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Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing

FOUNDATION INVESTIGATION & DESIGN REPORT
ETOBICOKE CREEK BRIDGE
HIGHWAY 410 EXTENSION – PHASE III
FROM 300 m EAST OF HEART LAKE ROAD TO HIGHWAY 10
AGREEMENT No. 2005-A-000230, W.P. 107-00-01, SITE: 24-744

PREPARED FOR: Giffels Associates Ltd.
30 International Blvd.
Toronto, Ontario

Attention: Mr. Stephen Chiu, P.Eng.
Manager, Transportation Engineering

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Terraprobe Limited.
10 Bram Court
Brampton, Ontario
L6W 3R6
Phone: (905) 796 2650
Fax: (905) 796 2250

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of the Etobicoke Creek bridge on the proposed four-lane of Highway 410 in the Town of Caledon, Ontario. Previous, preliminary investigations in the general vicinity of this site were carried out by Golder Associates Ltd. (Golder) and the Ministry of Transportation (MTO), and the factual data from these investigations have been used as general reference for the preparation of this report.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained.

Terraprobe conducted the investigation as a sub-consultant to Giffels Associates Ltd., under the Ministry of Transportation Ontario (MTO) Agreement Number 2005-A-000230.

The following documents are referenced in the preparation of this report:

- Golder Associates Ltd., "Supplementary Foundation Feasibility Investigation, Proposed Highway 410 Extension, Bovaird Drive to Highway 10", W.P. 22-79-00, MTO District 6, Toronto, GEOCRES No. 30M12-208, dated April 1999.
- Ministry of Transportation, "Highway 410 Route Planning Study, Bovaird Drive Northerly to Highway 10", W.P. 22-79-00, MTO District 6, GEOCRES 30M12-208, dated January 24, 1989.

2 SITE DESCRIPTION

The site is located about 310 m west of Kennedy Road and north of Mayfield Road in the Town of Caledon. Westerly from Kennedy Road and along the proposed Hwy. 410 alignment the ground surface elevation decreases to the flood plain level and the east bank of the creek by about 19 m over a horizontal distance of 275 m. Beyond the west bank of the creek the ground surface rises by about 8 m to the top of bank, over a horizontal distance of 20 m and then becomes relatively level beyond.

Planted conifers exist on the floodplain on the east side of the creek. Further beyond, the area is vegetated with mature stands of deciduous and coniferous trees and light vegetation consisting of grass and small shrubs.

Etibocoke creek meanders within its floodplain flowing south towards Lake Ontario.

The site is located in the physiographic region of Southern Ontario referred to as the Peel Plain whose topography slopes gradually and gently towards Lake Ontario. Etobicoke Creek and other rivers have cut deep valleys across the Peel Plain.

The Peel Plain is known to consist of generally clayey and silty soils that cover the central portion of the regions of York, Peel and Halton[1]. There are exceptions to be noted in these major soil groups. Trains of sandy alluvium can be found at various places in the stream valleys. These overburden soils are underlain by the Georgian Bay Formation.

The Georgian Bay Formation is of Middle Ordovician Age. This unit consists essentially of blue-grey and green-grey shales with interbeds of calcareous sandstone, siltstone and grey argillaceous limestone. The limestone and siltstones are very fossiliferous.

Stress relief features such as folds and faults are common in the Georgian Bay Formation. In these features the rock is heavily fractured and sheared, and contains layers of shale rubble and clay. Weathering is much deeper than the surrounding rock in these features and often there can be lateral displacement of the stress relief features resulting in sound unweathered bedrock overlying fractured and weathered bedrock. Weathering near the soil contact is generally accompanied by softening and increased fissility.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between March 24 and April 07, 2005 and consisted of drilling and sampling six boreholes to depths ranging from 6.6 m to 36.1 m. The boreholes were numbered EC-1, EC-2, EC-4, EC-5, EC-5A and EC-6 and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix D.

The borehole locations were marked in the field by surveyors from Shiu Geomatics Limited who also provided Terraprobe with the coordinates and geodetic elevations. Utility clearances were obtained by Terraprobe prior to drilling.

The drilling, sampling and in-situ testing operations were conducted with a track mounted CME 75 drill rig owned and operated by Groundworks Drilling Limited of Toronto, Ontario. A combination of solid and hollow-stem auger drilling techniques and casing and washboring

[1] Chapman and Putnam, "The Physiography of South Ontario", 3rd Edition, 1984.

methods were used to advance the boreholes. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. The boreholes at the abutments and pier locations were also advanced 3.0 m to 3.1 m into bedrock by NQ size diamond coring techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers consisting of 19 mm PVC pipe with a slotted screen were installed and enclosed in sand in selected boreholes to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1.

Table 3.1 – Piezometer Installation Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
EC-2	21.3/232	Piezometer with 1.5 m slotted screen installed with filter sand to 19.5 m, bentonite seal from 19.5 m to 18.9 m, drill cuttings from 18.9 m to 0.6 m and bentonite seal from 0.6 m to ground surface.
EC-4	30.9/215.2	Piezometer with 1.5 m slotted screen installed with filter sand to 24.3 m, bentonite seal from 24.3 m to 24.1 m, drill cuttings from 24.1 m to 0.9 m and bentonite seal from 0.9 m to ground surface.
EC-5	27.6/219.3	Piezometer with 1.5 m slotted screen installed with filter sand to 25.6 m, bentonite seal from 25.6 m to 25.3 m, drill cuttings from 25.3 m to 0.9 m and bentonite seal from 0.9 m to ground surface.

Artesian conditions were encountered in Borehole EC-4 during the field investigations and the piezometer in this borehole was decommissioned shortly thereafter by removing the standpipe, augering to the water bearing layers and backfilling with bentonite seal.

The drilling and sampling operations were supervised on a full time basis by a member of Terraprobe's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Terraprobe's Brampton laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR) and Rock Quality Designation (RQD) were determined.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to gradation analysis and Atterberg Limits tests were conducted on samples retrieved from the cohesive deposits. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and the figures are presented in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A and Borehole 123A drilled by Golder as part of the preliminary investigation (Appendix C). Details of the encountered soil and rock stratigraphy are presented in these appendices and on the "Borehole Locations and Soil Strata" drawing in Appendix D. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general, the site is underlain by topsoil and about 28 m to 33 m of overburden soils consisting of sands and silts, clayey silt, till soils and cobbles and boulders. These soils are underlain by the Georgian Bay formation.

5.1 Topsoil and Peat

Topsoil ranging from 150 mm to 200 mm thick was encountered across the site. Topsoil thickness may vary between and beyond the boreholes.

A 0.76 m thick layer of peat was encountered in Golder's Borehole 123A drilled in March 1999.

5.2 Clayey Silt and Silty Clay

Layers of clayey silt were encountered in some boreholes. An upper brown clayey silt layer containing trace to some sand, trace gravel, trace rootlets and occasional wood pieces was encountered on the west side of the creek. This upper clayey silt layer extends to depths of 0.7 m (Elev. 252.3 m) and 1.4 m (Elev. 251.9 m) below ground surface.

Deeper layers of clayey silt and silty clay approximately 4.6 m and 3.1 m thick were encountered in Borehole EC-5 and Golder's Borehole 123A respectively. These layers extend to depths of 13.2 m (Elev. 233.7 m) and 7.1 m (Elev. 239.5 m) below ground surface.

A sample from the lower clayey silt deposit was subjected to a grain size distribution test and the results are illustrated in Figure B1. These results show a grain size distribution consisting of 0 % gravel, 1 % sand, 81 % silt and 18 % clay size particles.

SPT 'N' values ranged from 4 to 8 blows for 0.3 m penetration in the surficial layers indicating a firm consistency. Standard Penetration tests in the deeper clayey silt and silty clay deposits gave 'N' values that ranged from 45 to 121 blows per 0.3 m penetration. Based on these results the deeper clayey silt and silty clay deposits are described as hard. The moisture content of samples from these deposits varies from 15% to 28% by weight.

5.3 Silty Sand

An upper layer of silty sand and sandy silt was encountered in most of the boreholes. This deposit was not encountered at Borehole EC-6. This layer extends to depths ranging from 0.7 m to 4.0 m below ground surface or to elevations ranging from 242.6 m to 252.6 m. In Borehole 123A this sandy silt stratum is divided by a 0.4 m thick layer of dense gravelly sand.

A deeper deposit of silty sand and sand was encountered across the site. This deposit extends to depths ranging from 16.2 m (Elev. 229.9 m) to 22.3 m (Elev. 231.0 m) below ground surface.

Samples from the upper and lower deposits were subjected to grain size distribution tests and the results are illustrated in Figure B2. These results show a grain size distribution consisting of 0 % gravel, 60-86 % sand, 10-33 % silt and 4-7 % clay size particles.

The upper deposit is considered to have a very loose to compact relative density based on SPT 'N' values that ranged from 2 to 17 blows for 0.3 m penetration. The lower layer is considered to have a loose to dense relative density based on SPT 'N' values that ranged from 7 to 45 blows for 0.3 m penetration. The moisture content of samples from the upper and lower deposits ranges from 14% to 30% by weight.

5.4 Sand and Gravel

Discontinuous layers of sand and gravel and gravelly sand were encountered. These deposits extend to depths ranging from 1.8 m (Elev. 244.8 m) to 2.1 m (Elev. 251.2 m) below ground surface.

A sample from this stratum was subjected to a grain size distribution test and the results are illustrated in Figure B3.

Standard Penetration tests in this deposit gave 'N' values that ranged from 26 to 30 blows per 0.3 m penetration. Based on these results the deposit is considered to have a compact relative density. The moisture content of samples from this stratum ranged from 21% to 27% by weight.

5.5 Sand and Silt Till

The site is underlain by upper and lower deposits of sand and silt till. The upper till deposit was fully penetrated in the deeper boreholes (Boreholes EC-2, EC-4 and EC-5) where it extends to depths of 3.7 m to 10.1 m or to elevations ranging from 238.3 m to 243.2 m.

A lower sand and silt till layer was also encountered in the deeper boreholes at depths ranging from 19.3 m to 23.9 m below ground surface. This lower till layer extends to depths of 26.9 m to 30.5 m or to elevations ranging from 216.4 m to 224.9 m.

The results of grain size distribution tests conducted on samples obtained from the upper and lower deposits are illustrated in Figures B4 and B5 respectively. These results show grain size distributions consisting of 1-18% gravel, 30-79% sand, 11-64% silt and 3-8% clay size particles.

Standard Penetration tests in these deposits gave 'N' values that ranged from 7 to more than 100 blows per 0.3 m penetration but generally 'N' values ranged from 22 to more than 100 blows per 0.3 m penetration. Based on these results the deposits are considered to have a generally compact to very dense relative density with occasional loose to compact zones. The moisture content of samples from this stratum ranged from 3% to 27% by weight.

5.6 Clayey Silt Till

Discontinuous upper and lower layers of clayey silt till were encountered across the site in some boreholes. The upper till deposit ranges from 0.7 m to 1.2 m in thickness and it extends to depths ranging from 1.4 m to 2.9 m below ground surface or to elevations of 244.8 m to 252.3 m.

A lower layer of clayey silt till was encountered in Boreholes EC-2 and EC-4. It is about 1 m to 2.3 m thick and extends to depths of 27.9 m (Elev. 218.2 m) and 30.7 m (Elev. 222.6 m) where it overlies the bedrock surface.

The grain size distribution plots of tested samples from these till deposits are presented in Figure B6. These results show a grain size distribution consisting of 2-27% gravel, 28-41% sand, 25-40% silt and 11-26% clay size particles.

Three samples were also subjected to Atterberg Limits tests and the results are presented in Figure B7. The index values from these tests are summarized below:

Liquid Limit:	24-26%
Plastic Limit:	14-15%
Plasticity Index:	10-11%
Natural Moisture Content:	10-16%

These values are characteristic of clayey soils of low plasticity.

