.2			7	SS	27		89	
			8	SS	55		88	
			9	SS	23		87	
			10	SS	31		86	
85.1	END OF BOREHOLE							
9.9	Note: 1. Borehole dry on completion of drilling.							

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



PLAN



PROFILE ALONG MARTINDALE ROAD



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

CONT No.
WP No. 607-00-00

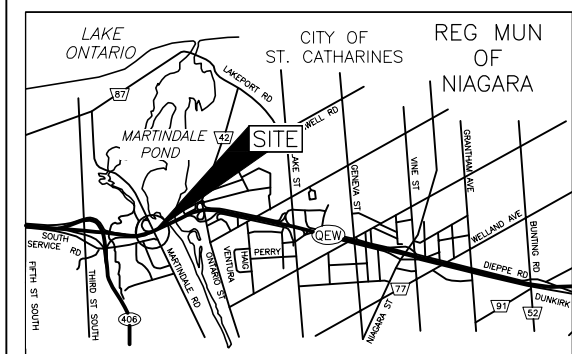
MARTINDALE ROAD UNDERPASS
QEW WIDENING, ST. CATHARINES
BOREHOLE LOCATIONS AND
SOIL STRATA



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
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 Nov 26, 2005
- WL in piezometer, measured on May 13, 2005
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
201	94.9	4782038.3	322982.7
202	95.4	4782014.8	322976.9
203	95.1	4782020.1	322990.8
204	90.5	4781982.5	322973.0
205	90.1	4781995.3	323008.4
206	95.2	4781957.8	323000.6
207	95.1	4781962.5	323012.9
208	94.5	4781939.3	323008.2

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 Contract Documents.

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

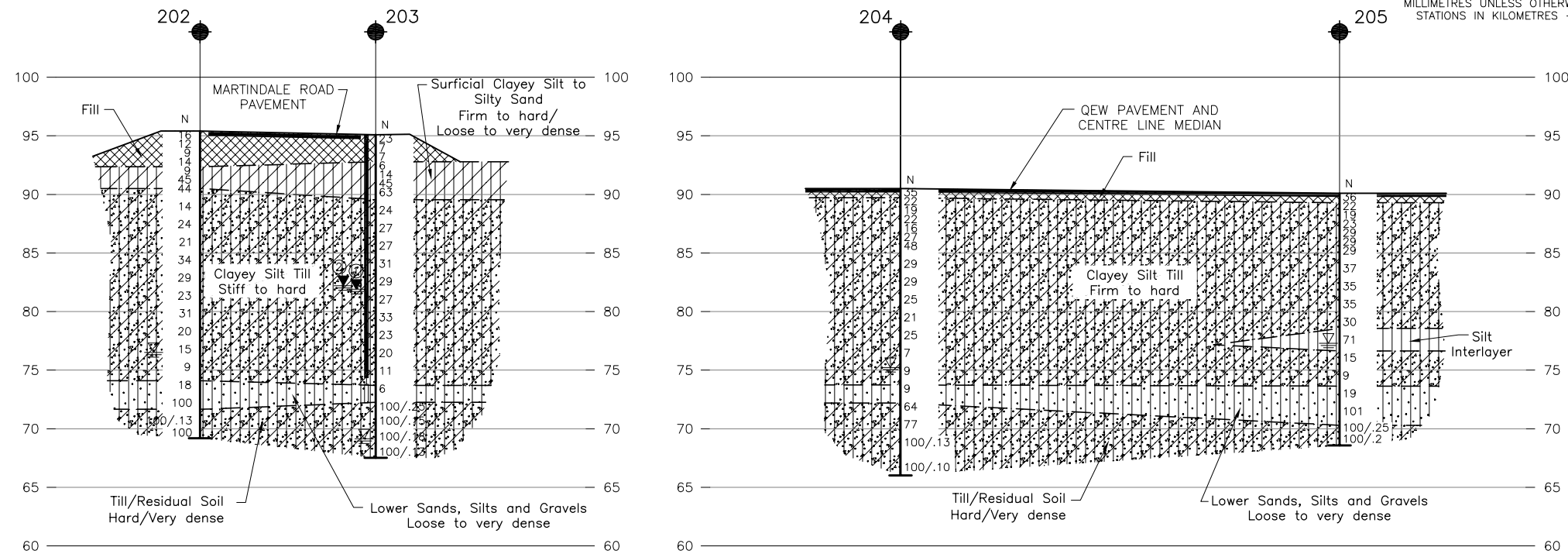
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE

Base plans provided in digital format by Morrison Hershfield Limited, drawing file nos. 18103-01.dwg, received October 16, 2006; qew final.dwg, m401.dwg and hm_elec_design.dwg, received June 1, 2003.



