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**FOUNDATION INVESTIGATION
AND DESIGN REPORT
WELLAND AVENUE OVERPASS
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 Welland Avenue overpass structure (MTO Structure Site No. 18-160).

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 existing Welland Avenue overpass is a single-span, reinforced concrete rigid frame structure; a pedestrian tunnel is present immediately east of the east abutment. The site is located approximately 650 m east of the QEW – Niagara Street interchange, in the City of St. Catharines in the Region of Niagara. The area surrounding the existing overpass is occupied by commercial and residential property developments.

The overall surface topography in the City of St. Catharines is relatively flat-lying, with a gentle slope downward to the north toward Lake Ontario. The natural ground surface in the immediate area of the structure site is at about Elevation 100 m to 101 m, and the existing QEW grade has a maximum elevation of approximately 108 m as it crosses Welland Avenue, which is at an elevation of about 101.5 m. The existing QEW approach embankments are sloped at approximately 2.5 horizontal to 1 vertical (2.5H:1V) above the existing retaining walls along the north and south sides of the QEW; these existing retaining walls vary from about 2 m to 4 m in height in the vicinity of the structure.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the proposed widening of the Welland Avenue structure in December 2004, at which time seven boreholes (Boreholes 501 to 507) 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 solid 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, 1.5 m and 3.0 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 standpipe piezometer was installed in each of Boreholes 503 and 507 to monitor the groundwater level at the site. The standpipe piezometers are 50 mm in diameter and consist of a 1.5 m long slotted screen installed within a filter sand pack, then backfilled to ground surface using bentonite pellets, as shown on the 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 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, and an oedometer (consolidation) test was carried out on one selected sample of firm cohesive soil.

The borehole locations were measured by Golder relative to site features, and the ground surface elevations at the borehole locations were determined from the digital terrain model (DTM) for this project. 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>Ground Surface Elevation (m)</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>
501	101.2	4,781,339.1	327,298.4
502	101.2	4,781,342.0	327,287.0
503	101.4	4,781,350.0	327,255.0
504	101.5	4,781,321.0	327,215.0
505	101.4	4,781,313.0	327,227.0
506	101.4	4,781,316.5	327,181.5
507	101.5	4,781,306.1	327,232.2

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, seven boreholes (501 to 507) were advanced. The borehole locations and ground surface elevations are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets and Figures

¹ 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.

1 to 6. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes 501 to 507 were advanced from the local road (Welland Avenue, Dieppe Road or Dunkirk Road) grade, where the existing ground surface is between Elevation 101.2 m and 101.5 m. The boreholes encountered topsoil or asphalt overlying up to 1.3 m of fill, underlain by an extensive cohesive till deposit – the upper 4 m to 5 m of the till consists of a stiff to hard silty clay “crust”, which overlies an 11 m to 14 m thick, firm to stiff silty clay till, which is in turn underlain by a 5 m to 8.5 m thick, stiff to hard clayey silt till. Underlying the silty clay to clayey silt till deposit is a dense to very dense cohesionless deposit that varies in composition from gravelly sand to silt.

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

4.2.1 Topsoil, Asphalt and Fill Materials

Approximately 100 mm of topsoil was encountered immediately below the ground surface in Boreholes 501, 502 and 503. About 200 mm of asphalt was encountered in Boreholes 504, 505 and 506.

Between 0.7 m and 1.3 m of fill materials were encountered underlying the topsoil in Boreholes 502 and 503, below the asphalt in Boreholes 504 to 506, and immediately below the ground surface in Borehole 507. The fill typically consists of sand and gravel or sand containing some gravel; however, in Borehole 503, about 0.8 m of clayey silt fill is present above the sandy fill, and in Borehole 507, the fill consists of sandy silt containing trace gravel.

The measured Standard Penetration Test (SPT) “N” values within the sand and gravel to sandy silt fill range from 7 to 28 blows per 0.3 m of penetration, indicating that the cohesionless fill has a loose to compact relative density. A single SPT “N” value of 15 blows per 0.3 m of penetration was measured in the cohesive fill, suggesting that this material has a very stiff consistency.

4.2.2 Silty Clay Till

A till deposit was encountered beneath the topsoil and fill in all of the boreholes, extending to about Elevation 82 m to 84 m (to depths of about 17.5 m to 19.5 m below the Welland Avenue grade). This deposit is about 15.9 m to 18.4 m in thickness as encountered in Boreholes 502 to 505.

This till deposit is comprised of silty clay containing trace to some sand and trace gravel; the results of grain size distribution tests conducted on four samples of this till are shown on Figure 1. Atterberg limit testing was carried out on sixteen samples of the till and measured plastic limits of 17 to 23 per cent, liquid limits of 35 to 49 per cent, and plasticity indices of 16 to 27 per cent. These results, which are plotted on a plasticity chart on Figure 2, confirm that this deposit is a silty clay of intermediate plasticity.

The silty clay till deposit becomes less stiff with depth, as follows:

- The upper approximately 4 m of the deposit, above about Elevation 96 m, is generally stiff to very stiff, based on measured SPT “N” values that generally range from 12 to 30 blows per 0.3 m of penetration. A few higher SPT “N” values (up to 35 blows per 0.3 m of penetration) were measured within this portion of the deposit.
- Between approximately Elevation 96 m and 90 m, the deposit is generally stiff, the measured SPT “N” values vary from 7 to 11 blows per 0.3 m of penetration, and field vane testing measured undrained shear strengths of approximately 85 kPa to greater than 100 kPa. An oedometer test was conducted on a sample of this portion of the till from Borehole 503, and measured a preconsolidation pressure of approximately 250 kPa. The oedometer test results are shown on Figures 3A to 3D.

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
503 / 10	9.4 m / 92.0 m	19.8	185	255	70	0.32	0.04	0.75	1.4

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

- Between approximately Elevation 90 m and the base of the deposit (which was encountered between about Elevation 82 m and 84 m), the deposit is generally firm to stiff, based on measured undrained shear strengths of about 45 kPa to 90 kPa. The measured SPT “N” values in this portion of the till deposit vary from 6 to 12 blows per 0.3 m of penetration.

4.2.3 Clayey Silt Till

The silty clay till grades to a clayey silt till at about Elevation 82 m to 84 m (about 17.5 m to 19.5 m below the Welland Avenue grade). The clayey silt till is about 5.2 m to 8.5 m thick, as encountered in Boreholes 502 to 505.

The clayey silt till contains some sand and trace gravel; the result of a grain size distribution test on one selected sample of this deposit is shown on Figure 4. Atterberg limit testing was

conducted on five samples of this deposit, and measured plastic limits of 14 to 15 per cent, liquid limits of 21 to 28 per cent, and corresponding plasticity indices of 6 to 13 per cent. The results, which are plotted on a plasticity chart on Figure 5, confirm that this till is comprised of a clayey silt of low plasticity.

The measured SPT “N” values within the till range from 12 to 31 blows per 0.3 m of penetration, indicative of a variable, stiff to hard consistency; however, most of the SPT “N” values are between 15 and 30 blows per 0.3 m of penetration, indicating that this till is typically very stiff.

4.2.4 Lower Gravelly Sand to Silt

Below the clayey silt till deposit lies a cohesionless soil deposit, with its surface encountered in Boreholes 502 to 505 between Elevations 75.3 m and 77.0 m (at about 24.4 m to 26.2 m depth below the Welland Avenue grade). All four boreholes were terminated within this deposit.

The deposit varies in composition from a silty sand containing trace gravel and shale fragments, to a gravelly sand containing some silt, to a silt containing trace to some sand, trace gravel and clay; clayey silt seams were noted within a sandy silt portion of this deposit, as encountered in Borehole 503. The results of grain size distribution tests carried out on two samples of this lower gravelly sand to silt deposit are shown on Figure 6.

The SPT “N” values measured within the lower gravelly sand to silt deposit range from 33 to greater than 100 blows per 0.3 m of penetration, indicative of a dense to very dense relative density. Typically, the lower SPT “N” values (33 to 82 blows per 0.3 m of penetration) were encountered in the upper 1 m to 1.5 m of the deposit; the surface of the “100-blow” material was encountered in the boreholes between approximately Elevation 75.5 m and 74 m.

4.3 Groundwater Conditions

The lower gravelly sand to silt deposit is water-bearing, and forms the main overburden aquifer at the site. During drilling, the water level encountered in the boreholes varied from about 24.4 m to 25.9 m depth below ground surface; this is typically near the surface of the gravelly sand to silt deposit.

Two standpipe piezometers were installed to monitor the groundwater level at the site: the piezometer in Borehole 503 was sealed into the lower gravelly sand to silt deposit, and the piezometer in Borehole 507 was sealed into the upper portion of the silty clay till. The following table summarizes the piezometric water level measurements:

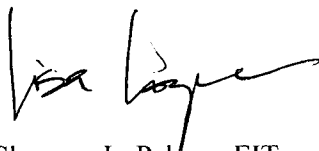
<i>Borehole No.</i>	<i>December 20, 2004</i>		<i>May 13, 2005</i>		<i>December 6, 2005</i>	
	<i>Depth</i>	<i>Elevation</i>	<i>Depth</i>	<i>Elevation</i>	<i>Depth</i>	<i>Elevation</i>
503	13.7 m	87.7 m	11.6 m	90.2 m	11.6 m	90.2 m
507	6.0 m	95.5 m	1.6 m	99.9 m	1.9 m	99.6 m

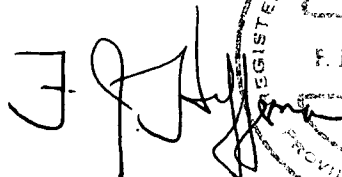
Based on the measurements shown above, there is a downward hydrogeological gradient at the site. It is noted that the measurements in December 2004 likely do not represent stabilized groundwater levels, since these were made less than one week following the piezometer installation.

5.0 CLOSURE

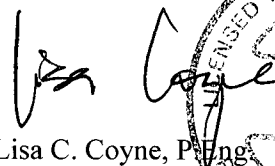
This Foundation Investigation Report was prepared by Shannon Palmer, EIT, and reviewed by Lisa Coyne, P.Eng., an Associate and Senior Engineer with Golder, with technical input from Murty Devata, P.Eng., a Specialist Foundations Consultant to Golder. Fin Heffernan, P.Eng., a Designated MTO Contact for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

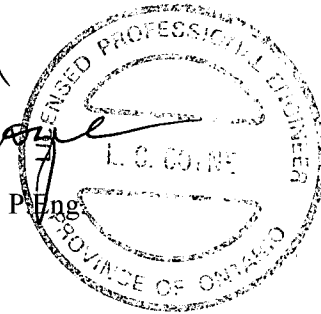
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PART B

**FOUNDATION DESIGN REPORT
WELLAND AVENUE OVERPASS
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 geotechnical recommendations for the design of the foundations for the proposed Welland Avenue overpass replacement structure. 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 replacement 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 existing single-span Welland Avenue overpass is to be replaced with a new, wider and longer single-span structure, and the approach embankments will be widened by up to about 5 m on each side of the existing QEW embankment. New retaining walls will be constructed along the north and south sides of the QEW to accommodate the approach embankment widening within the available embankment footprint. The widened portions of the QEW will be maintained at approximately Elevation 108 m within the limits of the structure and its immediate approach embankments, generally requiring placement of approximately 1.5 m to 2 m of new fill atop the existing embankment side slopes and behind the new retaining walls, based on the proposed embankment cross-sections.