Standard Penetration tests in the upper till layer yielded 'N' values ranging from 3 to 26 blows per 0.3 m penetration indicating a soft to very stiff consistency. Blow counts more than 100 blows per 0.3 m penetration were obtained in the lower till layer indicating a hard consistency. The moisture content of samples from both of these deposits ranged from 8% to 20% by weight.

5.7 Silt

Layers of silt were encountered across the site. This silt deposit extends to depths of 13.2 m to 19.3 m or to elevations ranging from 227.3 m to 235.5 m.

Samples from this deposit were subjected to grain size distribution tests and the results are illustrated in Figure B8. These results show a grain size distribution consisting of 0 % gravel, 1-2 % sand, 90-93 % silt and 5-10 % clay size particles. A sample from Borehole EC4 was also subjected to an Atterberg Limits test and the results presented in Figure B9 indicate a non-plastic silt.

Standard Penetration tests in this deposit gave 'N' values ranging from 10 to 57 blows per 0.3 m penetration. Based on these results the deposit is considered to have a compact to very dense relative density. The moisture content of samples from this stratum ranged from 16% to 26% by weight.

5.8 Sandy Silt

A deposit of grey sandy silt was encountered across the site. This deposit extends to depths of 19.3 m to 23.9 m or to elevations ranging from 224.6 m to 229.4 m.

Samples from this deposit were subjected to grain size distribution tests and the results are illustrated in Figure B10. These results show a grain size distribution consisting of 0% gravel, 14-34% sand, 56-80% silt and 4-10% clay size particles.

The blow counts from Standard Penetration tests in this deposit ranged from 7 to 51 blows per 0.3 m penetration. Based on these results the deposit is considered to have a loose to very dense relative density. The moisture content of samples from this stratum ranged from 19% to 25% by weight.

5.9 Cobbles and Boulders

A 2.6 m thick deposit of cobbles and boulders was encountered above the bedrock in Borehole EC-5A at a depth of 30.5 m below ground surface. This deposit extends to a depth of 33.1 m (Elev. 213.8 m), where it overlies the bedrock surface.

Based on a recorded 'N' value of more than 100 blows per 0.3 m penetration, this layer is considered to have a very dense relative density.

5.10 Bedrock (Georgian Bay Formation)

The overburden soils described above are underlain by the Georgian Bay Formation. Bedrock was proved by coring at the abutments and pier locations. Table 5.1 summarizes the bedrock depth and the elevations to the top of bedrock.

Table 5.1 – Depth to Bedrock

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
West Abutment	EC-2	30.7	222.6
East Abutment	EC-5A	33.1	213.8
Pier	EC-4	27.9	218.2

The bedrock is described as slight to highly weathered and its colour is grey. It is thin to medium bedded with fossiliferous limestone interbeds. Core recovery in the bedrock ranged from 42% to 88%. The RQD values generally ranged from 0% to 25% indicating very poor rock quality. Vertical and subvertical joints were observed in the rock cores which contributed to the relatively low RQD values. In Borehole EC2 a clay seam was encountered in the rock and a rubble zone was observed in the rock cores retrieved from Borehole EC5A.

5.11 Water Levels

A standpipe piezometer was installed in Boreholes EC-2, EC-4 and EC-5. Artesian conditions were observed in Boreholes EC-4 and EC-5 where piezometers were instrumented in the deeper deposits of sands and silts. The water level readings measured on separate visits made after the completion of drilling are presented in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
EC-2	April 18, 2005	4.0	249.3
	May 17, 2005	3.9	249.4
EC-4*	March 31, 2005	2.0	248.1
EC-5*	April, 18, 2005	0.4	247.3
	May 17, 2005	1.1	248.0

* Artesian Condition

Based on these observations, the local groundwater level generally follows the contours of the land and is about 249.4 m at the west limit of the site falling to just below ground surface in the flood plain and rising to about 246.2 m at the east limit. Artesian conditions can be expected in the lower granular deposits below the floodplain of the creek.

Perched water can also be expected to occur in the surficial permeable layers of sands, gravel and silts that are underlain by relatively impermeable layers of clayey silt and clayey silt till.

All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events. The groundwater level will also be controlled by the water level in the creek.

LICENSED PROFESSIONAL ENGINEER
R.A. Abdul
R.A. ABDUL
Feb 12, 2007
PROVINCE OF ONTARIO

Rehman Abdul

Prepared by:
R. Abdul, P.Eng.,
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
Michael Tanos
MICHAEL TANOS
13.2.2007
PROVINCE OF ONTARIO

Michael Tanos

Report Reviewed by:
Michael Tanos, P.Eng.,
Review Principal

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approaches for the proposed structure.

The proposed Highway 410 will cross over Etobicoke Creek via a two span structure approximately 28.9 m wide and spanning 45 m between abutments and pier. The proposed finished grades at the structure will be about Elev. 252.1± m and Elev. 253.3± m at the east and west abutments respectively.

At the east abutment the approach fill will be about 6.5± m high and the forward slopes adjacent to the bridge abutment will be retained by concrete toe walls.

Based on the vertical alignment of the highway and the profile grade at the west approach, an embankment is not required at this location. However, approximately 3± m of cut will be required along the NBL at the south limits of this approach to achieve the desired approach grade. Minor slope flattening/regrading of the existing slopes is also proposed.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the investigations.

7 STRUCTURE FOUNDATIONS

The proposed bridge is a two span structure with two abutments and a pier as foundation elements.

The stratigraphy encountered at the abutment and pier locations consist of 28 m to 33 m of overburden soils overlying bedrock. The groundwater level at the west abutment location lies at Elev. 249.4 m at the west approach falling to just below ground surface in the floodplain and rising to about Elev. 246.2 at the east approach. Artesian conditions were encountered in the lower granular deposits in boreholes drilled at the pier and east abutment locations.

Consideration was given to the following foundation types:

- Spread footings
- Augered Caissons (drilled shafts)
- Driven piles

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix E.

7.1 Spread Footings

Based on the subsurface stratigraphy encountered at this site and the vertical alignment of the highway, this option is feasible for supporting both bridge abutments.

A spread footing is not considered to be feasible for supporting the pier. Based on a H.W.L of Elev. 247.9 m a spread footing is likely to be submerged during storm events and the geotechnical resistance of the founding soils will be relatively low under these submerged conditions. Moreover, there is a discontinuous silty clay layer (Borehole 123A) at this location that will result in differential settlement of the footing under imposed loads.

The recommended founding depths and geotechnical resistances for abutment footings (minimum footing width of 2 m) founded on undisturbed competent natural soils are tabulated below.

Table 7.1 – Geotechnical Resistances at Abutment Locations

Borehole Location	Existing Ground Surface Elev. (m)	Recommended Bottom of Footing Level Below Existing Ground Surface (m)	Founding Elevation (m)	Factored Geotech. Resistance at U.L.S (kPa)	Geotech. Resistance at S.L.S (kPa)	Subgrade Material
EC2 West Abutment	253.3	2.3 – 2.9 2.9 – 4.6	251.0–250.4 250.4–248.7	500* 500*	*325 *325	Clayey Silt Till Sand & Silt Till
EC5 East Abutment	246.9	1.5	245.4	325**	225**	Clayey Silt Till

* Assumes that the distance between the footing edge and the face of the slope is not less than twice the footing width.

** Designed for submerged conditions during storm events.

These values are for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4.

The SLS value quoted above corresponds to a settlement of up to 25 mm, a significant portion of which will be complete by the end of construction.

Resistance to lateral forces/sliding resistance between the concrete footing and very stiff clayey silt till should be evaluated in accordance with the CHBDC, 2000. Assume an ultimate coefficient of friction of 0.5 for the very stiff clayey silt till and 0.7 for the dense to very dense sand and silt till.

7.2 Augered Caissons (Drilled Shafts)

Augered caisson foundations were also considered for supporting the structure. However, the overburden is not considered to be suitable for this scheme and the caissons must be founded on the bedrock at depths in the order of 28 to 33 m below original ground surface. The base of the caissons would be about 27 to 34 m below the groundwater level, resulting in high hydrostatic heads at the base.

The permeable nature of the overburden soil and the presence of boulders above the bedrock at the east abutment location would make it difficult to seal the bottom of the liner into the founding stratum to exclude groundwater. Unwatering of the caisson would be impractical and attempts to do so might result in continued flow of fines into the excavation.

Installation of deep caissons to bedrock is also expected to be a more expensive option than driven piles.

For these reasons, the use of a caisson foundation is not recommended.

7.3 Driven Piles

The subsurface conditions at the site are considered suitable for the design of foundations supported on steel H-piles.

At the west abutment and pier locations steel H-piles are likely to be driven to bedrock. However, at the east abutment location piles are expected to encounter effective refusal in the lower sand and silt till overlying the layer of cobbles and boulders.

Two steel pile sections believed to be currently available have been considered for use in the proposed foundations.

7.3.1 Axial Resistance

The factored, vertical, concentric, geotechnical resistances at ULS and SLS for these pile sections when driven to bedrock, and their approximate tip elevations are presented in Table 7.2.

Table 7.2 – Axial Resistance of Various Pile Sections Driven to Bedrock

Pile Type	Reference Borehole	Support Location	Estimated Pile Tip Elevation (m)	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
HP 310 x 110	EC-2	West Abutment	222.6±	1700	1200
	EC-4	Pier	218.2±	1700	1200
HP 360 x 132	EC-2	West Abutment	222.6±	2300	1600
	EC-4	Pier	218.2±	2300	1600

Piles driven at the east abutment location and encountering effective refusal in the very dense lower sand and silt till should be designed on the basis of the concentric, axial geotechnical resistances given in Table 7.3.

Table 7.3 – Axial Resistance of Various Pile Sections (East Abutment)

Pile Type	Piles Driven Into Sand and Silt Till			
	Reference Borehole	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)	Estimated Pile Tip Elevation (m)
HP 310 x 110	EC-5/EC-5A	1500	1050	220.0±
HP 360 x 132	EC-5/EC-5A	2000	1400	219.0±

The structural resistance of the pile should be checked by the structural designer.

The H-piles for the recommended foundation scheme will be driven either to bedrock at the west abutment and pier, or to effective refusal at the east abutment. Piles will penetrate through till layers that are likely to contain cobbles and boulders. It is therefore recommended that the pile tips be fitted with driving shoes to provide reinforcement to the pile section and effective contact where the piles will reach bedrock.

The contract documents should contain a NSSP alerting the contractor to the fact that cobbles and boulders may be encountered in the soil. The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving. If the required resistance according to the Hiley Formula is not achieved and further driving is likely to cause damage to the pile, pile driving should cease and the contract administrator should be notified. Suggested wording for the NSSP is included in Appendix G.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which the piles will be driven.

7.3.2 Downdrag

Downdrag on the piles is not considered to be an issue at this site.

7.3.3 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. At the west abutment location the upper 3 m of pile will lie in the native compact to very dense sand and silt till. At the east abutment location the upper 3 m of pile will lie in the very stiff clayey silt till and the compact to very dense silty sand till. Therefore, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is driven, the space between the pile and the CSP should be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 7.4.

Table 7.4 – Integral Abutment Sand Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 μm	#30	80%-100%
425 μm	#40	40%-80%
250 μm	#60	5%-25%
150 μm	#100	0%-6%

7.3.4 Lateral Resistance

The lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$k_s = n_h \cdot z / D$ [cohesionless soils] (kN/m^3)

$k_s = 67\ S_u/D$ [cohesive soils] (kN/m^3)

$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p$ [cohesionless soils] (kPa)

$p_{ult} = 9\ S_u$ [cohesive soils] (kPa)

where
z = depth of embedment of pile (m)
D = pile width (m)
S_u = undrained shear strength (Table 7.5) (kPa)
n_h = coefficient of horizontal subgrade reaction (Table 7.5) (kN/m^3)
γ = unit weight (Table 7.5) (kN/m^3)
K_p = passive earth pressure coefficient

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance or the factored structural flexural resistance of the pile. For design purposes a maximum horizontal passive resistance of 120 kN (ULS) is recommended.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$.