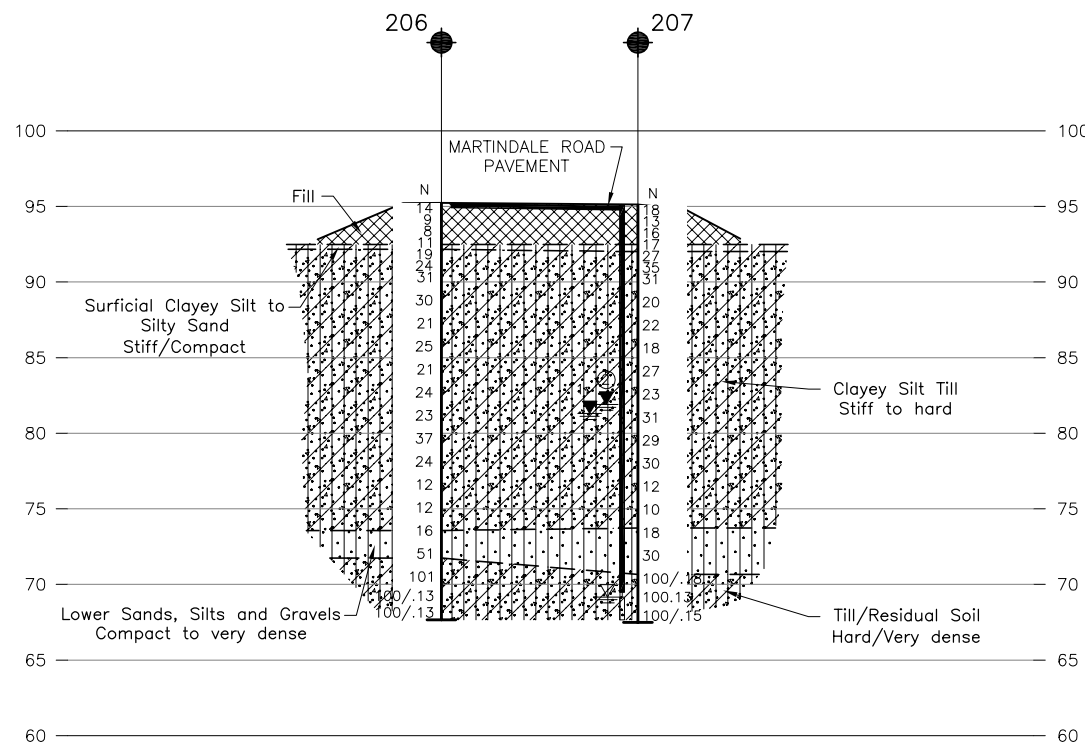
NO.	DATE	BY	REVISION
Geocres No.			
HWY: QEW	PROJECT NO. 04-1111-002		DIST.
SUBM'D. BLT	CHKD. BLT	DATE: OCT 2006	SITE:
DRAWN: MSM	CHKD. ASP	APPD. ASP	DWG. 1

B-B'
1

SECTION ALONG NORTH ABUTMENT

C-C'
1

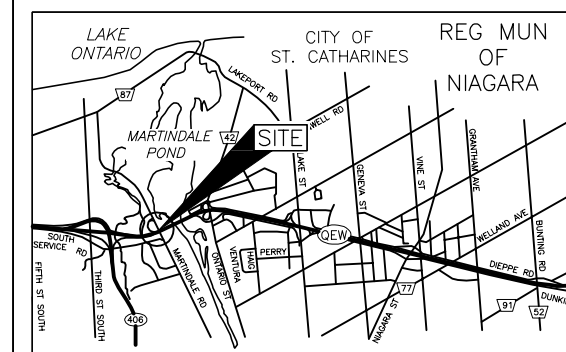
SECTION ALONG CENTRE PIER (IF ADOPTED)

D-D'
1

SECTION ALONG SOUTH ABUTMENT

SCALE
5 0 5 10 mCONT No.
WP No. 607-00-00MARTINDALE ROAD UNDERPASS
QEW WIDENING, ST. CATHARINES
SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

KEY PLAN

SCALE
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)
- ① WL in piezometer, measured on Nov 26, 2005
- ② WL in piezometer, measured on May 13, 2005
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
201	94.9	4782038.3	322982.7
202	95.4	4782014.8	322976.9
203	95.1	4782020.1	322990.8
204	90.5	4781982.5	322973.0
205	90.1	4781995.3	323008.4
206	95.2	4781957.8	323000.6
207	95.1	4781962.5	323012.9
208	94.5	4781939.3	323008.2

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 Contract 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. 18103-01.dwg, received October 16, 2006; qew final.dwg, m401.dwg and hm_elec_design.dwg, received June 1, 2003.