The main concern for the design and construction of the Welland Avenue overpass replacement structure is consolidation settlement within the thick compressible silty clay till deposit that underlies this site, and the impacts that this settlement will have on the performance of the replacement structure and the approach embankments.

Shallow foundations are not feasible for support of the replacement structure, since the geotechnical resistance at Serviceability Limit States (SLS) for the abutment footings would have to be restricted to less than about 150 kPa in order to limit the settlement to less than 25 mm. Steel H-piles driven to found within the “100-blow” lower gravelly sand to silt deposit represent the most feasible and cost-effective foundation solution for this structure. As an alternative to steel H-pile foundations, caissons founded within the “100-blow” lower gravelly sand to silt deposit could also be used for support of the new structure; it is noted that this lower deposit is

water-bearing and special measures would be required for caisson construction, as discussed in Section 6.4.

As discussed further in Section 6.6, a maximum of about 60 mm to 75 mm of settlement will occur under the outer portions of the existing embankments as a result of the widened approach embankment loadings, assuming the placement of about 1.5 m to 2 m of conventional earth or granular fill on the existing embankment side slopes. Approximately one-third of the predicted settlement represents essentially elastic compression within the upper portion of the till and this will occur relatively quickly following construction (within three to six months). It will take approximately 2.5 years to complete 90 per cent of the remaining consolidation settlement within the firm to stiff silty clay till. The design of deep foundations at this site will therefore have to accommodate the downdrag loads generated by consolidation of the silty clay till deposit under the approach embankment loading, unless settlement mitigation measures are adopted.

In order to reduce the magnitude of post-construction settlement under the outer portions of the approach embankments, the use of ultra-lightweight slag fill or EPS fill for construction of the approach embankment widening is the most feasible settlement mitigation measure; the use of EPS fill provides an additional advantage in eliminating downdrag loads on the deep foundations. It is understood that there is not time in the construction staging schedule to allow preloading, which could take up to about one year in combination with surcharging, or about three to six months in combination with both surcharging and the use of wick drains installed to the base of the silty clay till deposit.

Geotechnical recommendations for the design of foundations for the bridge abutments and associated wing walls and settlement mitigation measures for the approach embankment widening 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 may be supported on steel H-piles driven to found within the “100-blow” lower gravelly sand to silt deposit. The surface of the “100-blow” soil was encountered between about Elevation 75.5 m and 74 m in the boreholes, as summarized in the table below. For design, the following pile tip levels may be assumed based on 3 m of penetration into the “100-blow” lower gravelly sand to silt deposit.

<i>Foundation Element</i>	<i>Relevant Boreholes</i>	<i>Estimated Elevation Of “100-Blow” Soil</i>	<i>Estimated Pile Tip Elevation</i>
East Abutment	502, 505	74 m to 75 m	71 m to 72 m
West Abutment	503, 504	74 m to 75.5 m	71 m to 72.5 m

In the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the silty clay to clayey silt till and lower gravelly sand to silt 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 these deposits 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 very dense lower gravelly sand to silt deposit, a factored axial resistance at Ultimate Limit States (ULS) of 1,650 kN may be assumed for design. The axial geotechnical resistance at Serviceability Limit States (SLS) may be taken as 1,450 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 very dense lower gravelly sand to silt deposit, 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 3,300 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 firm to stiff silty clay till deposit at this site. As discussed further in Section 6.6, up to about 60 mm to 75 mm of settlement will occur under the outer portion of the embankment as a result of placement of 1.5 m to 2 m of conventional earth or granular fill on the existing embankment side slopes, to achieve the final QEW grade of approximately Elevation 108 m. Negative skin friction or downdrag loads will need to be taken into account in the design of the piles supporting the abutments, unless settlement mitigation measures are adopted. The downdrag loads will apply only to the outer piles at the abutments – essentially the northern and southern 15 m for each abutment. For the "central" portion of the west and east abutments, downdrag loads do not apply.

In calculating the magnitude of the downdrag force, the methods described in both the *Canadian Foundation Engineering Manual* as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] were considered. Considering the larger predicted settlement of the

silty clay till deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the silty clay 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 400 kN. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the CHBDC.

Downdrag loads could be eliminated with the use of EPS fill as backfill behind the abutments within the embankment widening area; 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. 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:

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

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:

$$k_h = \frac{67s_u}{B} \quad \text{Where}$$

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 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.

<i>Soil Unit</i>	<i>Elevation</i>	<i>n_h</i>	<i>s_u</i>
East abutment:			
Existing compact to dense embankment fill	Above 100 m	15 MPa/m	–
Stiff to very stiff silty clay till	100 m to 94 m	–	150 kPa
Stiff silty clay till	94 m to 89 m	–	100 kPa
Firm to stiff silty clay till	89 m to 83 m	–	60 kPa
Very stiff to hard clayey silt till	83 m to 76 m	–	200 kPa
Very dense (“100-blow”) lower gravelly sand to silt	Below 76 m	35 MPa/m	–
West abutment:			
Existing compact to dense embankment fill	Above 100 m	15 MPa/m	–
Stiff to very stiff silty clay till	100 m to 96 m	–	150 kPa
Upper firm to very stiff silty clay till	96 m to 90 m	–	90 kPa
Lower firm to stiff silty clay till	90 m to 82 m	–	60 kPa
Very stiff to hard clayey silt till	82 m to 76 m	–	200 kPa
Very dense (“100-blow”) lower gravelly sand to silt	Below 76 m	35 MPa/m	–

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 (R)</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).

A maximum factored lateral resistance of 180 kN at ULS and a maximum lateral resistance of 85 kN at SLS (for 10 mm of horizontal deflection at pile cap level) are recommended for HP 310 x 110 piles, based on the flange width. 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*.

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” lower gravelly sand to silt deposit for support of the Welland Avenue replacement structure. The

surface of the “100-blow” material was encountered between about Elevation 75.5 m and 74 m in the boreholes, as summarized in the table below. The following design base elevations may be used at the abutments, based on approximately 2 m of embedment within the “100-blow” gravelly sand to silt:

<i>Foundation Element</i>	<i>Relevant Boreholes</i>	<i>Estimated Elevation Of “100-Blow” Till</i>	<i>Estimated Caisson Base Elevation</i>
East Abutment	502, 505	74 m to 75 m	72 m to 73 m
West Abutment	503, 504	74 m to 75.5 m	72 m to 73.5 m

The silty clay to clayey silt till and the lower gravelly sand to silt deposits should be expected to contain cobbles and boulders, which may pose difficulties in advancing caissons / liners.

Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons, and basal heave could occur in the gravelly sand to silt soils 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. 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 very dense nature of the gravelly sand to silt deposit, and that such difficulties could 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.

6.4.1 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” lower gravelly sand to silt soils. 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>	
	<i>ULS</i>	<i>SLS</i>
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 west and east 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 6.3.2.

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 EPS fill as backfill behind the abutments within the widening areas, 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.

6.4.3 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 350 kN at ULS, and a maximum lateral resistance of 200 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 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.

6.6 Approach Embankment Design and Construction

The existing approach embankments will be widened by up to about 5 m, generally involving placement of a “wedge” of fill, with a maximum thickness of about 1.5 m to 2 m, on top of the existing embankment side slopes.

6.6.1 Subgrade Preparation and Embankment Construction

In order to minimize differential settlement between the existing and widened portions of the approach embankments, it is recommended that all topsoil and softened / loosened soils be stripped from the existing embankment side slopes below the widening 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.

If conventional earth or granular fill is used for the widening, 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.

6.6.2 Approach Embankment Stability

Due to the limited footprint for widening of the QEW (at this location, Dunkirk and Dieppe Roads run immediately adjacent to the toe of the existing embankment side slopes), new retaining walls will be constructed along both the north and south sides of the widened QEW. The factor of safety related to global stability for properly designed and constructed retaining walls at this site will be greater than 1.3; detailed discussion regarding the retaining walls is provided in Golder’s Foundation Investigation and Design Report for the retaining walls on this project.

6.6.3 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 firm to very stiff silty clay till deposits as a result of the widening of the existing approach embankments. Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the existing and new embankment fill itself is expected to be up to about 10 mm. As noted above, the use of granular fill for the new embankment construction would minimize this magnitude, since the majority of settlement of granular fills will occur during construction.

In order to estimate the magnitude 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 correlations with the undrained shear strength, Atterberg limits, and SPT “N” values:

<i>Soil Unit</i>	<i>Bulk Unit Weight (kN/m³)</i>	<i>Preconsolidation Pressure (kPa)</i>	<i>C_c</i>	<i>C_r</i>	<i>Elastic Modulus (MPa)</i>
Embankment fill (range of parameters assumed for earth fill and granular fill)	19-21	–	–	–	–
Stiff to very stiff silty clay till ('Crust')	20	–	–	–	30
Middle firm to very stiff silty clay till	19	260 – 400	0.2 to 0.3	0.025 to 0.03	–
Lower firm to stiff silty clay till	19	210 – 300	0.35 to 0.4	0.04 to 0.05	–
Stiff to hard clayey silt till	21	–	–	–	30-50
Very dense (100-blow) lower gravelly sand to silt	22	–	–	–	100

The additional loading due to the placement of conventional earth or granular fill on the embankment side slopes will exceed the preconsolidation pressure in the firm to stiff portion of the silty clay till deposit. As a result, up to about 60 mm to 75 mm of settlement will occur within this deposit. Approximately one-third of the predicted settlement will occur relatively quickly, within about three months following embankment construction. It is estimated that 90 per cent of the consolidation settlement would be completed approximately 2.5 years following the embankment widening; this settlement would affect the widening area, including the QEW outside lanes/shoulders behind the abutments, and the concrete retaining walls at the QEW embankment toe.

If the short concrete retaining walls at the QEW embankment toe can accommodate the predicted post-construction consolidation settlement of about 40 mm to 50 mm with the use of construction slip joints, or if these concrete retaining walls are supported on deep foundations, then no

settlement other mitigation measures may be necessary in regard to these walls. In this case, the post-construction settlement of the QEW shoulder/outside lanes behind the abutments could be mitigated by padding prior to final paving of the QEW, provided that the construction staging schedule permits.

Due to the construction staging and the proximity of the local roads to the QEW embankment toe at the Welland Avenue structure, there is no opportunity for preloading or surcharging of the approach embankment widening areas prior to construction of the new Welland Avenue overpass and associated concrete retaining walls in this area. Therefore, if it is desired to reduce the magnitude of post-construction consolidation settlement under the embankment widening to closer to 25 mm, the most practical mitigation option is to use lightweight fill. Consideration could be given to the use of ultra-lightweight slag fill (having a bulk unit weight of approximately 11.5 kN/m^3) for the embankment widening. In this case, it is predicted that up to about 40 to 50 mm of settlement will occur under the widened portions of the approach embankments. Again, about one-third of this settlement will occur within about three months following construction, leaving about 25 mm to 35 mm of longer-term consolidation settlement; the majority (90 per cent) of this consolidation settlement is predicted to occur within two years following completion of the embankment widening. Based on the predicted magnitudes of settlement as discussed above, it is recommended that ultra-lightweight slag fill be used for the embankment widenings to improve the performance of the QEW pavements and the short concrete retaining walls at the embankment toe in this area. In terms of constructability, ultra-lightweight slag fill would require careful placement and compaction in order to maintain its composition and desired bulk unit weight; the slag fill would have to be tapered into the conventional earth/granular fill used for the QEW embankment widening to the east and west of the Welland Avenue overpass.