Table 7.5 – Recommended Soil Parameters

Area Reference Borehole No	Applicable Elevation	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m^3)*
West Abutment EC-2	253.1 – 252.6	Silty Sand	18	25	-	2000
	252.6 – 251.9	Clayey Silt	19	0	40	-
	251.9 – 251.2	Sand & Gravel	19	35	-	6000
	251.2 – 250.4	Clayey Silt Till	20	0	150	-
	250.4 – 243.2	Sand & Silt Till	20	35	-	8500
	243.2 – 235.5	Silt	19	35	-	8500
	235.5 – 231.0	Silty Sand	19	35	-	6000
	231.0 – 229.4	Sandy Silt	19	35	-	8500
	229.4 – 224.9	Sand & Silt Till	20	35	-	8500
	224.9 – 222.6	Clayey Silt Till	20	0	200	-
East Abutment EC-5 & EC-5A	246.7 – 245.5	Silty Sand	18	25	-	5000
	245.5 – 244.8	Clayey Silt Till	20	0	150	-
	244.8 – 238.3	Silty Sand Till	20	35	-	6000
	238.3 – 233.7	Clayey Silt	20	0	200	-
	233.7 – 230.7	Silty Sand	19	25	-	4000
	230.7 – 224.6	Sandy Silt	19	25	-	4000
	224.6 – 216.4	Sand & Silt Till	20	35	-	8500

* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3rd edition, 1992

Since the piles are end bearing, the vertical resistance will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the equation for k_s quoted above may be used in conjunction with appropriate reduction factors.

Where a pile group is oriented ***perpendicular*** to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented ***parallel*** to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

For conventional abutments, the lateral resistance may be provided by battered piles.

7.3.5 Pile Tips

Due to the possible presence of cobbles and boulders in the till layers; the tips of all piles should be fitted with driving shoes to protect the pile tip from damage and also to provide effective contact with the bedrock surface.

7.3.6 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

The Contract Documents should contain a NSSP alerting the Contractor to the presence of cobbles and boulders in the till soils.

7.3.7 Pile Driving

At the east abutment, pile driving should be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile".

"R" must have the minimum values shown in Table 7.6.

Table 7.6 – Ultimate geotechnical Resistance of Piles

Pile	Ultimate Resistance (R) (kN)
HP 310X110	3,100 kN
HP 360X132	4,200 kN

Hiley formula calculations need not be carried out until the pile has been driven below Elev. 221 m.

For practical refusal on bedrock, a refusal criterion of 5 blows for 6 mm for three consecutive sets is recommended. Alternatively, 16 blows for 20 mm or 20 blows for 25 mm penetration can also be used. These values are based on a typical hammer energy of 60 kilojoules/blow, with an energy transfer (efficiency) of 40%.

7.4 Recommended Foundation

The use of H-piles at the abutments allows for the design of an integral abutment structure. From a geotechnical point of view, it is recommended that all foundations for the main bridge structure be supported on steel H-piles.

7.5 Pile Cap Drainage Scheme

Artesian conditions were encountered in some boreholes across the site and upward migration of silt may occur. Therefore, mitigating measures such as a filtered pile cap drainage scheme (Figure F1, Appendix F) is recommended at all foundation elements.

The recommended construction sequence for this pile cap drainage scheme at the bridge abutments is outlined below:

- Excavate to underside of abutments.
- Preauger and install 600 mm diameter by 3000 mm long CSP, install piles and sand and cut off piles to desired elevation.
- Lay geotextile on prepared and approved subgrade, place 100 mm thick 19 mm clear stone (OPSS 1004) and subdrain. Ensure positive drainage between subdrain and its outlet.
- Proceed with normal integral abutment construction.

The recommended construction sequence for the pile cap drainage scheme at the bridge pier is outlined below:

- Excavate as required, prepare site and drive piles.
- Lay geotextile on prepared and approved subgrade and place 19 mm clear stone (OPSS 1004) to underside of pile cap.
- Form and pour pile cap.

- Continue with placement of 19 mm clear stone (OPSS 1004) around perimeter of pile cap and install subdrain arrangement. Ensure positive drainage between subdrain and its outlet.
- Proceed with normal bridge construction.

7.6 Frost Cover

Pile caps and footings should be provided with a minimum of 1.2 m of earth cover over the footing base (founding elevation).

8 RETAINING WALLS

The forward slopes of the east approach fills will be retained by concrete toe walls. These retaining walls can be supported on spread footings founded on the very stiff clayey silt till at Elev. 245.5 m. Provided a minimum footing width of 2 m is maintained, a footing founded on the very stiff clayey silt till may be designed for the following values:

- Factored geotechnical resistance of 325 kPa at Ultimate Limit State (ULS)
- Geotechnical resistance of 225 kPa at Serviceability Limit State (SLS)

Alternatively, concrete (Type II) gravity toe walls can be used if preferred, in which case the walls will have to be supported on an engineered fill pad placed on the very stiff clayey silt till encountered at Elev. 245.5m.

All overburden materials including topsoil and the surficial silty sand layer should be removed below the footprint of the engineered fill pad. The engineered fill should consist of OPSS Granular A compacted to 100% of its Standard Proctor maximum dry density (SPMDD) at ±2% of optimum moisture content (OPSS 501, Section 501.08.02, Method A) and conforming to the geometry illustrated in Figure F2 in Appendix F. It is recommended that the thickness of the fill pad be equal to or greater than the footing width.

Type II toe walls (OPSD 4066.01) founded on a compacted Granular A pad may be designed for geotechnical resistances of 300 kPa at Ultimate Limit State and 200 kPa at Serviceability Limit State.

These values are for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4.

The SLS values quoted above corresponds to a settlement of up to 25 mm, a significant portion of which will be complete by the end of construction.

Resistance to lateral forces/sliding resistance between the concrete footing and its subgrade should be evaluated in accordance with the CHBDC, 2000. Assume an ultimate coefficient of friction of 0.5 for the very stiff clayey silt till and 0.7 for the compacted Granular A.

9 EXCAVATION AND BACKFILL

9.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site may be classified as Type 3 soils above the water table and Type 4 soils below the water table. Excavation below the groundwater level is not recommended without prior dewatering. Provided dewatering is carried out as described below, excavations may be sloped at 2H:1V

9.2 Foundations

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01.

10 GROUNDWATER CONTROL

The groundwater level is at or just below ground surface in the flood plain. Beyond the flood plain near the east and west limits the groundwater level is estimated to range between 1.4 m and 3.9 m below ground surface. It should be noted that artesian conditions were encountered in the deeper layers of sands and silts encountered below the flood plain of the creek.

The groundwater level should be temporarily lowered prior to excavation if any excavation is required that will penetrate below the groundwater level.

The design of the unwatering system should be the responsibility of the Contractor. However, suitable systems that might be employed include pumping from filtered sumps for nominal penetration below the groundwater level and the use of vacuum wellpoints for deeper excavations below the groundwater level.

Any accumulation of water from the base of the excavation should be removed prior to placing concrete or compacting granular fill. Placement of concrete or compacting engineered fill must be done in the dry.

11 APPROACH EMBANKMENTS

At the east approach embankment construction using either non-cohesive earth fill or rock fill is feasible provided the embankment is constructed on the underlying clayey silt glacial till encountered at this site. Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles will be driven.

Settlement in the order of 100 mm will occur under the loading imposed by the 6.5± m of approach fill but due to the non-plastic nature of most of the foundation soils, most of the settlement will be immediate and essentially complete when construction of the fill is completed.

The embankments will also experience settlement resulting from consolidation of the fill. This settlement is expected to be about 65 mm for a 6.5± m high embankment. The settlement within the non-cohesive fill should be immediate in nature and essentially be complete shortly after construction has been completed.

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry and also to a large degree on the material used to construct the embankment. If the embankment is constructed of blast rock fill, it may be assumed that the side slopes will be stable at inclinations up to 1.25H:1V. Embankments constructed using granular material, select subgrade material or non-cohesive earth fill will have stable side slopes at inclinations of up to 2H:1V.

For the purpose of embankment stability analyses, the commercially available slope stability program Slope W developed by Geo-Slope International Ltd. was used. The Bishop's simplified method for stability analysis was employed.

Global stability analyses were conducted for 2H:1V SSM or earth fill embankments and for 1.25H:1V rock fill embankments. The stability of the embankment was also assessed for storm events and rapid drawdown conditions using a static H.W.L of Elev. 247.9 m.

Consideration was initially given to constructing the approach embankments on the compact silty sand layer but factors of safety less than 1.3 were obtained under rapid drawdown conditions during storm events.

For an approach embankment constructed on the native very stiff clayey silt till, factors of safety against global failure greater than 1.4 were obtained during storm events.

Therefore, it is recommended that the topsoil and the surficial layer of silty sand be stripped below the footprint of the embankment prior to constructing the approach fills. Complete removal of the underlying silty sand layer encountered between Elev. 246.7 m and Elev. 245.5 m is essential in order to maintain embankment stability during storm events.

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002.

The approach fills should be constructed in advance of pile driving operations.

Earth fill embankment slopes and cut slopes must be provided with erosion protection in accordance with OPSS 572.

Cut slopes approximately 3± m high along the NBL of the west approach will be stable provided that the cut slope geometry is not steeper than 2H:1V.

At the west approach where minor regrading/slope flattening is required, bonding between the existing soils and the new fill can be achieved as per OPSD 208.010.

12 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used at the east approach subject to the requirements presented in this section. A conventional concrete abutment will be required for the contemplated design but RSS could be used for wing walls and other retaining structures. RSS walls may be considered as a replacement to the concrete toe walls in which case the RSS will be founded on an engineered fill core resting on native clayey silt till.

RSS walls should be specified to be "High Performance" and "High Appearance". The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

12.1 Foundation

The performance of an RSS is dependent, among other factors, on the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system.

At this site, it is recommended that the levelling pad for the RSS wall be centred on top of a pad of engineered fill consisting of OPSS Granular A compacted in accordance with the OPSS 501, Method A. The engineered fill pad should be at least 500 mm thick and should be at least twice as wide as the levelling pad.

In addition to the requirements for the levelling pad, the RSS can be founded on well compacted approach fill or on the native very stiff clayey silt till. All founding subgrades should be inspected and evaluated by the Quality Verification Engineer (QVE), at the time of construction.

The following parameters may be used for the design of the RSS:

- Bearing resistance for the levelling pad on engineered fill:
    - Factored ULS 300 kPa
    - SLS 200 kPa
- Ultimate coefficient of sliding resistance of cast in-situ concrete levelling pad on Granular A = 0.70
- Ultimate coefficient of sliding resistance of RSS mass on Granular A = 0.70
- Ultimate coefficient of sliding resistance of RSS mass on native clayey silt till = 0.5

All topsoil, organics and soft soils should be removed from the subgrade of the RSS wall prior to placement of the fill.

The entire block of reinforced earth must also be designed against various modes of failure including sliding and overturning.

The settlement of a wall founded on an engineered fill pad will depend on the thickness of the pad, the material used, the foundation soils and the quality of construction. However, settlements are expected to be less than 25 mm and to occur essentially as the RSS is constructed.

12.2 Global Stability

The global stability of the RSS wall is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment.