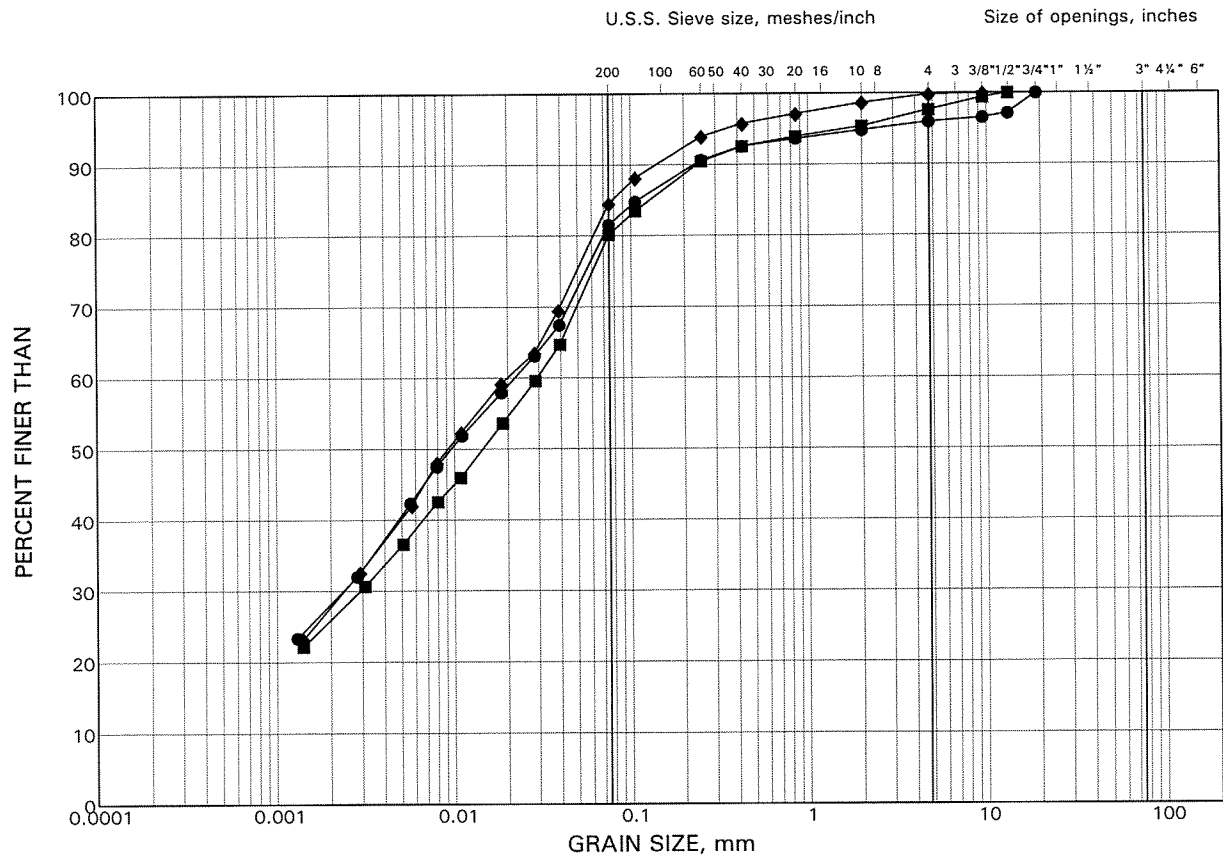
NO.	DATE	BY	REVISION

Geocres No.		PROJECT NO. 04-1111-002		DIST.
SUBM'D. BLT	CHKD. BLT	DATE: OCT 2006	SITE:	
DRAWN: MSM	CHKD. ASP	APPD. ASP	DWG. 2	

GRAIN SIZE DISTRIBUTION TEST RESULTS

Fill

FIGURE 1



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

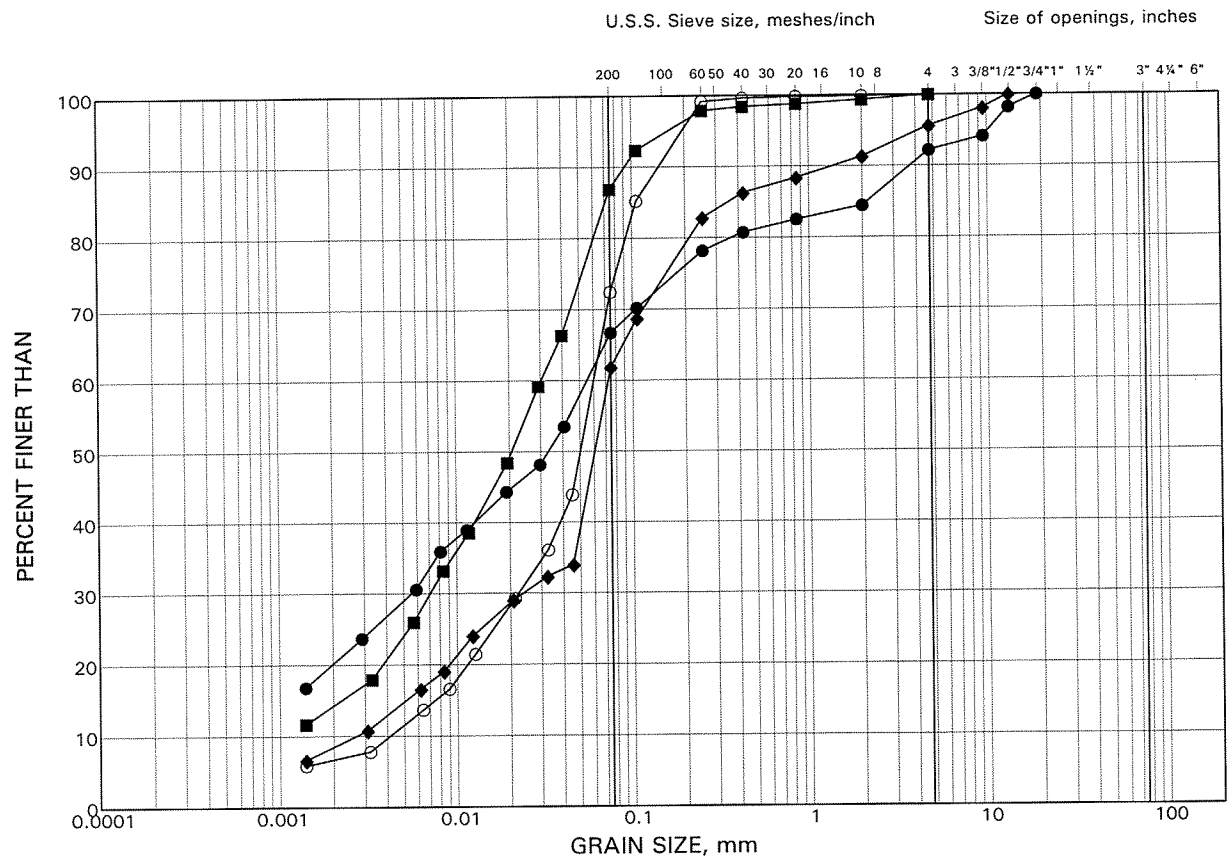
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	202	2	94.3
■	207	4	92.5
◆	208	3	93.1