In order to reduce the magnitude of post-construction settlement under the widened approach embankments to less than 25 mm, and eliminate downdrag loading on the deep foundation elements, consideration could be given to the use of EPS fill for the embankment widening. However, there is concern regarding the durability of EPS blocks in the QEW embankment, based on the high traffic volume and heavy truck loading in this area; as a result, the use of EPS fill is not a preferred option for the approach embankment widening at this site.

6.7 Excavation and Groundwater Control

6.7.1 Open-Cut Excavations

Excavations for the pile caps will extend into the existing embankment fill. 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 fills are classified as Type 3 soil, according to the OHSA. Where space and construction staging layouts

permit, 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

It is expected that excavation support will be required in some areas for temporary roadway protection to facilitate excavation for the new foundation elements for the replacement structure. 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


The base of the existing embankment fill should be expected to be water-bearing, particularly during wet periods of the year, with groundwater "perched" atop the underlying, less permeable silty clay till deposit. It is anticipated that the groundwater in any excavations that extend to this level can be adequately controlled by pumping from properly filtered sumps.


7.0 CLOSURE

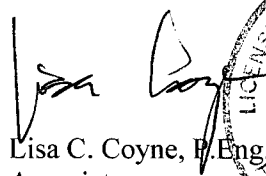
This Foundation Design Report was prepared by Shannon Palmer and Houda Jadi, P.Eng., and reviewed by Lisa Coyne, P.Eng., an Associate and Senior Engineer of Golder, with technical input from Murty Devata, P.Eng., a Specialist Foundations Consultant to Golder. Fin Heffernan, P.Eng., a Designated MTO Contact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


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TABLE 1
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
WELLAND AVENUE OVERPASS STRUCTURE

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Constructability / Practicality</i>	<i>Relative Costs</i>
Spread footings	<ul style="list-style-type: none"> • Not feasible for support of abutments 	<ul style="list-style-type: none"> • Not applicable 	<ul style="list-style-type: none"> • Low geotechnical resistance • Differential settlement across abutments 	<ul style="list-style-type: none"> • Not applicable 	<ul style="list-style-type: none"> • Not applicable
Steel H-piles driven to found within 100-blow lower gravelly sand to silt	<ul style="list-style-type: none"> • Feasible for support of abutments 	<ul style="list-style-type: none"> • Minimize differential settlement across abutments and between foundation elements • Readily installed 	<ul style="list-style-type: none"> • Piles may “hang up” on boulders within till and lower gravelly sand to silt deposits. • Piles will have to accommodate downdrag loads, unless EPS fill is adopted for the widenings. However, there is concern regarding the durability of EPS fill in the high traffic volume and heavy loading conditions for this section of QEW. 	<ul style="list-style-type: none"> • EPS fill installation would require more time and skill than conventional fill placement. As noted under disadvantages, there is concern regarding the long-term durability of the EPS fill under the high volume and heavy traffic loads on the QEW in this area. • If ultra-lightweight slag fill adopted, minor constructability issues related to placement and compaction of slag fill to preserve its structure and unit weight. 	<ul style="list-style-type: none"> • Less expensive than caissons
Caissons bored to found within 100-blow lower gravelly sand to silt	<ul style="list-style-type: none"> • Feasible for support of abutments 	<ul style="list-style-type: none"> • Minimize differential settlement across abutments and between foundation elements • Higher bearing resistances than for steel H-piles 	<ul style="list-style-type: none"> • Possibility of basal heave; temporary or permanent liners required during installation. • Potential difficulty with cobbles and boulders within till and lower gravelly sand to silt deposits • Caisson foundations will have to be designed to accommodate downdrag loads, unless EPS fill is adopted for the widenings 	<ul style="list-style-type: none"> • Comments regarding constructability and practicality of EPS and ultra-lightweight slag fill apply equally to caissons and to steel H-piles; see above. • Permanent liners recommended over temporary liners, to avoid difficulties with withdrawal of temporary liner due to length of caissons and presence of very dense soils near caisson base, and to avoid “necking” of the caissons which could occur as a result of the temporary liner withdrawal. 	<ul style="list-style-type: none"> • More expensive than steel H-piles, plus the cost of permanent liners.

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils

Consistency

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial 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 $= (\text{compressive strength})/2$
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)




PROJECT 04-1111-002			RECORD OF BOREHOLE No 501			1 OF 1 METRIC		
W.P. 607-00-00			LOCATION N 4781339.1 ; E 327298.4			ORIGINATED BY PKS		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY SLP		
DATUM Geodetic			DATE December 13, 2004			CHECKED BY LCC		
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
101.2	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
8.9	TOPSOIL							
0.1	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff to very stiff Brown Moist		1	SS	9		101	
			2	SS	17		100	45
			3	SS	22		99	
			4	SS	21		98	○
			5	SS	23		97	○
96.8			6	SS	13		96	○
4.4	SILTY CLAY, trace to some sand, trace gravel (TILL) Firm to stiff Grey Moist to wet		7	SS	7		95	×
			8	SS	8			+
94.5								×
6.7	END OF BOREHOLE							+
	Note: 1. Borehole dry upon completion of drilling operations.							

PROJECT 04-1111-002			RECORD OF BOREHOLE No 502			1 OF 3 METRIC		
W.P. 607-00-00			LOCATION N 4781342.0 ; E 327287.0			ORIGINATED BY PKS		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY SLP		
DATUM Geodetic			DATE December 13, 2004			CHECKED BY LCC		
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 (%)
101.2	GROUND SURFACE							
0.0	TOPSOIL							
100.4	Sand and gravel, trace rootlets (FILL) Loose Red Moist		1	SS	7		101	
0.8	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff to very stiff Brown Moist to wet		2	SS	9		100	
			3	SS	22		99	
			4	SS	23		98	
			5	SS	17		97	
			6	SS	14		96	
			7	SS	13		95	
95.5	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff Grey to reddish grey Wet		8	SS	11		94	
5.7			9	SS	9		93	
			10	SS	8		92	
			11	SS	8		91	
89.5	SILTY CLAY, trace to some sand, trace gravel (TILL) Firm to stiff Grey to reddish grey Wet		12	SS	5		90	
11.7			13	SS	7		89	
							88	
							87	

Continued Next Page

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

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 17/10/06

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 502		2 OF 3 METRIC									
W.P. <u>607-00-00</u>		LOCATION <u>N 4781342.0 ; E 327287.0</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>SLP</u>									
DATUM <u>Geodetic</u>		DATE <u>December 13, 2004</u>		CHECKED BY <u>LCC</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 ---						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
82.0	SILTY CLAY, trace to some sand, trace gravel (TILL) Firm to stiff Grey to reddish grey Wet		14	SS	10								
			15	SS	8								
			16	SS	7								
19.2	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Grey/red Moist to wet		17	SS	12								
			18	SS	15								
			19	SS	24								
76.8	SILTY SAND, trace gravel, trace shale fragments Dense to very dense Grey/red Wet		20	SS	38								
24.4													
			21	SS	109								
			22	SS	107								
72.1			23	SS	100/13								
29.1													

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


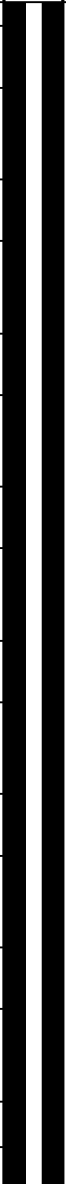



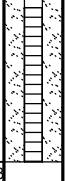
MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 17/10/06

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 502		3 OF 3 METRIC	
W.P. <u>607-00-00</u>		LOCATION <u>N 4781342.0 ; E 327287.0</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>SLP</u>	
DATUM <u>Geodetic</u>		DATE <u>December 13, 2004</u>		CHECKED BY <u>LCC</u>	

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa					W _p	W	W _L					
									20	40	60	80	100	WATER CONTENT (%)				
	END OF BOREHOLE																	
	Note:																	
	1. Water level at 24.4 m depth upon completion of drilling operations.																	

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 17/10/06

PROJECT 04-1111-002			RECORD OF BOREHOLE No 503			1 OF 3 METRIC		
W.P. 607-00-00			LOCATION N 4781350.0 ; E 327255.0			ORIGINATED BY PKS		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY SLP		
DATUM Geodetic			DATE December 16, 2004			CHECKED BY LCC		
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
101.4	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%) 10 20 30
100.9	TOPSOIL		1	SS	15		101	
100.6	Clayey silt, some sand and gravel (FILL)							
100.8	Very stiff Brown/red Moist		2	SS	8		100	
100.0	Sand, some gravel (FILL)							
100.1	Loose Red Wet		3	SS	16		99	
100.2	SILTY CLAY, trace to some sand, trace gravel (TILL)							
100.3	Stiff to very stiff Brown to brown/grey Wet		4	SS	24		98	
100.4			5	SS	16		97	
100.5			6	SS	15		96	
100.6			7	SS	12		95	
100.7							94	
100.8							93	
100.9							92	
101.0							91	
101.1							90	
101.2							89	
101.3							88	
101.4							87	
96.0	SILTY CLAY, trace to some sand, trace gravel (TILL)		8	SS	7			
96.1	Stiff Grey Wet							
96.2								
96.3								
96.4								
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PROJECT 04-1111-002			RECORD OF BOREHOLE No 503			2 OF 3 METRIC												
W.P. 607-00-00			LOCATION N 4781350.0 ; E 327255.0			ORIGINATED BY PKS												
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY SLP												
DATUM Geodetic			DATE December 16, 2004			CHECKED BY LCC												
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/m³	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 ---																		
83.5	SILTY CLAY, trace to some sand, trace gravel (TILL) Firm to stiff Grey Wet		14	SS	7		86											
								85										
								84										
17.9	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Grey Wet	15	TO	PH			83											
							82											
							81											
							80											
							79											
							78											
							77											
77.0	GRAVELLY SAND to SAND, some gravel, some silt, trace clay, containing shale fragments Dense to very dense Grey Wet		16	SS	15		83											
								82										
								81										
							80											
							79											
							78											
							77											
							76											
							75											
							74											
24.4	GRAVELLY SAND to SAND, some gravel, some silt, trace clay, containing shale fragments Dense to very dense Grey Wet	17	SS	15		81												
						80												
						79												
						78												
						77												
						76												
						75												
						74												
						73												
74.0	SANDY SILT, containing clayey silt seams Very dense Grey Wet		18	SS	22		80											
								79										
								78										
							77											
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72.2			20	SS	33		77											
29.2			21	SS	110		76											
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+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 17/10/06

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 503				3 OF 3 METRIC										
W.P. <u>607-00-00</u>		LOCATION <u>N 4781350.0 ; E 327255.0</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>SLP</u>										
DATUM <u>Geodetic</u>		DATE <u>December 16, 2004</u>				CHECKED BY <u>LCC</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 ---															
	END OF BOREHOLE Note: 1. Water level at 24.4 m depth upon completion of drilling operations. 2. Water level in piezometer at 13.7 m depth (Elevation 87.7 m) on December 20, 2005. 3. Water level in piezometer at 11.6 m depth (Elevation 90.2 m) on May 13, 2005 and on December 6, 2005.															