Stability analyses of RSS toe walls were carried out considering the following variables:

- RSS founded at the base of the embankment on native clayey silt till.
- Embankment fill behind the RSS is inclined at 2H:1V.
- High Water Level at Elev. 247.9 m and rapid drawdown conditions.

Analysis carried out on RSS walls located at the base of the embankment indicates that an anchor length equivalent to 75% of the wall height is required in order to obtain a factor of safety of 1.3 during storm events.

If the RSS wall has to be founded in the embankment slope, the specific geometry and soil conditions must be analyzed to determine the requirements for global stability. The actual design configuration must be checked for global stability prior to finalization.

The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

13 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material. In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments no greater than 300 mm and should include adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular B Type II.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3501.000, and rock backfill should be placed to the extents shown in OPSD 3505.000.

All granular material should meet the specifications of Special Provision 110F13 “Amendment to OPSS 1010, March 1993”.

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SSP 105S10.

The design of the abutment should incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

14 EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3501.000 or OPSD 3505.000, as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill and 1.25H:1V for rock backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

$P_h = K(\gamma h + q)$
P_h = horizontal pressure on the wall (kPa)
K = earth pressure coefficient (see table 13.1)
γ = unit weight of retained soil (see table 13.1)
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 14.1.

Table 14.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°; \gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°; \gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42°; \gamma = 19.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	5.0	-

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral or semi-integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass. However, the use of Granular B Type I may be restricted if the approach embankment consists of rock fill.

The factors in the table above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 2000.

15 EROSION PROTECTION

During storm events and assuming a H.W.L of Elev. 247.9 m the east approach embankment, concrete toe walls and the bridge abutment will be partially submerged. Therefore, rock fill would be the ideal material to use as fill to achieve the desired grade.

Alternatively, consideration can be given to using granular fill provided that rock protection (rip-rap) is used to armour the slopes and other areas that are susceptible to erosion. If this approach is adopted then it must be recognized that surface water can cause erosion beneath the rip-rap. Furthermore, during storm events movement of fines through the rip-rap blanket will occur. Therefore, a properly designed granular or fabric filter blanket would be required.

16 SEISMIC CONSIDERATIONS

16.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 0. The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.08

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

16.2 Liquefaction Potential

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method[1]. Using this method, it is estimated that there is negligible potential for soil liquefaction of the foundation soils below the abutments and pier.

Furthermore, since the foundation loads will be transferred by steel piles to bedrock or to the very dense sand and silt till, the vertical geotechnical resistance of the piles will not be compromised.

The immediate approach embankment will be constructed above the groundwater level and therefore there is negligible potential for soil liquefaction below the embankment. Some toe failure may occur but is expected to be limited and readily repairable.

16.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

In calculating the active, passive and at rest earth pressure coefficients the angle of friction between the wall and backfill material is assumed to be 0.5 ϕ. For the design of retaining walls, the coefficients of horizontal earth pressure in Table 16.1 may be used:

Table 16.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
	Granular A or Granular B Type II $\phi = 35°$; $\delta = 17.5°$ $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\delta = 16°$ $\gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42°$; $\delta = 21°$ $\gamma = 19.0$ kN/m^3	
Wall Condition	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.28	0.45	0.31	0.55	0.21	0.30
Passive (K_{PE})	3.69	-	3.26	-	5.05	-
At Rest (K_{OE})**	0.53	-	0.58	-	0.44	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods

[1] Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 101, No. SM9, September, pp. 1249-1273.

17 CONSTRUCTION CONCERNS

During construction, the Contract Administrator should employ experienced geotechnical staff to observe construction activities related to foundation construction.

Potential construction concerns include, but are not necessarily limited to:

- the possibility of piles encountering boulders.
- the nature and geotechnical properties of non-cohesive earth fill used in thè approach fills.
- unwatering.
- Pollution, siltation or disruption of environmentally sensitive areas.

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
PROVINCE OF ONTARIO

Engineering Analysis and Report Preparation by:
R. Abdul, P.Eng.,
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
MICHAEL TANOS
13·2·2007
PROVINCE OF ONTARIO

Report Reviewed by:
Michael Tanos, P.Eng.,
Review Principal

APPENDICES

TERRAPROBE LIMITED

LIMITATIONS AND RISK

Procedures

The soil conditions were confirmed at the borehole locations only and conditions may vary between and beyond the boreholes. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of stratigraphic change.

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities.

Changes In Site And Scope

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The design advice is based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, or there is any additional information relevant to the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructibility issues and quality control may not be relevant or complete for the revised project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report

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EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	– °	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	– °	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $(w_L - w_P)$
I_L	1	LIQUIDITY INDEX = $(w - w_P)/I_P$
I_C	1	CONSISTENCY INDEX = $(w_L - w)/I_P$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

APPENDIX A

Record of Borehole Sheets

TERRAPROBE LIMITED

RECORD OF BOREHOLE No EC-1

METRIC

W.P. 107-00-01 LOCATION Coords: N:4845155.1 E:276749.2 (Etobicoke Creek) ORIGINATED BY MS

DIST HWY 410 Phase III BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 05.04.05 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
253.0	Ground Surface						
0.0	**200mm TOPSOIL**						
0.2	**CLAYEY SILT** - some sand, trace gravel, trace rootlets and wood pieces, damp, firm, brown	1	SS	4			
252.3 / 0.7	**SILTY SAND** trace gravel, damp, loose, brown	2	SS	5		252	
		3	SS	7			
250.9 / 2.1	**SAND AND SILT** trace gravel, trace clay, moist to wet, dense to very dense, brown (GLACIAL TILL)	4	SS	51		251	
		5	SS	50/ 15cm		250	
		6	SS	57		249	9 41 44 6
	grey	7	SS	42	▽*	248	
	compact	8	SS	29		247	
		9	SS	36			
246.4 / 6.6	End of Borehole						

*Water level at 5.0m (unstabilized) and hole open to 5.3m on completion of drilling.

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

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No EC-2

METRIC

W.P. 107-00-01 LOCATION Coords: N:4845168.1 E:278764.5 (Etobicoke Creek) ORIGINATED BY MS

DIST HWY 410 Phase III BOREHOLE TYPE Solid Stem Augers, Hollow Stem Augers, Casing and NQ Coring COMPILED BY DB

DATUM Geodetic DATE 05.04.05 - 07.04.05 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
253.3	Ground Surface					
253.0 / 0.2	150mm TOPSOIL - Sandy, dark brown					
	SILTY SAND - trace gravel, trace rootlets and wood pieces, moist, very loose, brown	1	SS	2	253	
252.6 / 0.7	CLAYEY SILT trace gravel, trace sand, moist, firm, brown	2	SS	8	252	
251.9 / 1.4	SAND AND GRAVEL trace silt, trace clay, damp, compact, brown	3	SS	30*		36 47 9 8
251.2 / 2.1	CLAYEY SILT - Sandy, some gravel, moist, very stiff, brown (GLACIAL TILL)	4	SS	26	251	23 41 25 11
250.4 / 2.9	SAND AND SILT trace gravel, trace clay, damp to moist, dense to very dense, brown (GLACIAL TILL)	5	SS	34	250	
		6	SS	77	249	
		7	SS	32	248	
		8	SS	45		
		9	SS	30	247	
	---- compact	10	SS	17	246	
	---- grey	11	SS	74	245 / 244	
243.2 / 10.1	SILT trace sand, trace clay, moist to wet, dense to very dense, grey	12	SS	57	243 / 242	0 0 91 9
						April05,2005 / April06,2005
		13	SS	35	241 / 240	
		14	SS	31	239	

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

Continued Next Page

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

RECORD OF BOREHOLE No EC-2

METRIC

W.P. 107-00-01 | LOCATION Coords: N:4845168.1 E:278764.5 (Etobicoke Creek) | ORIGINATED BY MS

DIST ___ HWY 410 Phase III | BOREHOLE TYPE Solid Stem Augers, Hollow Stem Augers, Casing and NQ Coring | COMPILED BY DB

DATUM Geodetic | DATE 05.04.05 - 07.04.05 | CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT trace sand, trace clay, moist to wet, compact to dense, grey *(continued)*	15	SS	22	238	0 2 93 5
					237	
		16	SS	45	236	
235.5 / 17.8	**SILTY SAND** trace clay, wet, compact to dense, grey	17	SS	45	235	0 86 10 4
					234	
		18	SS	18	233	
		19	SS	37	232	
231.0 / 22.3	**SANDY SILT** trace gravel, trace clay, wet, very dense, reddish brown to grey				231	
		20	SS	51	230	April06,2005 / April07,2005
229.4 / 23.9	**SAND AND SILT** some gravel ,trace clay, moist, compact to very dense, reddish brown (GLACIAL TILL)	21	SS	86	229	
					228	
		22	SS	29	227	17 38 37 8
		23	SS	30	226	
224.9 / 28.4	**CLAYEY SILT** trace gravel, damp, hard, grey (GLACIAL TILL)				225	
		24	SS	120/ 8cm	224	

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

Continued Next Page

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

RECORD OF BOREHOLE No EC-2

METRIC

W.P. 107-00-01 LOCATION Coords: N:4845168.1 E:278764.5 (Etobicoke Creek) ORIGINATED BY MS

DIST HWY 410 Phase III BOREHOLE TYPE Solid Stem Augers, Hollow Stem Augers, Casing and NQ Coring COMPILED BY DB

DATUM Geodetic DATE 05.04.05 - 07.04.05 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS
					223		
222.6		25	SS	119/10cm		~10	
30.7	**SHALE BEDROCK** moderately to highly weathered, thinly to medium bedded, grey, very weak to weak with slightly weathered, medium strong to strong fossiliferous limestone interbeds. vertical to subvertical joints at 32.3m and 32.9m. Clay seam from 31.5m to 31.7m. (Georgian Bay Formation)	1	RUN	NQ	222		RUN#1 TCR=63% SCR=37% RQD=10%
		2	RUN	NQ	221 / 220		RUN#2 TCR=85% SCR=53% RQD=20%
219.5 33.8	End of Borehole						

*Auger refusal at 1.8m, probably due to boulders. Borehole moved 0.8m South and redrilled.

Wet cave at 21.3m upon completion.

Piezometer Installation consists of 19mm diameter, schedule 40 PVC pipe with a 1.52m slotted screen.