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt to Silty Sand

FIGURE 2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	201	5	91.6
■	202	6	91.3
◆	203	4	92.5
○	203	7	90.2

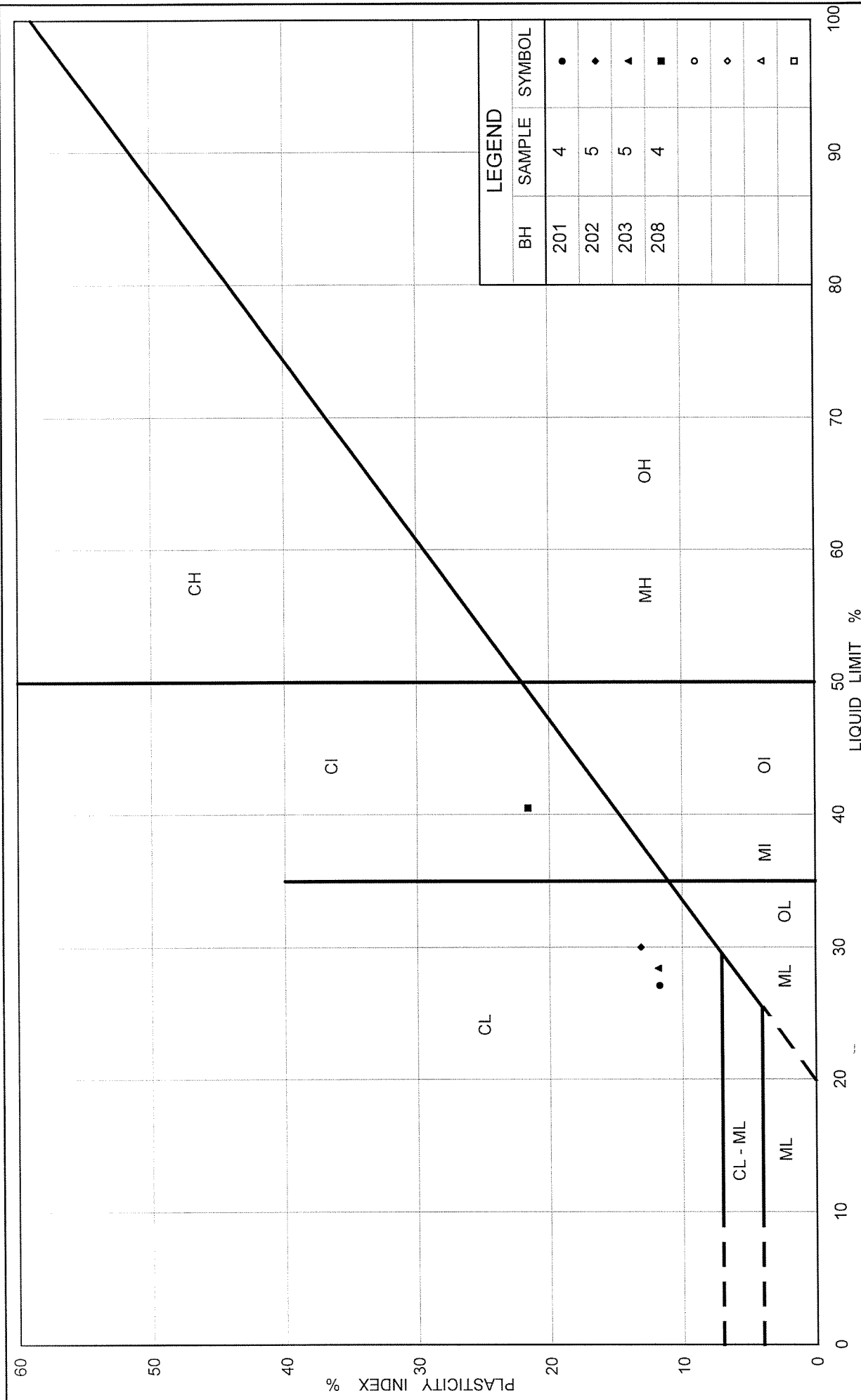


FIG No. 3