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 17/10/06


PROJECT 04-1111-002			RECORD OF BOREHOLE No 504			1 OF 3 METRIC		
W.P. 607-00-00			LOCATION N 4781321.0; E 327215.0			ORIGINATED BY PKS		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY SLP		
DATUM Geodetic			DATE December 20, 2004			CHECKED BY LCC		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	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
101.5	GROUND SURFACE							
0.0	Asphalt							
0.2	Sand and gravel (FILL) Compact Brown/red Moist		1	SS	28		101	
100.3			2	SS	13		100	
1.2	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff to very stiff Brown Moist		3	SS	17		99	
			4	SS	19		98	
			5	SS	16		97	
			6	SS	14		96	
			7	SS	13		95	
96.0							94	
5.5	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff Grey Wet		8	SS	8		93	
							92	
			9	SS	7		91	
							90	
			10	SS	9		89	
							88	
			11	SS	8		87	
89.5								
12.0	SILTY CLAY, trace to some sand, trace gravel (TILL) Firm to stiff Grey Wet		12	SS	7			
			13	SS	6			

Continued Next Page

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

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 17/10/06

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

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 504				3 OF 3 METRIC															
W.P. <u>607-00-00</u>		LOCATION <u>N 4781321.0;E 327215.0</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>SLP</u>															
DATUM <u>Geodetic</u>		DATE <u>December 20, 2004</u>				CHECKED BY <u>LCC</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 ---																				
71.0																					
30.5	SILT, trace to some sand, trace clay, trace gravel Very dense Grey Wet END OF BOREHOLE Note: 1. Water level in open borehole at 25.9 m depth upon completion of drilling operations.		23	SS	106		71														
70.6																					
30.9																					

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 505		1 OF 3 METRIC	
W.P. <u>607-00-00</u>		LOCATION <u>N 4781313.0; E 327227.0</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>SLP</u>	
DATUM <u>Geodetic</u>		DATE <u>December 15, 2004</u>		CHECKED BY <u>LCC</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							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED						

101.4	GROUND SURFACE													
0.0	Asphalt													
0.2	Sand and gravel (FILL) Compact Red Wet		1	SS	14		101							
			2	SS	20									
99.9							100							
1.5	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff to very stiff Brown/grey Wet		3	SS	25									
			4	SS	24		99							
			5	SS	15		98							
			6	SS	34		97						41	
			7	SS	21		96							
			8	SS	13		95							
94.5							94							
6.9	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff Grey Wet		9	TO	PH		93							
			10	SS	9		92							
			11	SS	9		91							
	Containing trace shale fragments below 12.5 m depth		12	SS	18		90							
88.0							89							
13.4	SILTY CLAY, trace to some sand, trace gravel (TILL) Firm to stiff Grey Wet		13	SS	8		88							
							87							

Continued Next Page

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

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 17/10/06

PROJECT 04-1111-002		RECORD OF BOREHOLE No 505		2 OF 3 METRIC								
W.P. 607-00-00		LOCATION N 4781313.0; E 327227.0		ORIGINATED BY PKS								
DIST Central HWY QEW		BOREHOLE TYPE 108 mm Diameter Solid Stem Augers		COMPILED BY SLP								
DATUM Geodetic		DATE December 15, 2004		CHECKED BY LCC								
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					
--- CONTINUED FROM PREVIOUS PAGE ---												
84.0 17.4	SILTY CLAY, trace to some sand, trace gravel (TILL) Firm to stiff Grey Wet		14	SS	12							
			15	TO	PH							
75.5 25.9	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Grey Wet		16	SS	16							
			17	SS	18							
			18	SS	31							
			19	SS	27							3 24 58 15
			20	SS	27							
			21	SS	49							
			22	SS	100/20							

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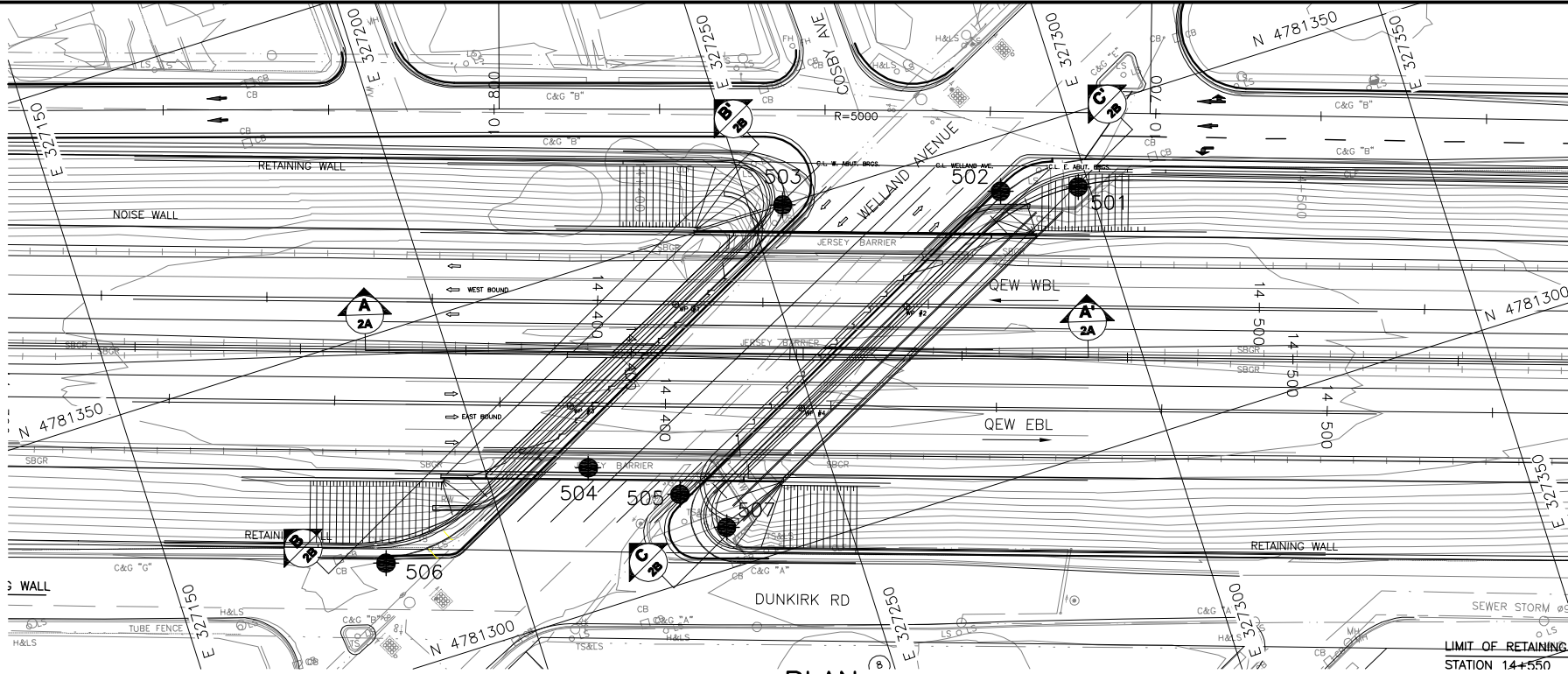
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 17/10/06

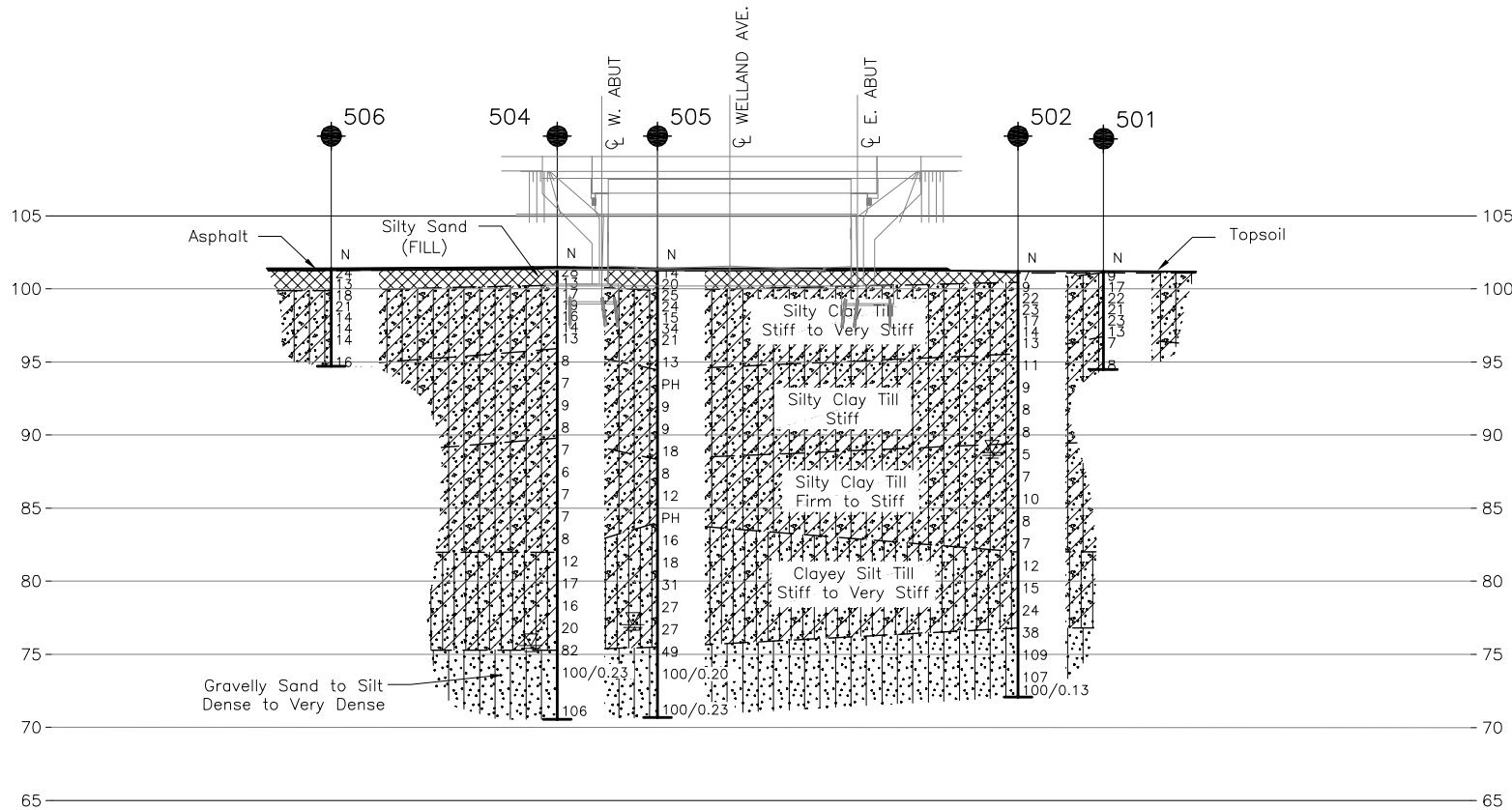
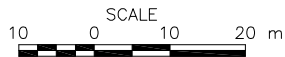
PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 505				3 OF 3 METRIC														
W.P. <u>607-00-00</u>		LOCATION <u>N 4781313.0 ; E 327227.0</u>				ORIGINATED BY <u>PKS</u>														
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>				COMPILED BY <u>SLP</u>														
DATUM <u>Geodetic</u>		DATE <u>December 15, 2004</u>				CHECKED BY <u>LCC</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 ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> 10 20 30 </div>								
70.7			23	SS	100/23		71													
30.7	END OF BOREHOLE Note: 1. Water level in open borehole at 24.4 m depth upon completion of drilling operations.																			