Water Level Readings:

Date	Depth(m)	Elevation(m)
Apr.18.05	4.0	249.3
May.17.05	3.9	249.4

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No EC-4

METRIC

W.P. 107-00-01	LOCATION Coords: N:4845183.0 E:278807.6 (Etobicoke Creek)		ORIGINATED BY MS
DIST	HWY 410 Phase III	BOREHOLE TYPE Hollow Stem Augers, Casing and NQ Coring	COMPILED BY DB
DATUM Geodetic	DATE 29.03.05 - 30.03.05		CHECKED BY RA

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
246.1	Ground Surface
245.9 / 0.2	**200mm TOPSOIL** - Sandy, dark brown
	SILTY SAND trace rootlets and wood pieces, moist to wet, brown; very loose / compact
244.0 / 2.1	**SAND AND SILT** trace gravel, trace clay, moist to wet, compact, grey (GLACIAL TILL)
242.4 / 3.7	**SILT** trace sand, trace clay, wet, grey; dense to very dense / compact; becoming sandy
232.9 / 13.2	**SILTY SAND** trace clay, wet, loose, grey

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	2	
2	SS	3	0 61 32 7
3	SS	17	
4	SS	22	
5	SS	22	
6	SS	43	
7	SS	52	0 0 90 10
8	SS	38	
9	SS	29	
10	SS	24	0 1 90 9
11	SS	26	
12	SS	10	
13	SS	14	
14	SS	7	

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (246 to 232); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No EC-4

METRIC

W.P. 107-00-01 LOCATION Coords: N:4845183.0 E:278807.6 (Etobicoke Creek) ORIGINATED BY MS

DIST HWY 410 Phase III BOREHOLE TYPE Hollow Stem Augers, Casing and NQ Coring COMPILED BY DB

DATUM Geodetic DATE 29.03.05 - 30.03.05 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND trace clay, wet, compact, grey *(continued)*	15	SS	13	
229.9 16.2	**SANDY SILT** trace clay, wet, compact, reddish brown to grey	16	SS	15	0 14 80 6
		17	SS	14	
226.8 19.3	**SAND AND SILT** trace gravel, trace clay, wet, loose to compact, reddish brown (GLACIAL TILL)	18	SS	12	
		19	SS	7	8 40 46 6
		20	SS	7	
		21	SS	24	
		22	SS	21	
219.2 26.9	**CLAYEY SILT** sandy, some gravel, wet, hard, grey (GLACIAL TILL)	23	SS	100/ 15cm	27 34 28 11
218.2 27.9	**SHALE BEDROCK** slightly weathered, thin to medium bedded, grey, weak to medium strong, with slightly weathered, thinly bedded, medium strong to strong fossiliferous limestone interbeds. vertical joints at 28.4m, 29.3, 24.7 and 30.0m. (Georgian Bay Formation)	1	RUN	NQ	RUN#1 TCR=86% SCR=71% RQD=7%

Elevation scale: 231, 230, 229, 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No EC-4

METRIC

W.P. 107-00-01 LOCATION Coords: N:4845183.0 E:278807.6 (Etobicoke Creek) ORIGINATED BY MS

DIST HWY 410 Phase III BOREHOLE TYPE Hollow Stem Augers, Casing and NQ Coring COMPILED BY DB

DATUM Geodetic DATE 29.03.05 - 30.03.05 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
215.2	**SHALE BEDROCK**		2	RUN	NQ		216				RUN#2 TCR=83% SCR=58% RQD=0%
30.9	End of Borehole Water level above ground surface (artesian condition) upon completion. Piezometer Installation consists of 19mm diameter, schedule 40 PVC pipe with a 1.52m slotted screen. Water Level Readings: Date Height(m) Elevation(m) Mar.31.05 2.0 248.1										

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No EC-5

METRIC

W.P. 107-00-01 | LOCATION Coords: N:4845223.0 E:278835.8 (Etobicoke Creek) | ORIGINATED BY MS

DIST ____ HWY 410 Phase III | BOREHOLE TYPE Hollow Stem Augers, D.C.P.T. | COMPILED BY DB

DATUM Geodetic | DATE 24.03.05 - 28.03.05 | CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
246.9	Ground Surface							
246.0 / 0.0	200mm TOPSOIL - black	1	SS	16				
0.2	SILTY SAND - topsoil stained, trace rootlets, damp to moist, compact, brown	2	SS	11				
245.5 / 1.4	CLAYEY SILT - sandy, some gravel, moist, very stiff, grey/brown (GLACIAL TILL)	3	SS	25	13	28	38	21
244.8 / 2.1	SILTY SAND trace gravel, trace clay, moist to wet, grey, (GLACIAL TILL) compact	4	SS	19				
	very dense	5	SS	62	3	79	11	7
		6	SS	100/13cm				
	compact to dense	7	SS	18				
		8	SS	30				
		9	SS	19				
		10	SS	40				
238.3 / 8.6	CLAYEY SILT trace sand damp, hard, grey	11	SS	106	0	1	81	18
		12	SS	121				
		13	SS	84				
233.7 / 13.2	SILTY SAND trace clay, wet, compact, grey	14	SS	11	0	60	33	7

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (246 to 232); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

March 24, 2005
March 28, 2005

Continued Next Page

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

ONTARIO MOT 1-00-0350 HWY 410 ETOBICOKE CREEK.GPJ ONTARIO MOT.GDT 28/07/05

RECORD OF BOREHOLE No EC-5

METRIC

W.P. 107-00-01	LOCATION Coords: N:4845223.0 E:278835.8 (Etobicoke Creek)	ORIGINATED BY MS
DIST ___ HWY 410 Phase III	BOREHOLE TYPE Hollow Stem Augers, D.C.P.T.	COMPILED BY DB
DATUM Geodetic	DATE 24.03.05 - 28.03.05	CHECKED BY RA

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND trace clay, wet, compact, grey *(continued)*	15	SS	15	231	
230.7 / 16.2	**SANDY SILT** trace to some clay, wet, loose to compact, reddish brown to grey	16	SS	9	230	0 34 56 10
		17	SS	7	229, 228	
		18	SS	14	227	
		19	SS	10	226, 225	0 27 69 4
224.6 / 22.3	**SAND AND SILT** some gravel, trace clay, moist, very dense, reddish brown (GLACIAL TILL)	20	SS	84	224	13 47 36 4
		21	SS	113	223, 222	
		22	SS	113	221, 220	18 45 34 3
219.3 / 27.6		23	SS	100/ 5cm		
	End of Borehole					

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

Auger refusal at 27.6m, probably on cobbles and boulders.

Attempted Dynamic Cone Penetration Test (D.C.P.T.), refusal at 27.6m.

Water level above ground surface (artesian condition) upon completion.

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No EC-5

METRIC

W.P. 107-00-01 LOCATION Coords: N 4845223.0 E 278835.8 (Etobicoke Creek) ORIGINATED BY MS

DIST HWY 410 Phase III BOREHOLE TYPE Hollow Stem Augers, D.C.P.T. COMPILED BY DB

DATUM Geodetic DATE 24.03.05 - 28.03.05 CHECKED BY RA

SOIL PROFILE		SAMPLES						DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	Piezometer Installation consists of 19mm diameter, schedule 40 PVC pipe with a 1.52m slotted screen. Water Level Readings: Date / Height(m) / Elevation(m) Apr.18.05 / 0.4 / 247.3 May17.05 / 1.1 / 248.0										

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No EC-5A

METRIC

W.P. 107-00-01 LOCATION Coords: N:4845221.5 E:278835.8 (Etobicoke Creek) ORIGINATED BY MS

DIST HWY 410 Phase III BOREHOLE TYPE Solid Stem Augers, Hollow Stem Augers, Casing, Washboring, and NQ Coring COMPILED BY DB

DATUM Geodetic DATE 01.04.05 - 04.04.05 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
246.9 / 0.0	Ground Surface										
	Augering to 28.9m, refer to Log EC-5 for inferred stratigraphy.						246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235, 234, 233, 232				

Continued Next Page

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

RECORD OF BOREHOLE No EC-5A

METRIC

W.P. 107-00-01 | LOCATION Coords: N:4845221.5 E:278835.8 (Etobicoke Creek) | ORIGINATED BY MS

DIST ____ HWY 410 Phase III | BOREHOLE TYPE Solid Stem Augers, Hollow Stem Augers, Casing, Washboring, and NQ Coring | COMPILED BY DB

DATUM Geodetic | DATE 01.04.05 - 04.04.05 | CHECKED BY RA

SOIL PROFILE ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Augering to 28.9m, refer to Log EC-5 for inferred stratigraphy.						231 230 229 228 227 226 225 224 223 222 221 220 219				
218.0 28.9	**SAND AND SILT** some gravel, trace clay, moist to wet, very dense, reddish brown - (GLACIAL TILL)		24	SS	100/ 13cm		218 217		○ (approx. 8)		

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No EC-5A

METRIC

W.P. 107-00-01 LOCATION Coords: N:4845221.5 E:278835.8 (Etobicoke Creek) ORIGINATED BY MS

DIST HWY 410 Phase III BOREHOLE TYPE Solid Stem Augers, Hollow Stem Augers, Casing, Washboring, and NQ Coring COMPILED BY DB

DATUM Geodetic DATE 01.04.05 - 04.04.05 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.4 30.5	**COBBLES AND BOULDERS** wet, very dense, grey	25	SS	100/ 5cm		216	
		1	RUN	NQ		215	
		2	RUN	NQ		214	
213.8 33.1	**SHALE BEDROCK** moderately to highly weathered, thin to medium bedded, grey, very weak to weak with slightly weathered, grey, medium strong to strong fossiliferous limestone interbeds. subvertical joints at 33.5m and 33.7m rubble zone at 34.6m. (Georgian Bay Formation)	3	RUN	NQ		213	RUN#3 TCR=43% SCR=24% RQD=25%
		4	RUN	NQ		212 211	RUN#4 TCR=42% SCR=18% RQD=10%
210.8 36.1	End of Borehole Borehole filled with drill water upon completion of coring.						

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

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No EC-6

METRIC

W.P. 107-00-01 | LOCATION Coords: N:4845231.7 E:278854.4 (Etobicoke Creek) | ORIGINATED BY MS

DIST | HWY 410 Phase III | BOREHOLE TYPE Solid Stem Augers | COMPILED BY DB

DATUM Geodetic | DATE 05.04.05 | CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
247.6	Ground Surface					
0.0	**160mm TOPSOIL**					
0.2	**CLAYEY SILT** - Sandy, trace gravel, trace rootlets, moist, brown, soft to 0.7m, very stiff below (GLACIAL TILL)	1	SS	3		
		2	SS	17		2 32 40 26
246.2 / 1.4	**SAND AND SILT** trace gravel, trace clay, moist to wet, compact to dense, grey (GLACIAL TILL)	3	SS	21		
		4	SS	19		
		5	SS	45		9 44 40 7
					▽*	
		6	SS	10		
		7	SS	16		
		8	SS	19		
		9	SS	46		
		10	SS	17		1 30 64 5
239.5 / 6.1	End of Borehole					

*Water level at 3.8m (unstabilized) and hole open to 4.3m on completion of drilling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (247, 246, 245, 244, 243, 242, 241, 240); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