PLASTICITY CHART Clayey Silt to Silty Sand

Ministry of Transportation



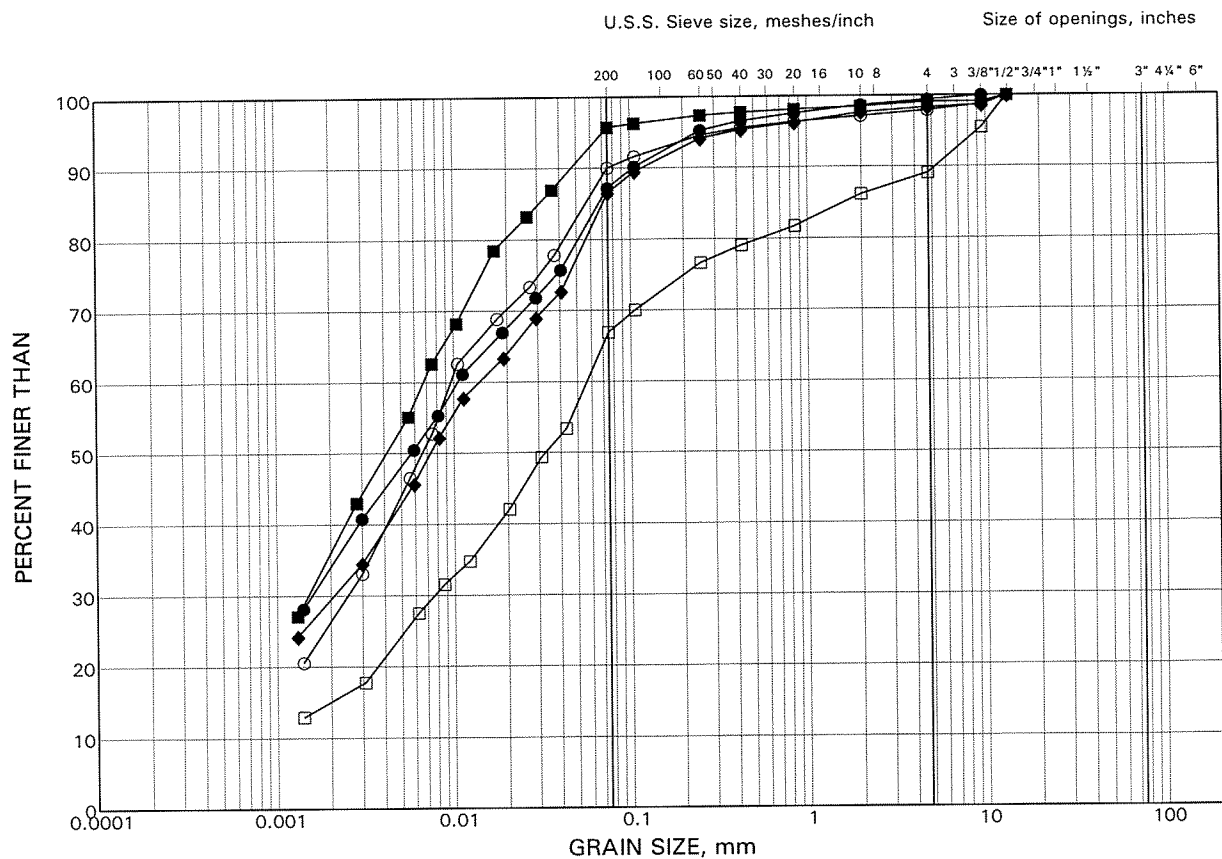
Ontario

Project No. 04-1111-002-2

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till

FIGURE 4



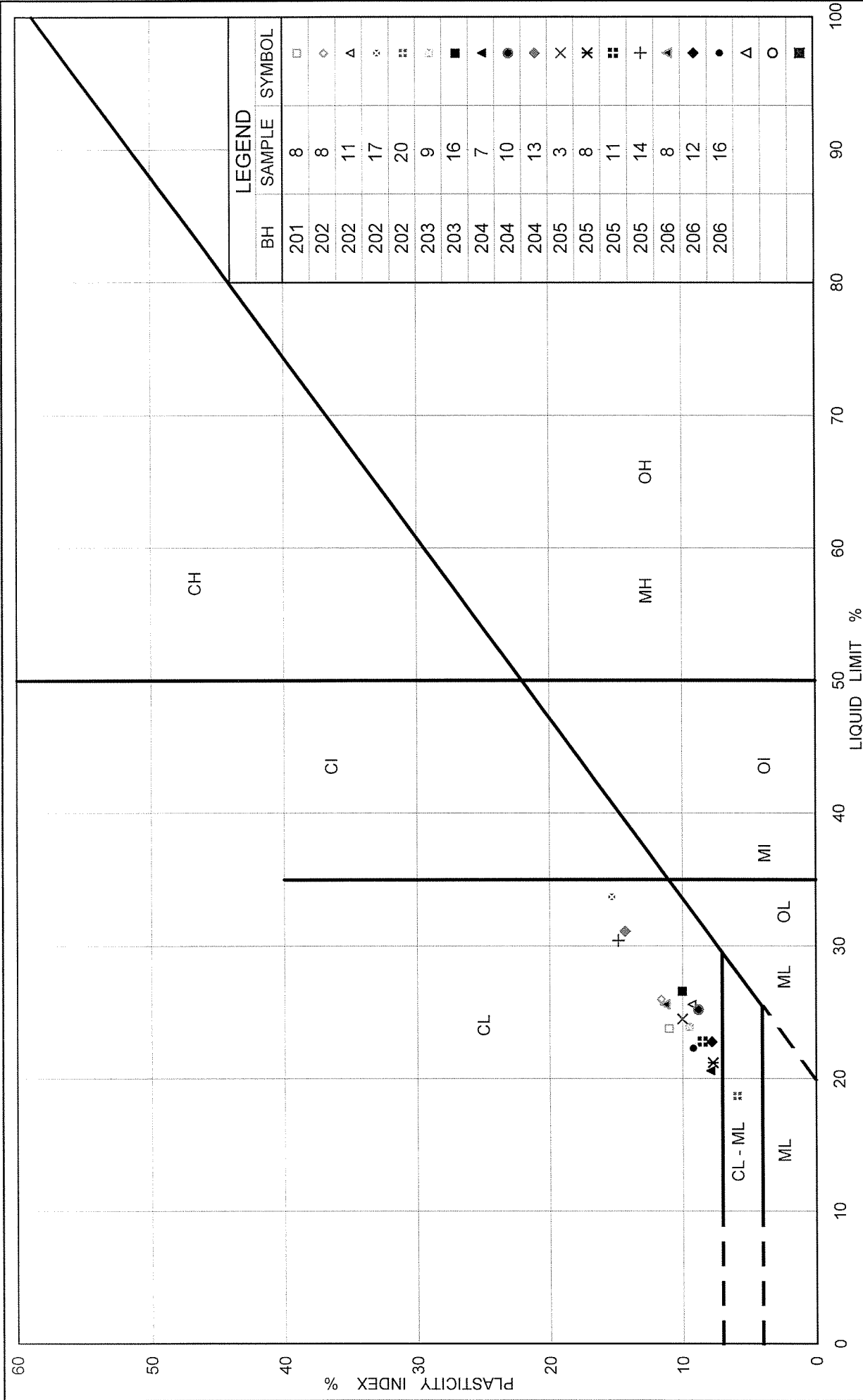