PROJECT 04-1111-002			RECORD OF BOREHOLE No 506			1 OF 1 METRIC		
W.P. 607-00-00			LOCATION N 4781316.5; E 327181.5			ORIGINATED BY PKS		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY SLP		
DATUM Geodetic			DATE December 21, 2004			CHECKED BY LCC		
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 NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
101.4	GROUND SURFACE							
0.0	Asphalt							
0.2	Sand and gravel (FILL) Compact Brown/red Moist		1	SS	24		101	
			2	SS	13		100	
99.9								
1.5	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff to very stiff Brown to brown/grey Moist		3	SS	18		99	
			4	SS	21		98	
			5	SS	14		97	
			6	SS	14		96	
			7	SS	14		95	
94.7			8	SS	16			
6.7	END OF BOREHOLE							
	Note: 1. Borehole dry upon completion of drilling operations.							

PROJECT 04-1111-002			RECORD OF BOREHOLE No 507			1 OF 1 METRIC											
W.P. 607-00-00			LOCATION N 4781306.1 ; E 327232.2			ORIGINATED BY PKS											
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY SLP											
DATUM Geodetic			DATE December 14, 2004			CHECKED BY LCC											
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/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	20 40 60 80 100	10 20 30				
101.5	GROUND SURFACE																
0.0	Sandy silt, trace gravel (FILL) Compact Red Moist		1	SS	15		101										
100.4			2	SS	11		100										
1.1	SILTY CLAY, trace to some sand, trace gravel and shale fragments (TILL) Stiff to hard Brown/grey Moist		3	SS	28		99										
			4	SS	31		98										
			5	SS	36		97										
			6	SS	18		96										
			7	SS	16		95										
96.0																	
5.5	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff Grey Wet		8	SS	10												
94.7																	
6.7	END OF BOREHOLE																
Notes:																	
1. Borehole dry upon completion of drilling operations.																	
2. Water level in piezometer at 6.0 m depth (Elevation 95.5 m) on December 20, 2004.																	
3. Water level in piezometer at 1.6 m depth (Elevation 99.9 m) on May 13, 2005.																	
4. Water level in piezometer at 1.9 m depth (Elevation 99.6 m) on December 6, 2005.																	



PLAN



QEW & PROFILE A-A'



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

CONT No.
WP No. 607-00-00

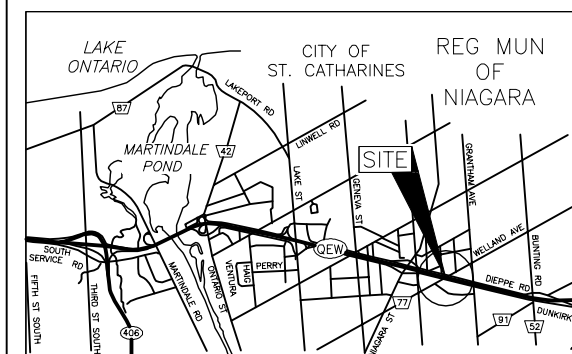
WELLAND AVENUE OVERPASS
QEW WIDENING, ST. CATHARINES
BOREHOLE LOCATIONS AND
SOIL STRATA



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
0 1 km

LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
501	101.2	4781339.1	327298.4
502	101.2	4781342.0	327287.0
503	101.4	4781350.0	327255.0
504	101.5	4781321.0	327215.0
505	101.4	4781313.0	327227.0
506	101.4	4781316.5	327181.5
507	101.5	4781306.1	327232.2

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 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. 18160-01.dwg, received October 16, 2006; x4026design.dwg and x4026baseplan.dwg, received March 21, 2005.



NO.	DATE	BY	REVISION
Geocres No.			
HWY. QEW		PROJECT NO. 04-1111-002	DIST.
SUBM'D. SLP	CHKD. LCC	DATE: OCT 2006	SITE: 18-160
DRAWN: MSM	CHKD. SLP	APPD. LCC	DWG. 1

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

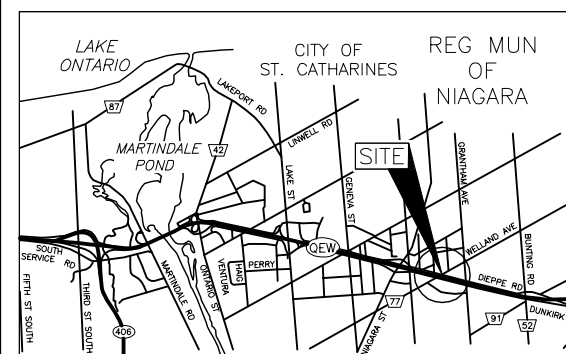
CONT No.
WP No. 607-00-00

WELLAND AVENUE OVERPASS
QEW WIDENING, ST. CATHARINES
SOIL STRATA

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



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 MMM DD, YYYY
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
501	101.2	4781339.1	327298.4
502	101.2	4781342.0	327287.0
503	101.4	4781350.0	327255.0
504	101.5	4781321.0	327215.0
505	101.4	4781313.0	327227.0
506	101.4	4781316.5	327181.5
507	101.5	4781306.1	327232.2

NOTES

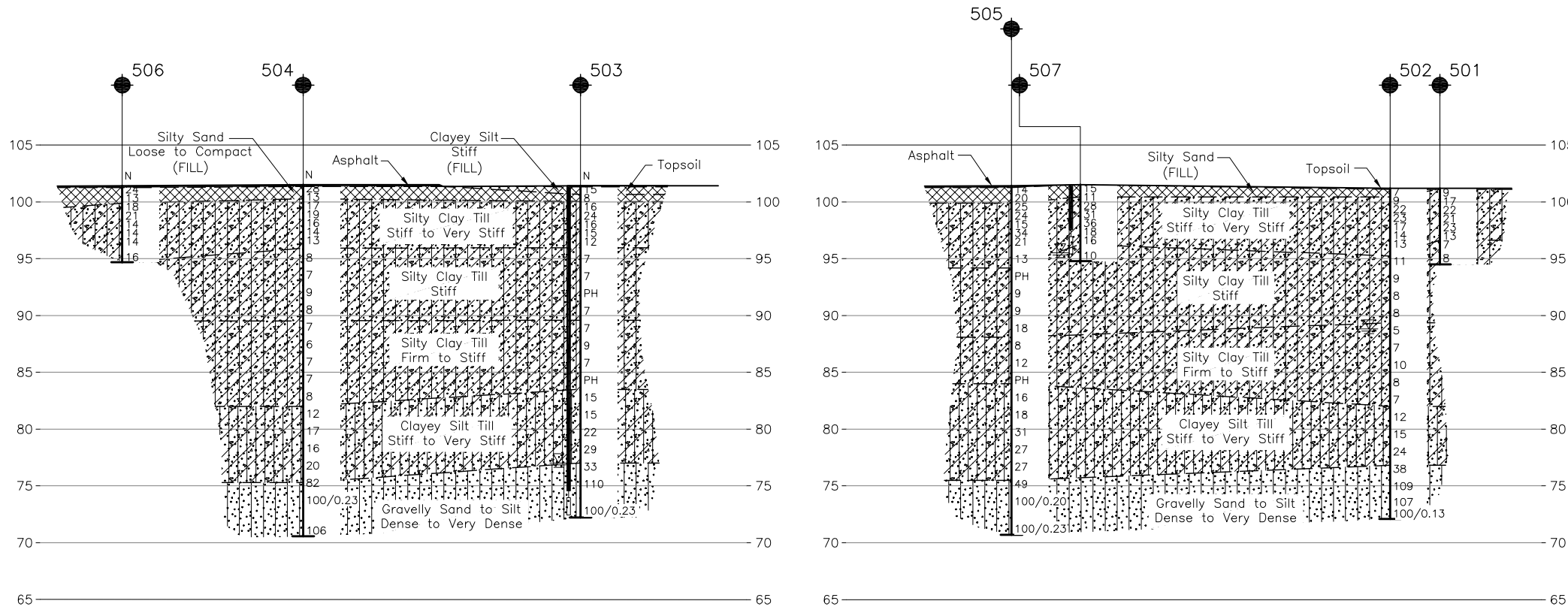
This drawing is for subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

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REFERENCE

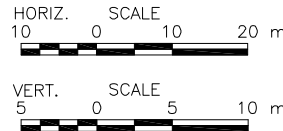
Base plans provided in digital format by Morrison Hershfield Limited, drawing file nos. 18160-01.dwg, received October 16, 2006; x4026design.dwg and x4026baseplan.dwg, received March 21, 2005.