APPENDIX B

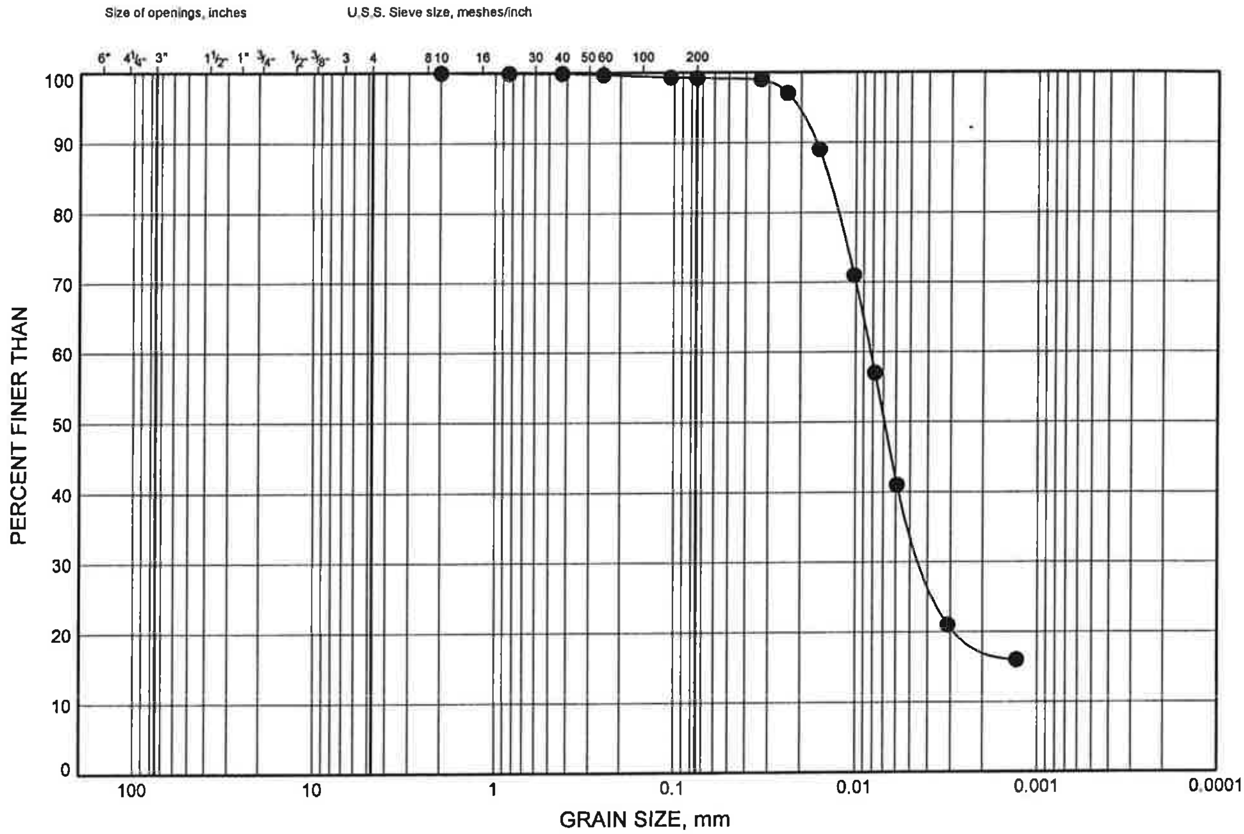
Laboratory Test Results

Terraprobe Limited

GRAIN SIZE DISTRIBUTION

FIGURE B1

Clayey Silt



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-5	9.3	237.6

Date May 2005

Project 107-00-01

Prep'd DB

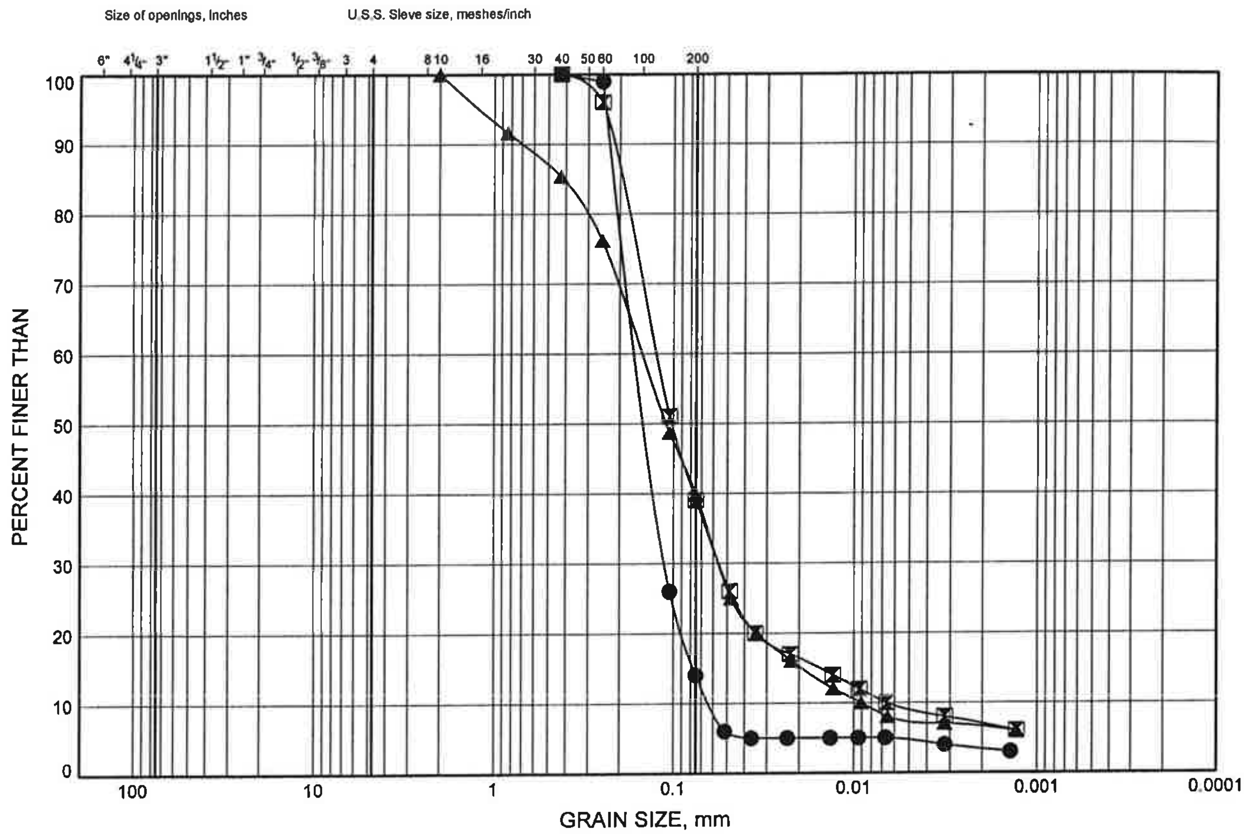
Chkd. RA

GSD 1-00-0350 HWY 410 ETOBICOKE CREEK.GPJ 12/02/07

GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty Sand



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-2	18.5	234.8
⊠	EC-4	1.0	245.1
▲	EC-5	13.9	233.0

Date July 2005
Project 107-00-01

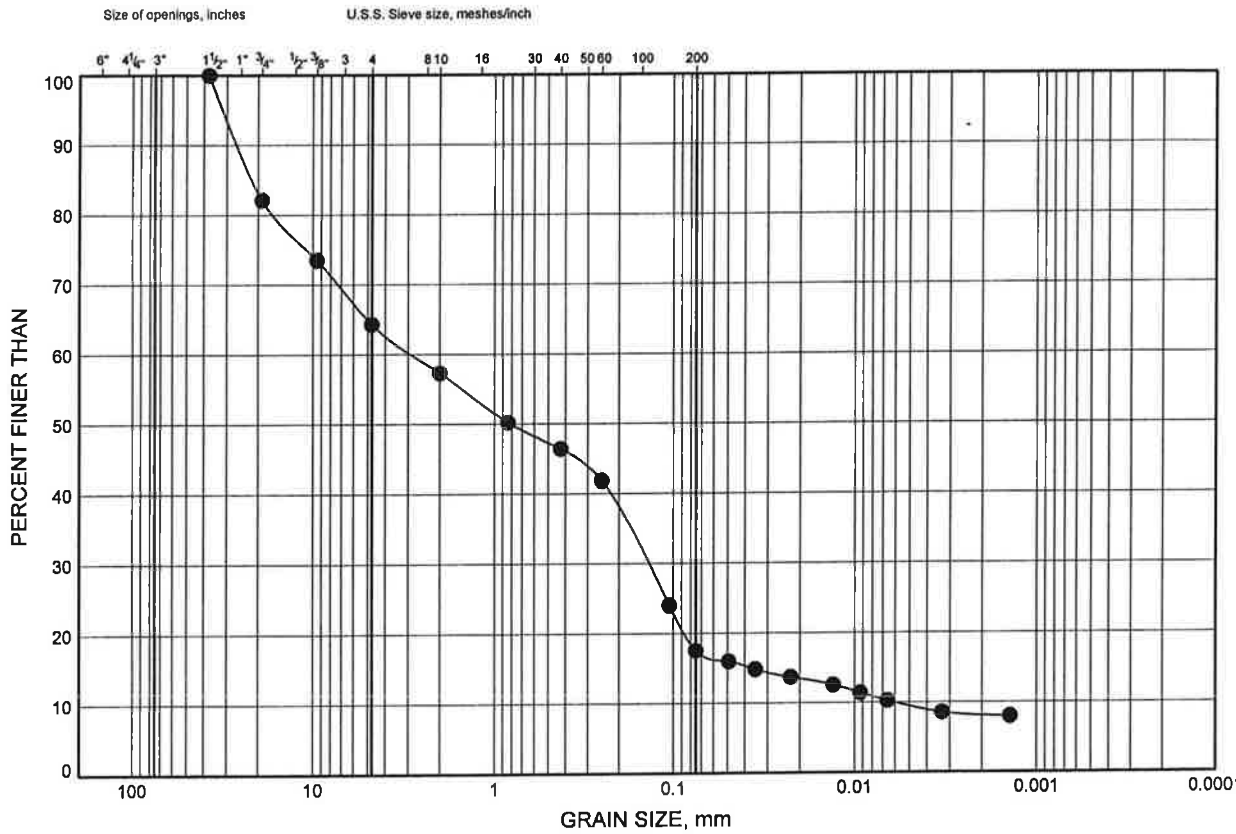
Prep'd DB
Chkd. RA

GSD 1-00-0350 HWY 410 ETOBICOKE CREEK.GPJ 12/02/07

GRAIN SIZE DISTRIBUTION

FIGURE B3

Sand and Gravel



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-2	1.7	251.6

Date May 2005

Project 107-00-01

Prep'd DB

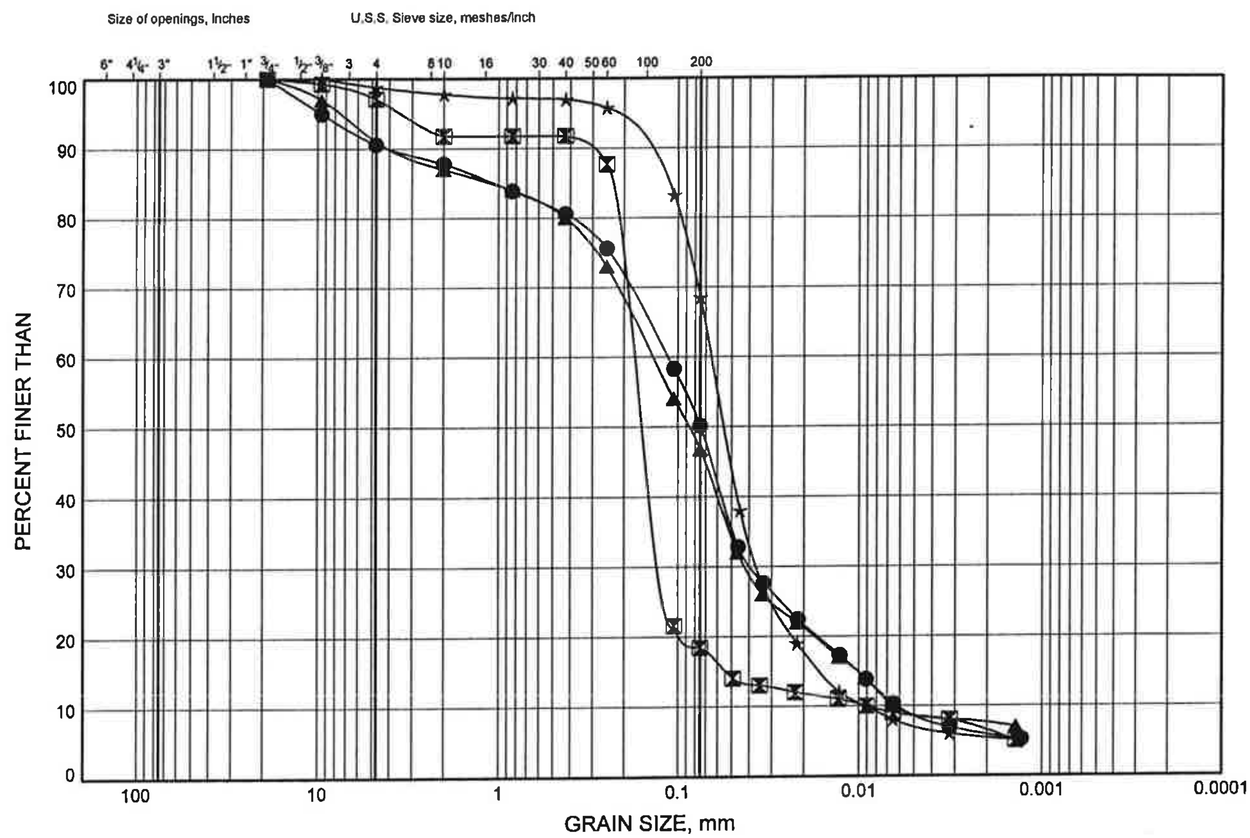
Chkd. RA

GSD 1-00-0350 HWY 410 ETOBICOKE CREEK.GPJ 12/02/07

GRAIN SIZE DISTRIBUTION

FIGURE B4

Sand and Silt Till (Upper Layer)