FIG No. 5A

PLASTICITY CHART
Clayey Silt Till

Ministry of Transportation



Ontario

Project No. 04-1111-002-2

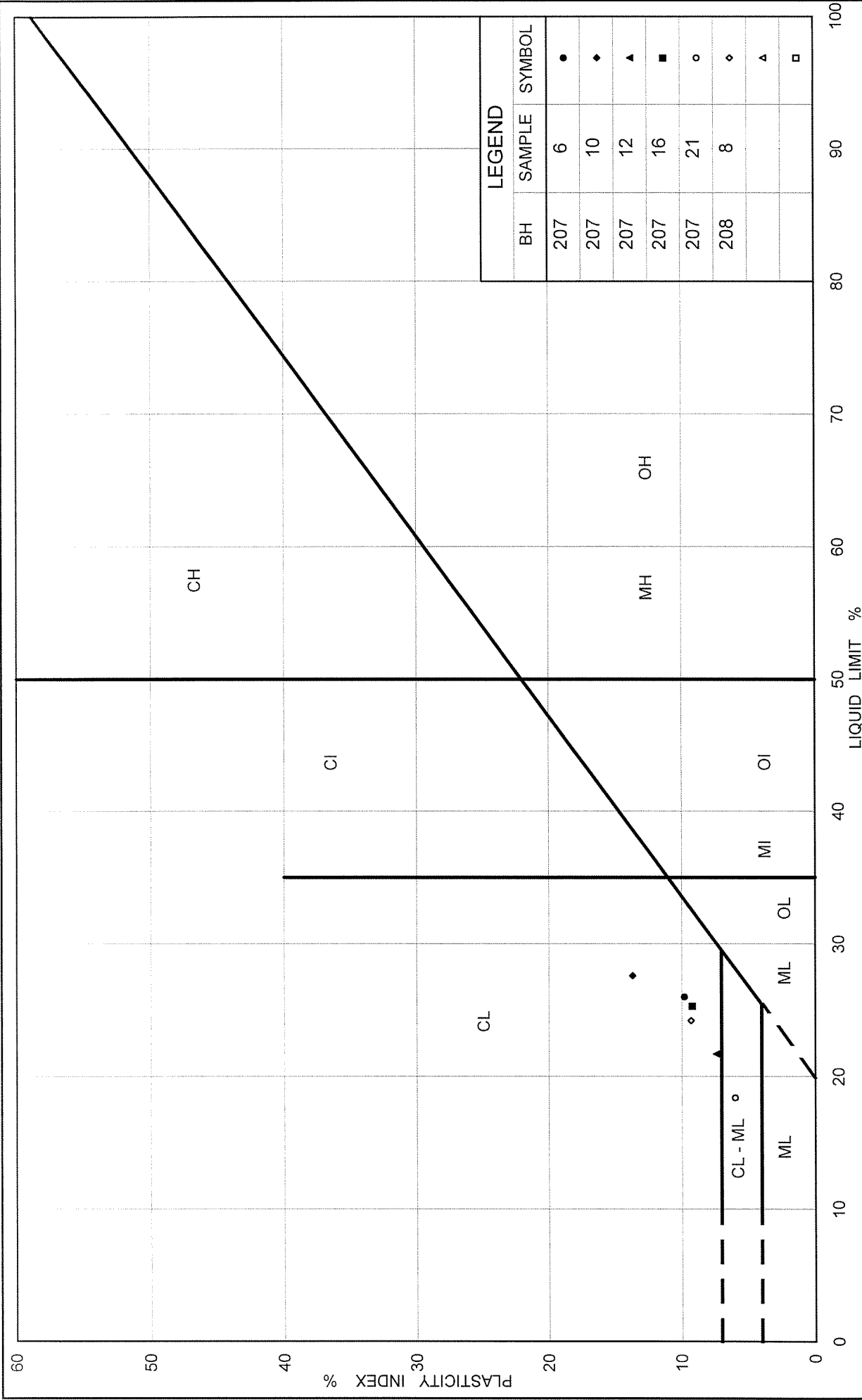


FIG No. 5B

PLASTICITY CHART
Clayey Silt Till

Ministry of Transportation