SECTION B-B' WEST ABUTMENT



SECTION C-C' EAST ABUTMENT

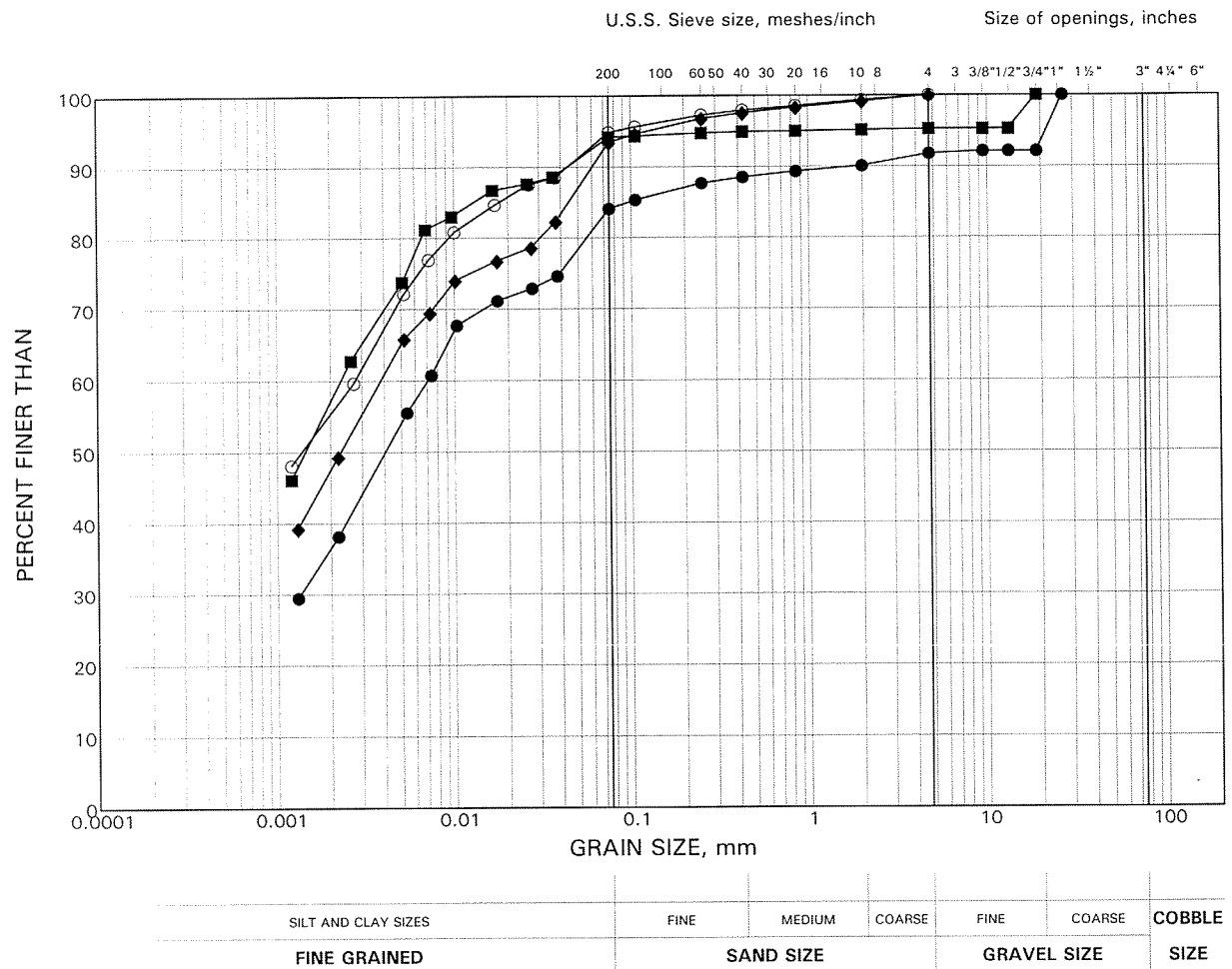


NO.	DATE	BY	REVISION
Geocres No.			
HWY. QEW	PROJECT NO. 04-1111-002		DIST.
SUBM'D. SLP	CHKD. LCC	DATE: OCT 2006	SITE: 18-160
DRAWN: MSM	CHKD. SLP	APPD. LCC	DWG. 2

GRAIN SIZE DISTRIBUTION TEST RESULTS

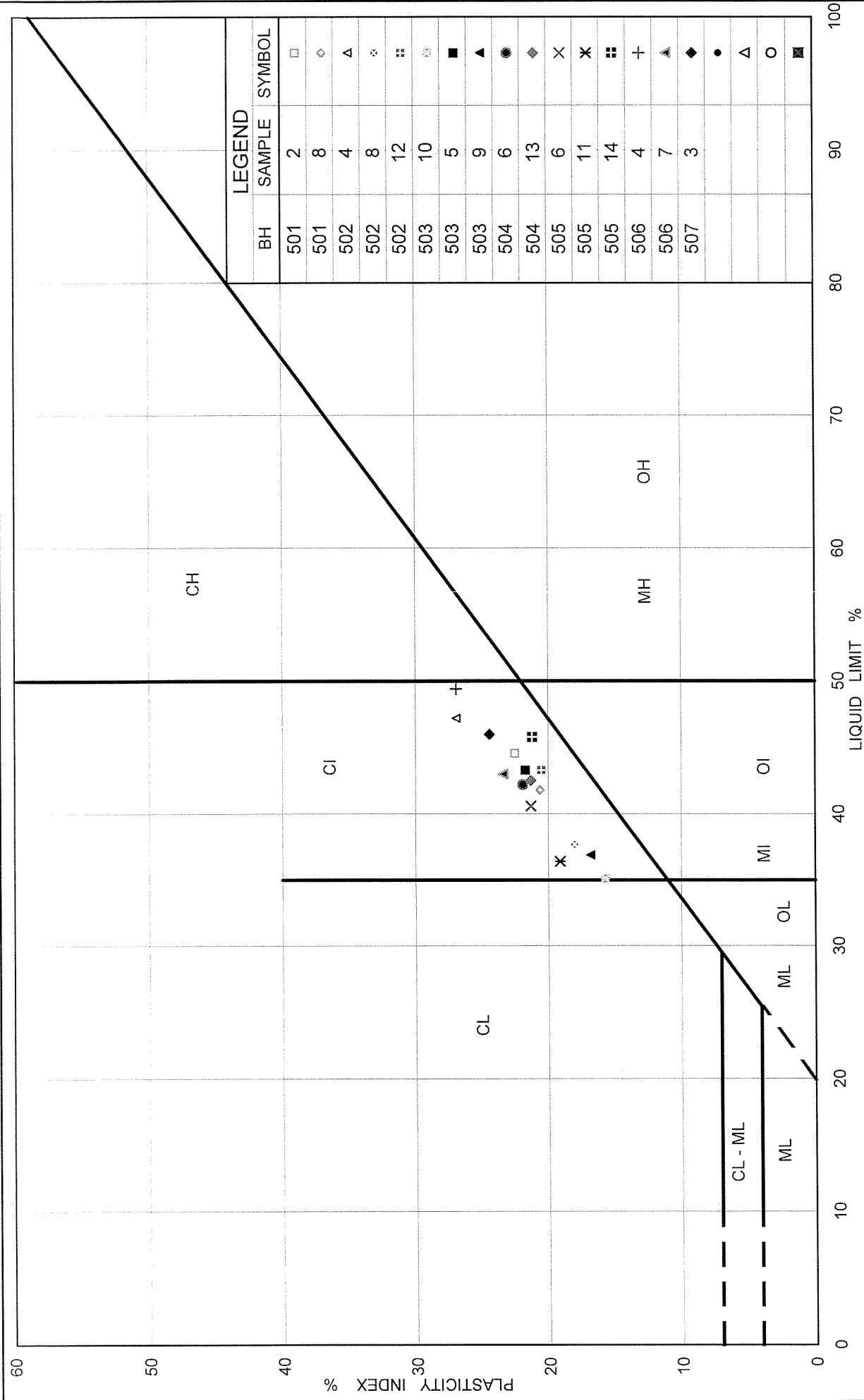
Silty Clay Till

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	502	15	84.1
■	503	13	87.4
◆	504	10	92.1
○	507	6	97.4



CONSOLIDATION TEST SUMMARY **SILTY CLAY TILL**

FIGURE 3A

SAMPLE IDENTIFICATION

Project Number	04-1111-002	Sample Number	10
Borehole Number	503	Sample Depth, m	9.1-9.8

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	01/15/2005		
Date Completed	01/31/2005		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	19.81
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	15.45
Area, cm ²	31.65	Specific Gravity, measured	2.76
Volume, cm ³	60.13	Solids Height, cm	1.084
Water Content, %	28.22	Volume of Solids, cm ³	34.32
Wet Mass, g	121.45	Volume of Voids, cm ³	25.81
Dry Mass, g	94.72	Degree of Saturation, %	103.5

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	1.900	0.752	1.900				
4.83	1.900	0.752	1.900	5	1.53E-01	0.00E+00	0.00E+00
9.46	1.897	0.749	1.899	64	1.19E-02	3.41E-04	3.99E-07
19.51	1.889	0.742	1.893	271	2.80E-03	4.19E-04	1.15E-07
38.91	1.874	0.728	1.882	287	2.61E-03	4.07E-04	1.04E-07
77.57	1.855	0.711	1.865	197	3.74E-03	2.59E-04	9.48E-08
154.88	1.827	0.685	1.841	124	5.79E-03	1.91E-04	1.08E-07
38.91	1.844	0.701	1.836			7.72E-05	
9.46	1.868	0.723	1.856			4.29E-04	
4.83	1.876	0.730	1.872			9.09E-04	
9.46	1.874	0.728	1.875	124	6.01E-03	2.27E-04	1.34E-07
19.51	1.868	0.723	1.871	375	1.98E-03	3.14E-04	6.09E-08
38.91	1.856	0.712	1.862	394	1.87E-03	3.26E-04	5.95E-08
77.57	1.841	0.698	1.849	287	2.52E-03	2.04E-04	5.05E-08
154.88	1.820	0.678	1.831	171	4.15E-03	1.43E-04	5.82E-08
309.55	1.769	0.631	1.795	271	2.52E-03	1.74E-04	4.28E-08
618.87	1.685	0.554	1.727	404	1.57E-03	1.43E-04	2.19E-08
1238.51	1.593	0.469	1.639	366	1.56E-03	7.81E-05	1.19E-08
2475.91	1.499	0.382	1.546	197	2.57E-03	4.00E-05	1.01E-08
1238.51	1.513	0.395					
309.55	1.554	0.433					
77.57	1.605	0.480					
19.51	1.664	0.535					
4.83	1.704	0.571					

Notes:

k calculated using cv based on t₉₀ values.

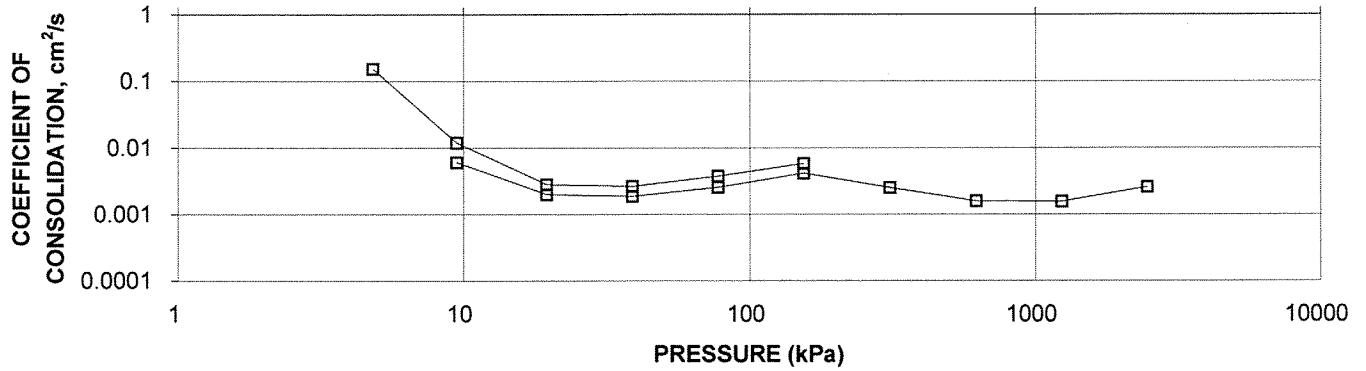
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.70	Unit Weight, kN/m ³	21.51
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	17.22
Area, cm ²	31.65	Specific Gravity, measured	2.76
Volume, cm ³	53.93	Solids Height, cm	1.084
Water Content, %	24.87	Volume of Solids, cm ³	34.32
Wet Mass, g	118.28	Volume of Voids, cm ³	19.61
Dry Mass, g	94.72		