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-1	4.0	249.0
⊠	EC-5	3.2	243.7
▲	EC-6	3.2	244.4
★	EC-6	7.8	239.8

GSD 1-00-0350 HWY 410 ETOBICOKE CREEK.GPJ 12/02/07

Date July 2005

Project 107-00-01

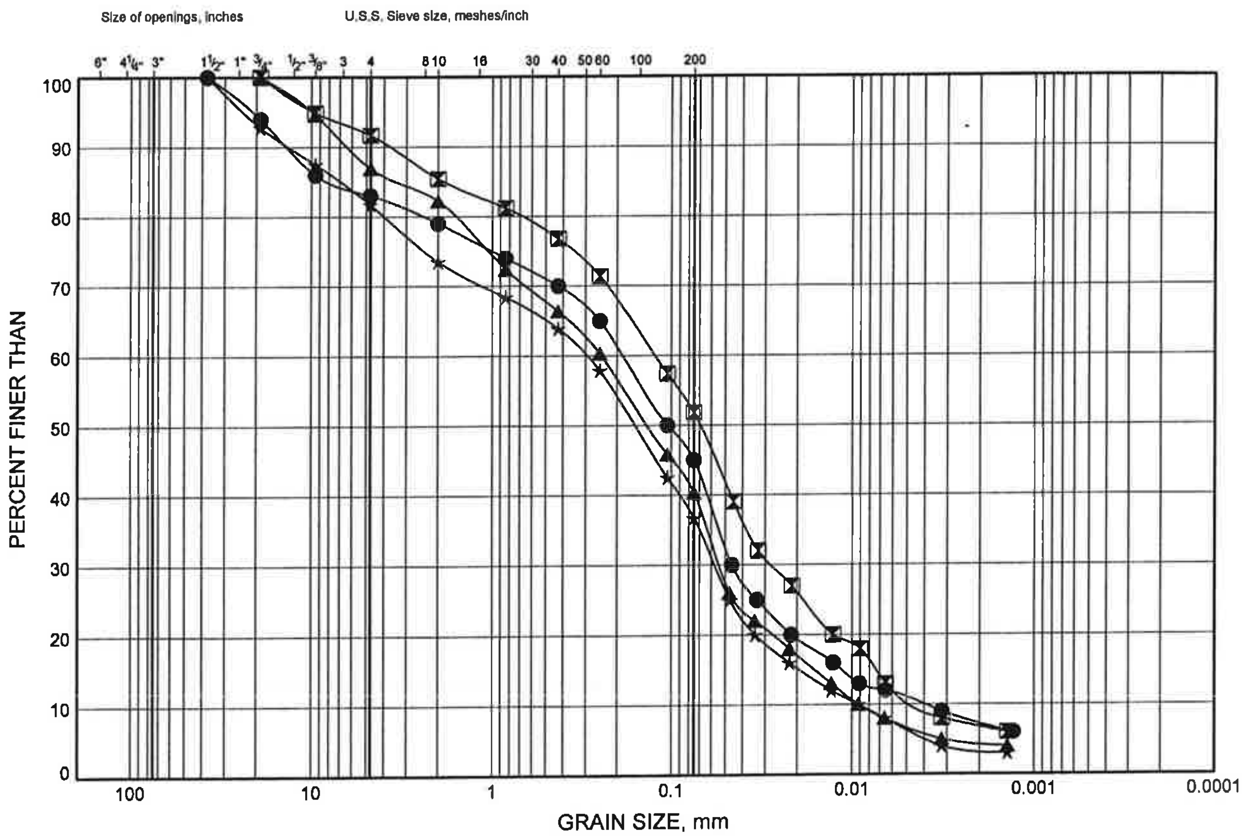
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B5

Sand and Silt Till (Lower Layer)



COBBLE SIZE	GRAVEL		SAND			FINE GRAINED
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-2	26.1	227.2
⊠	EC-4	21.5	224.6
▲	EC-5	23.1	223.8
★	EC-5	26.1	220.8

Date July 2005

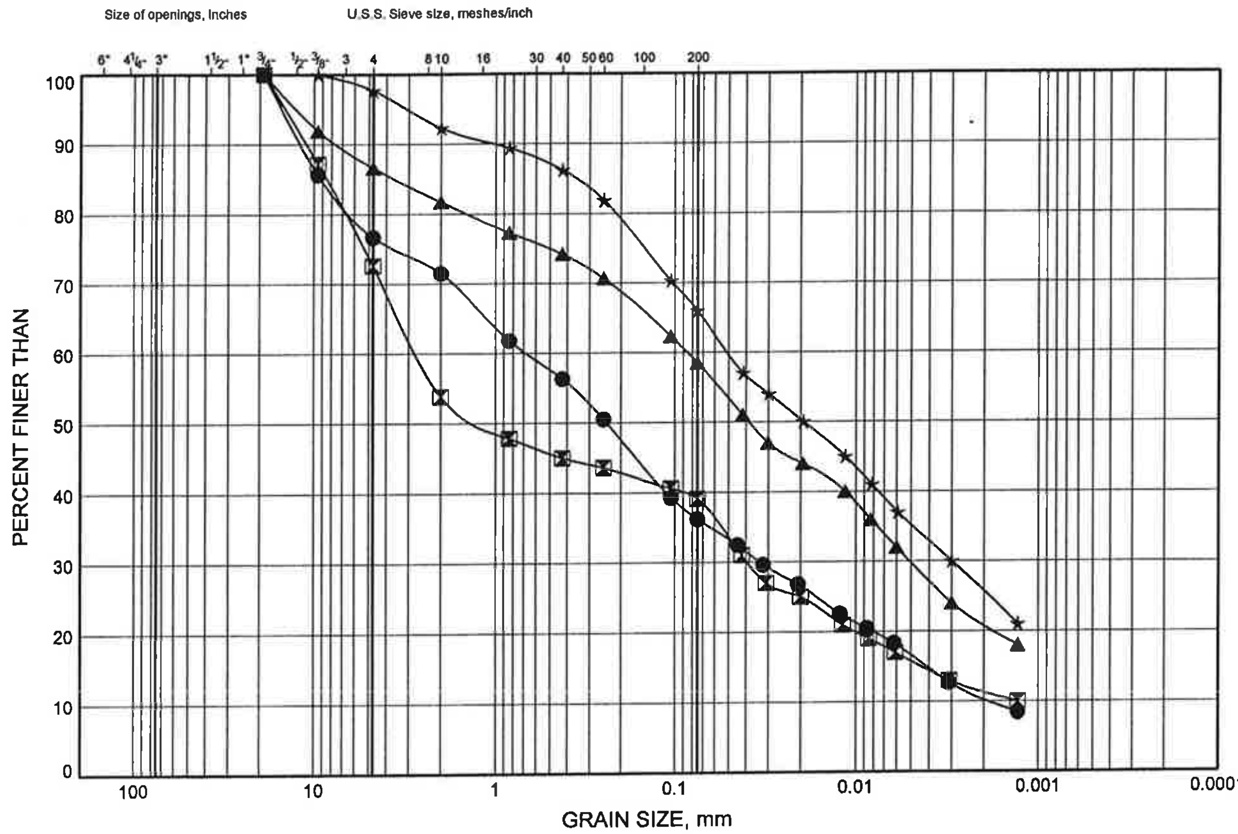
Project 107-00-01

Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION	FIGURE B6

Clayey Silt Till



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-2	2.5	250.8
⊠	EC-4	27.6	218.5
▲	EC-5	1.7	245.2
★	EC-6	1.0	246.6

Date July 2005

Project 107-00-01

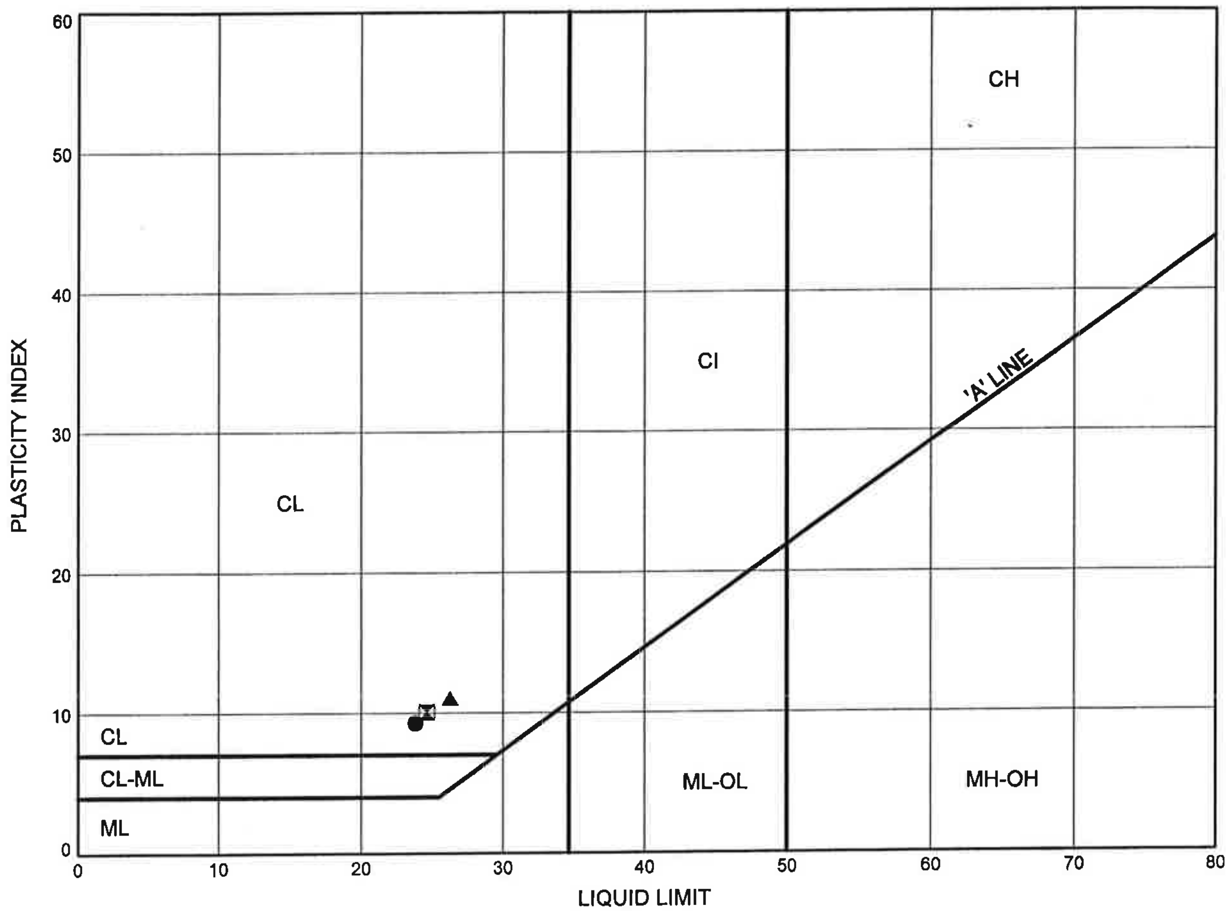
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B7

Clayey Silt Till



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-2	2.5	250.8
⊠	EC-5	1.7	245.2
▲	EC-6	1.0	246.6