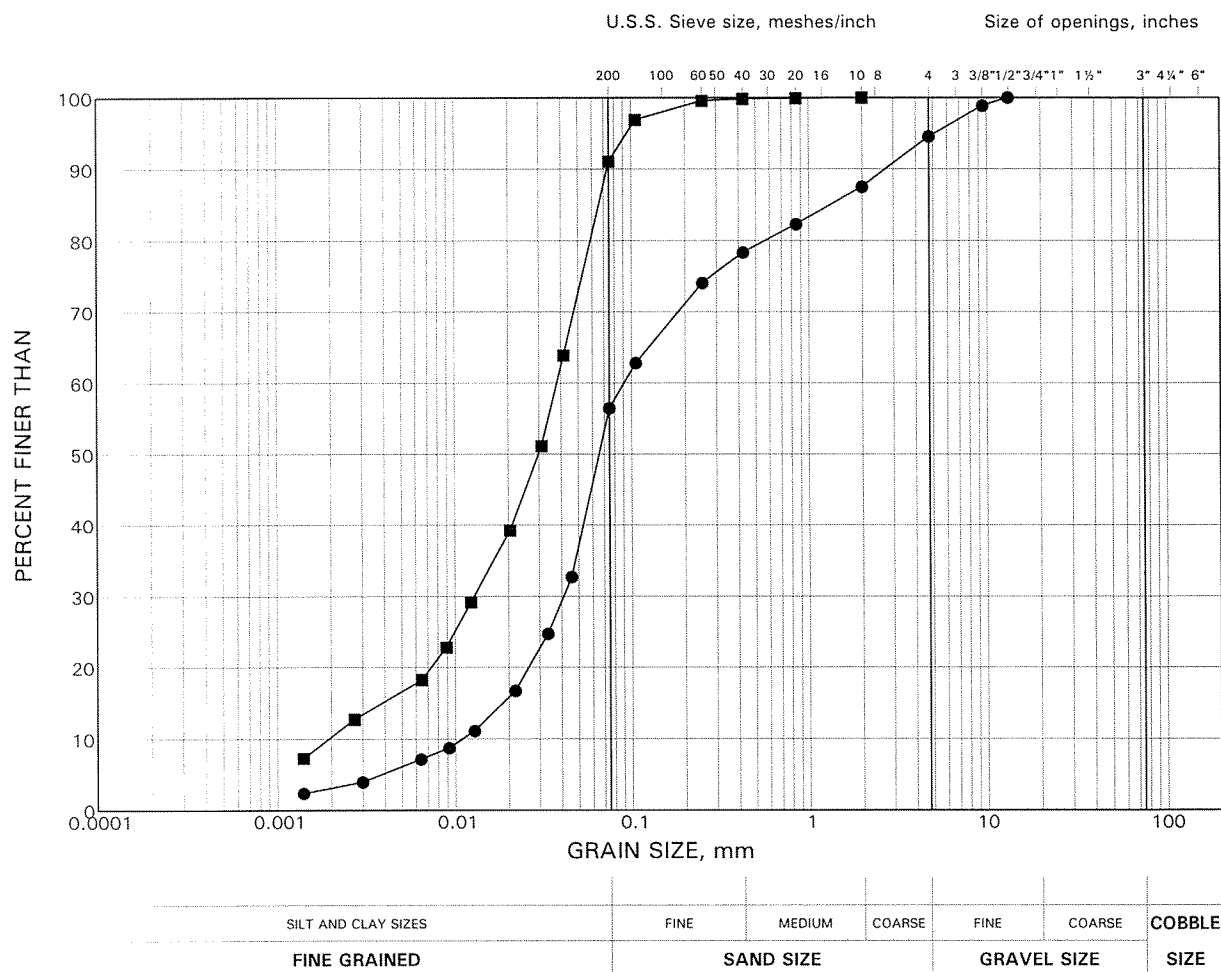
Ontario

Project No. 04-1111-002-2

GRAIN SIZE DISTRIBUTION TEST RESULTS

Lower Sands and Silts

FIGURE 6



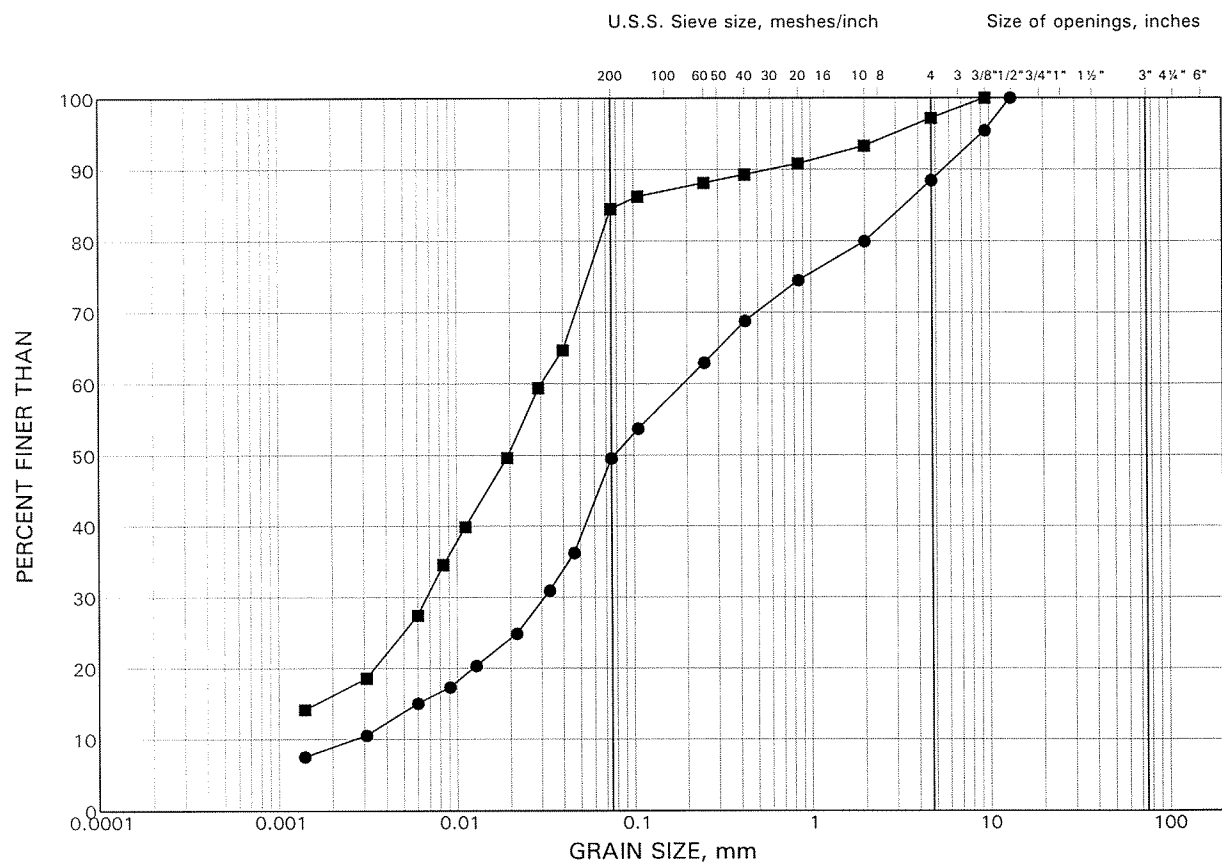
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	203	20	70.5
■	205	12	77.6