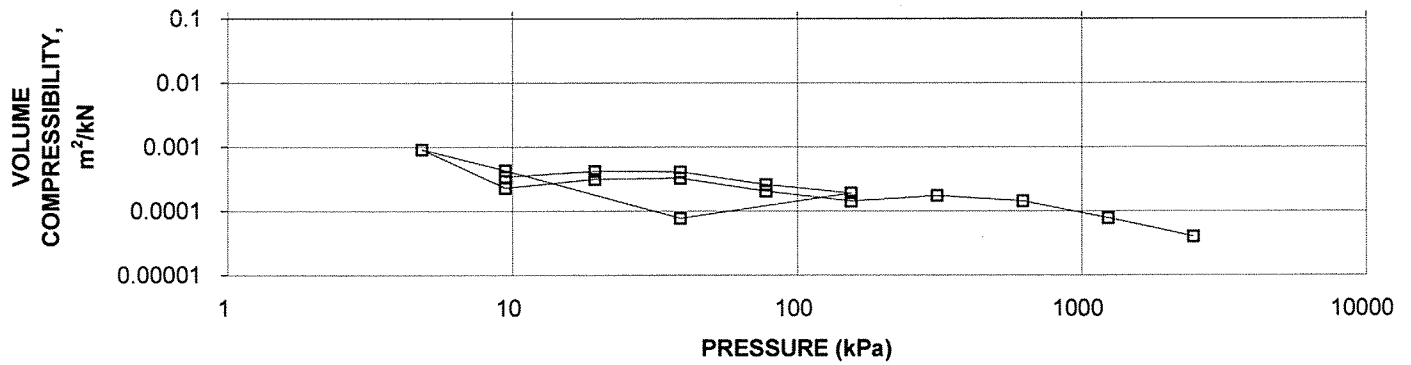
**CONSOLIDATION TEST SUMMARY
SILTY CLAY TILL**

FIGURE 3B

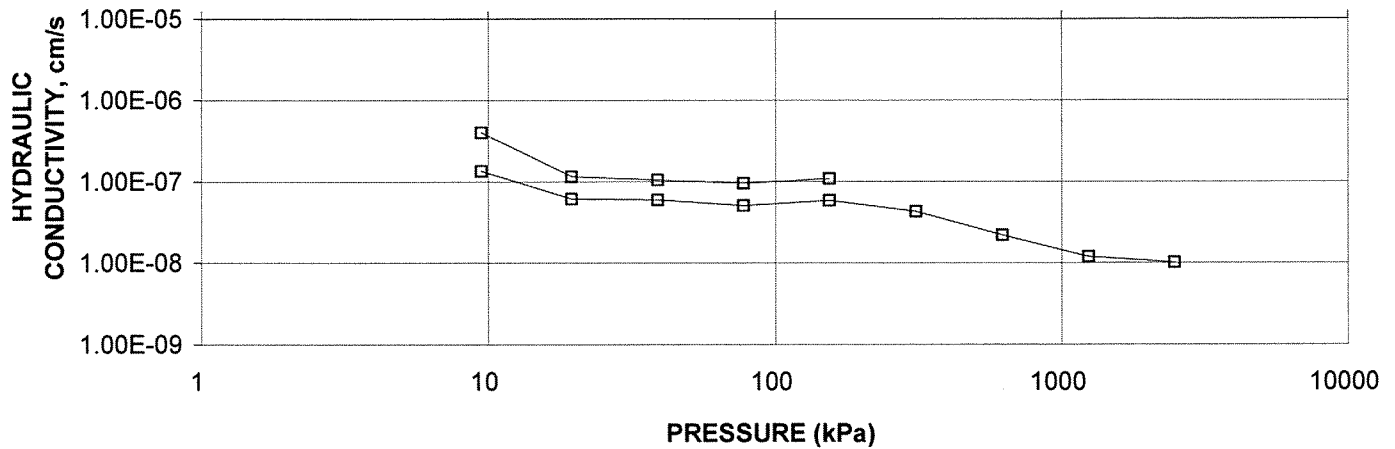
**CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH 503 SA 10**



**CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 503 SA 10**



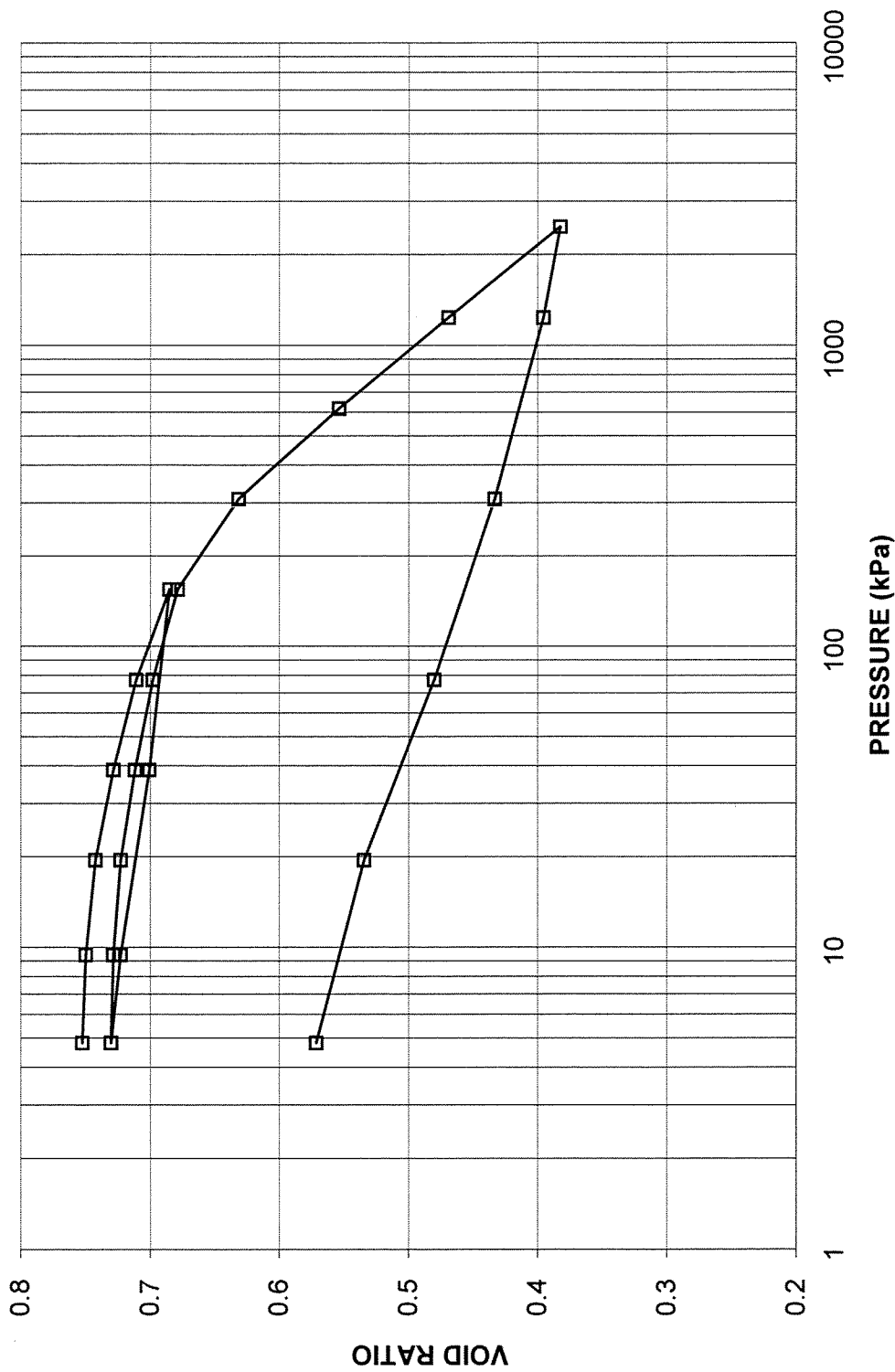
**CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 503 SA 10**



CONSOLIDATION TEST RESULTS
SILTY CLAY TILL

FIGURE 3C

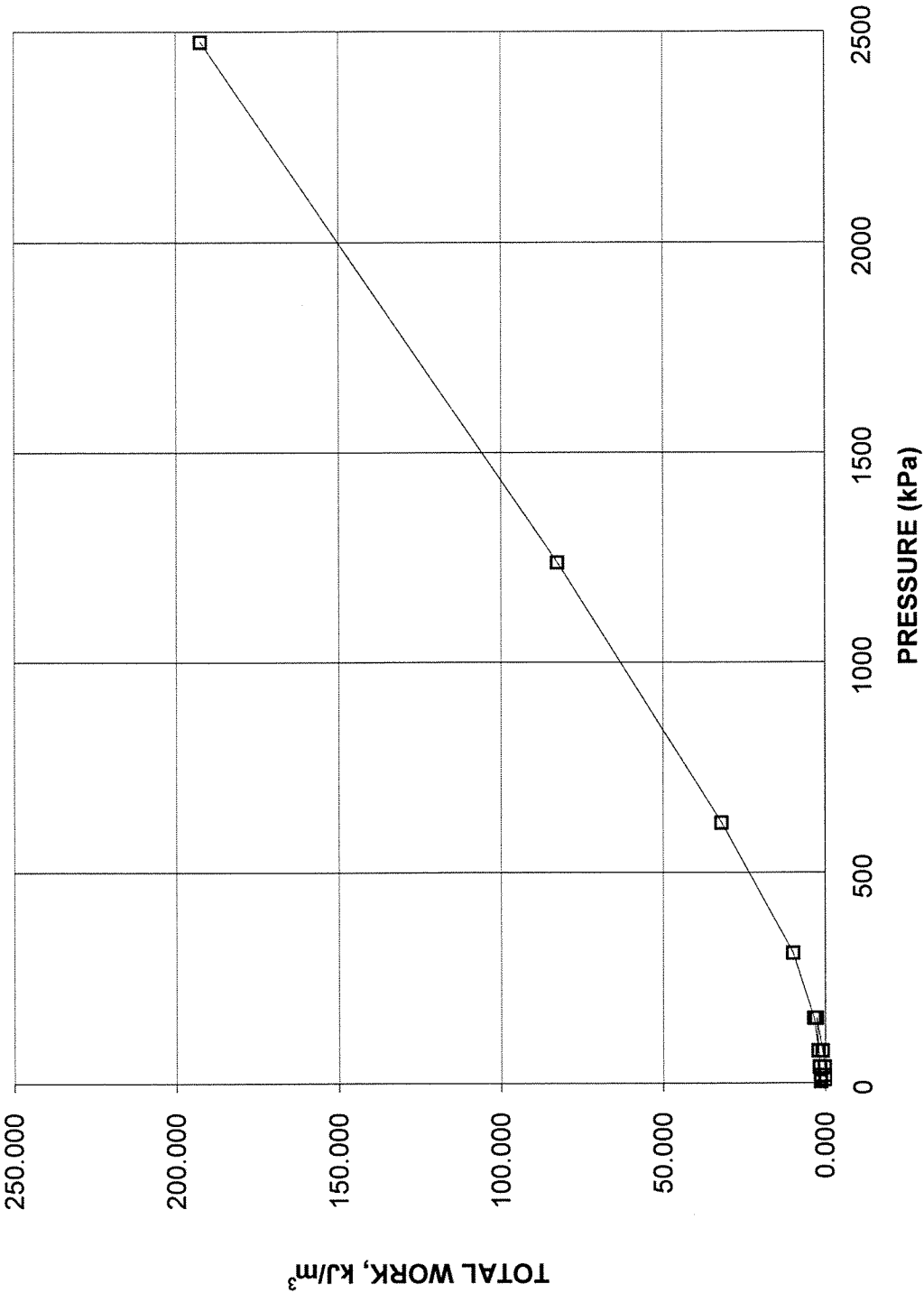
CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BOREHOLE 503, SAMPLE 10