GRAIN SIZE DISTRIBUTION TEST RESULTS

Till / Residual Soil

FIGURE 7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	204	18	68.9
■	206	20	70.7

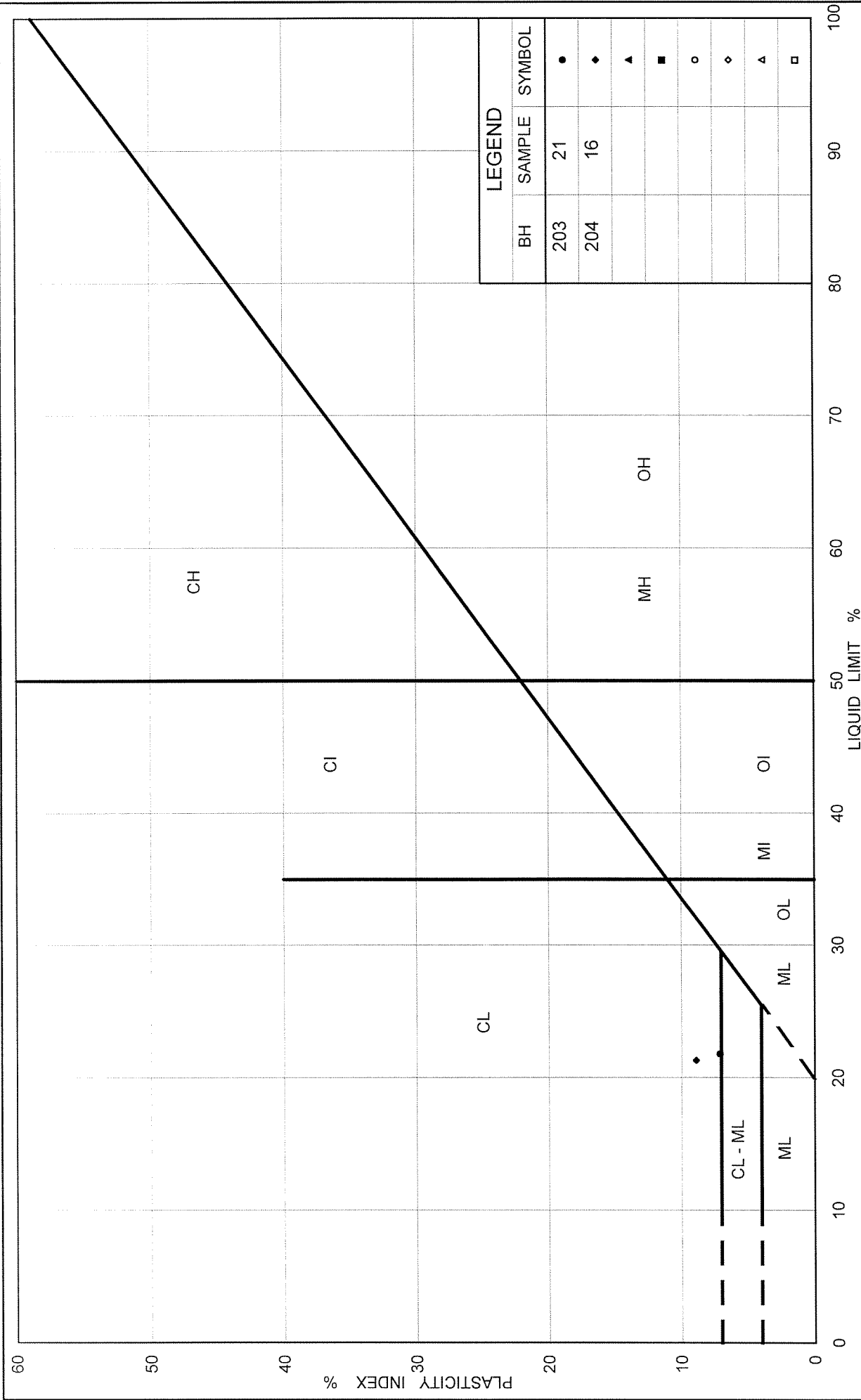


FIG No. 8

Ministry of Transportation

PLASTICITY CHART

Till / Residual Soil

Project No. 04-1111-002-2

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