CONSOLIDATION TEST RESULTS
SILTY CLAY TILL

FIGURE 3D

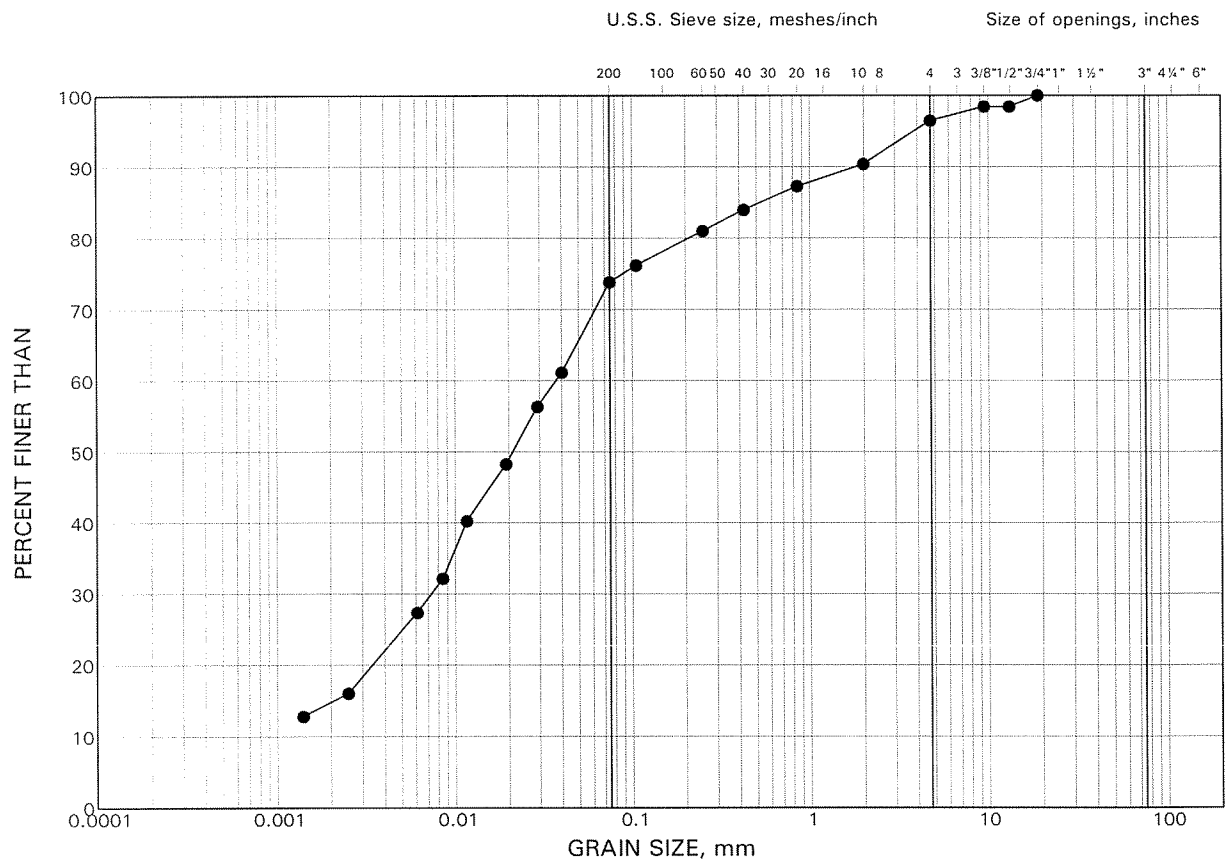
CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BOREHOLE 503, SAMPLE 10



GRAIN SIZE DISTRIBUTION TEST RESULT

Clayey Silt Till

FIGURE 4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	505	19	79.2

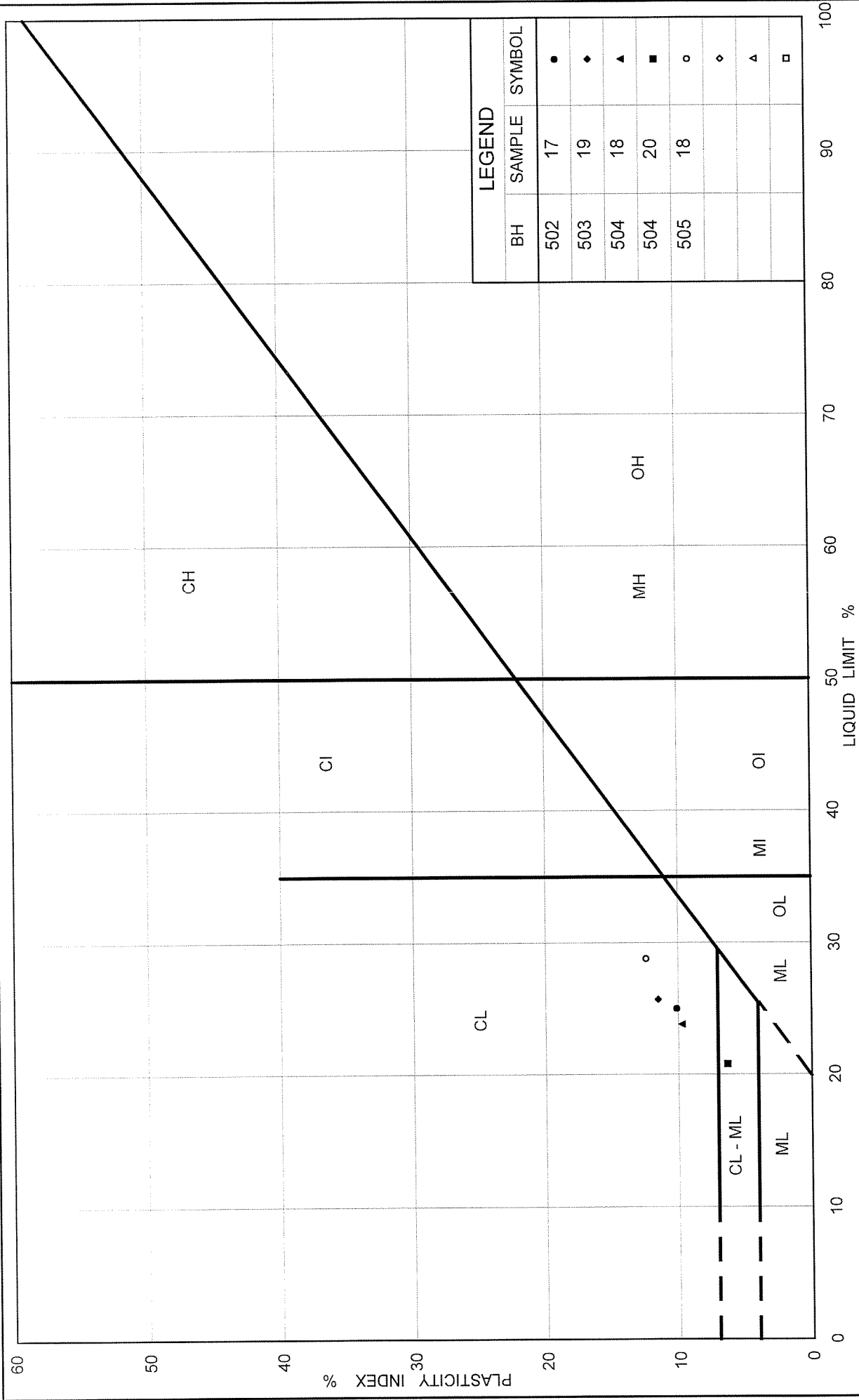


FIG No. 5

PLASTICITY CHART
Clayey Silt Till

Ministry of Transportation

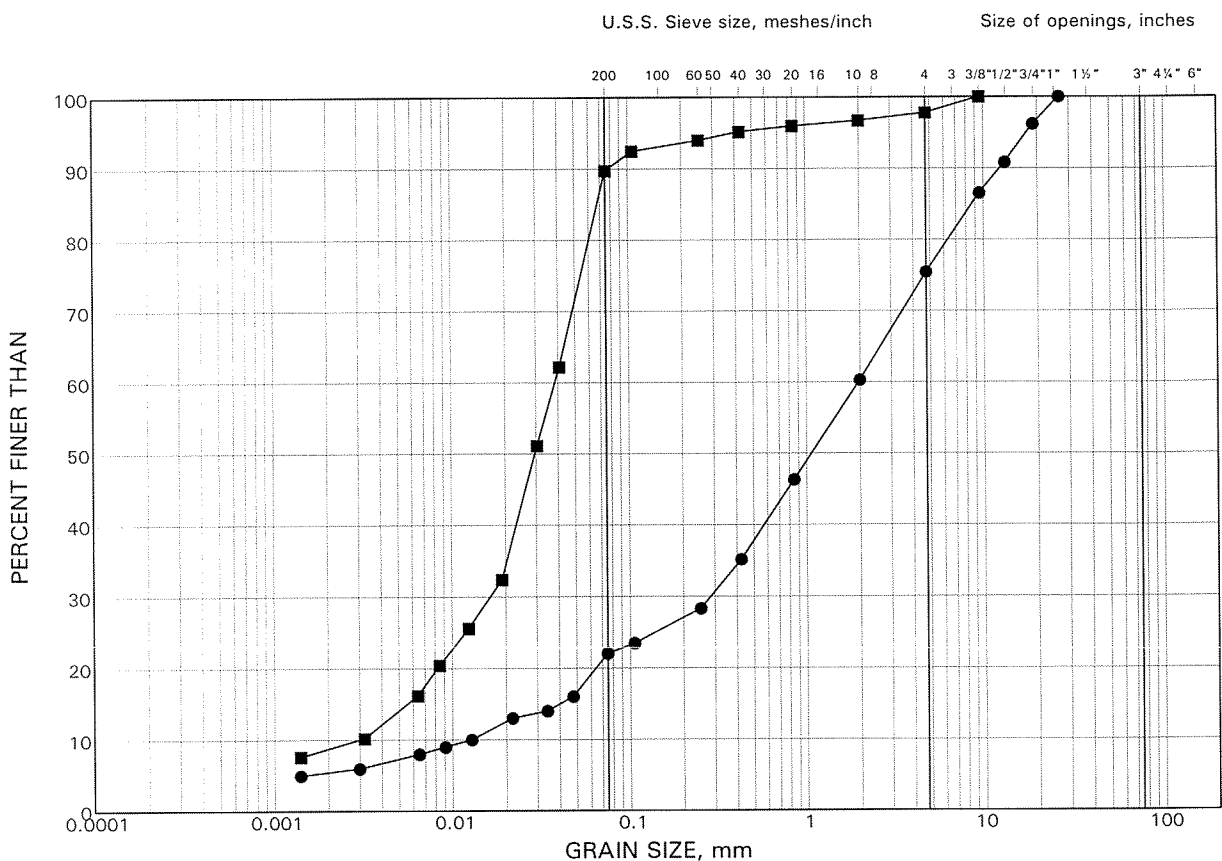


Ontario

GRAIN SIZE DISTRIBUTION TEST RESULTS

Gravelly Sand to Silt

FIGURE 6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	503	21	75.2
■	504	23	70.7

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