Date July 2005

Project 107-00-01

Prep'd DB

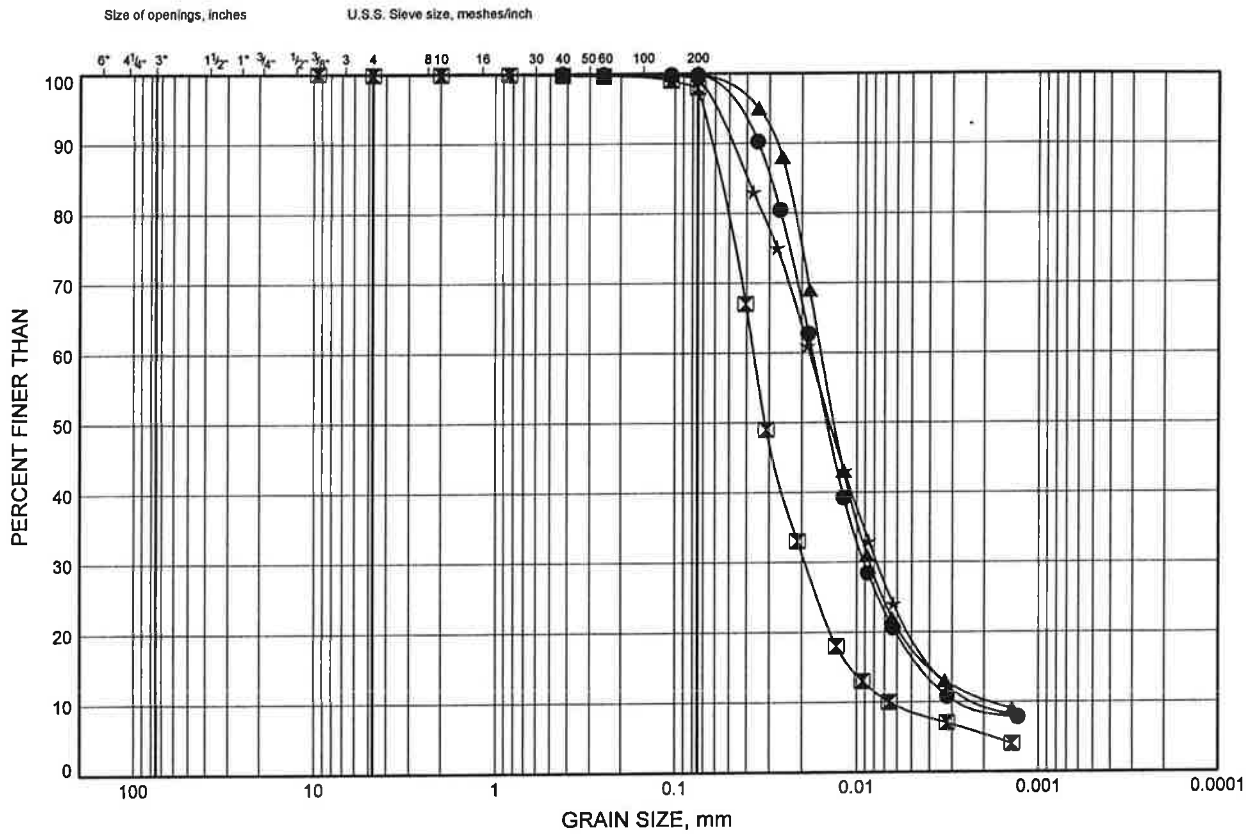
Chkd. RA

ALTR 1-00-0350 HWY 410 ETOBICOKE CREEK.GPJ 12/02/07

GRAIN SIZE DISTRIBUTION

FIGURE B8

Silt



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-2	10.9	242.4
⊠	EC-2	15.4	237.9
▲	EC-4	4.7	241.4
★	EC-4	7.8	238.3

Date July 2005

Project 107-00-01

Prep'd DB

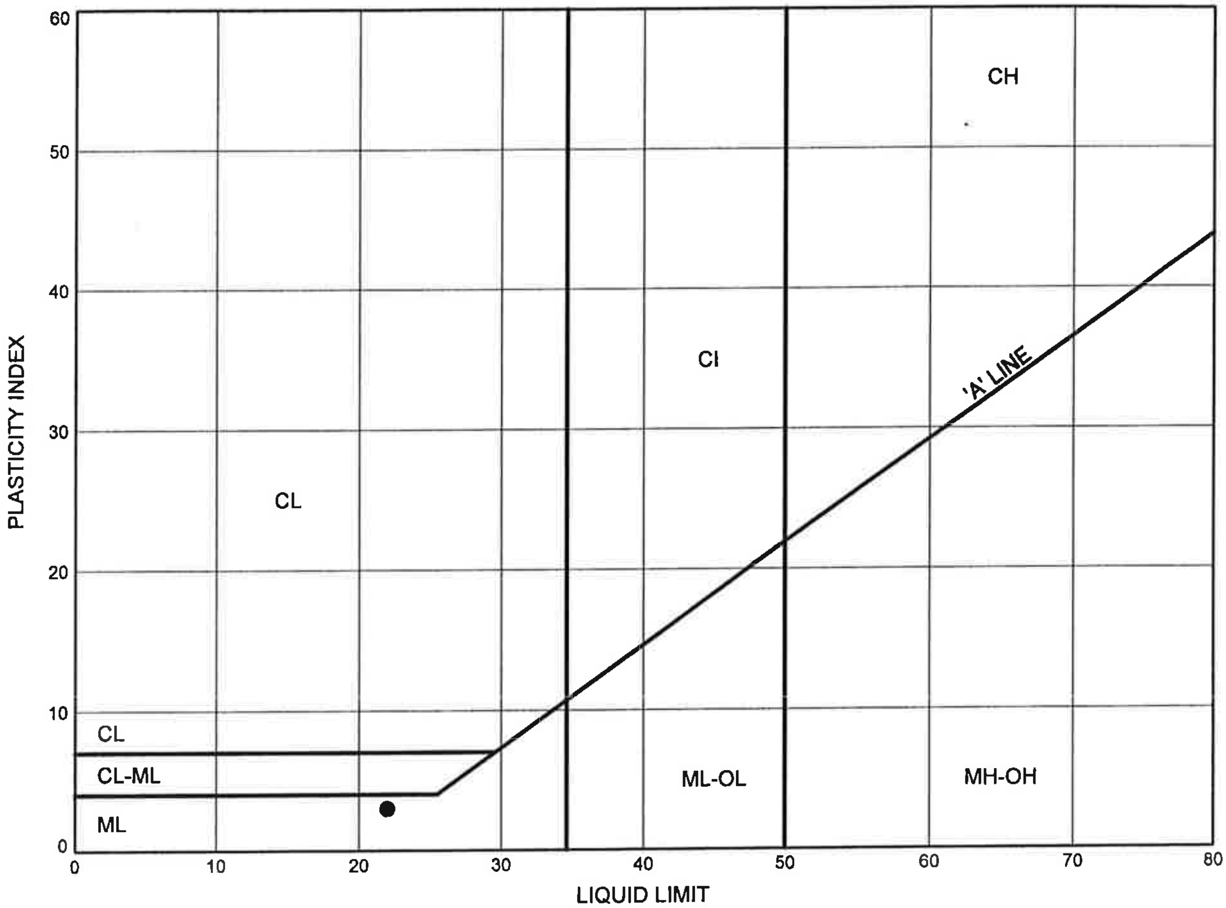
Chkd. RA

GSD 1-00-0350 HWY 410 ETOBICOKE CREEK.GPJ 12/02/07

ATTERBERG LIMITS TEST RESULTS

FIGURE B9

Silt



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-4	4.7	241.4

ALTR 1-00-0350 HWY 410 ETOBICOKE CREEK.GPJ 12/02/07

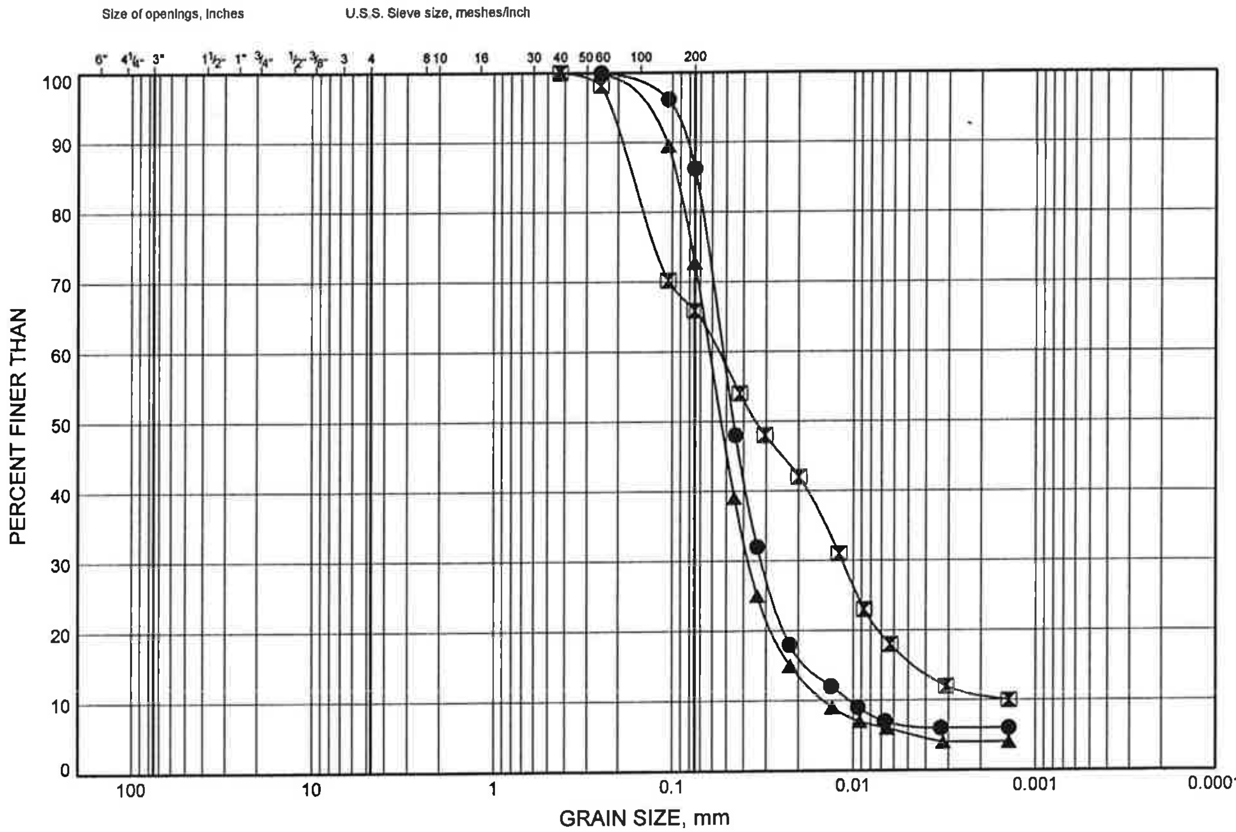
Date August 2005
Project 107-00-01

Prep'd DB
Chkd. RA

FIGURE B10

GRAIN SIZE DISTRIBUTION

Sandy Silt



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	EC-4	17.0	229.1
⊠	EC-5	17.0	229.9
▲	EC-5	21.5	225.4

Date July 2005

Project 107-00-01

Prep'd DB

Chkd. RA

GSD 1-00-0350 HWY 410 ETOBICOKE CREEK.GPJ 12/02/07

APPENDIX C

Record of Borehole Sheets (Previous Investigations)

Terraprobe Limited

W.P. 22-79-00
DIST. Stn 7+770 8.5
LOCATION: N 4844360 E 594793 (NAD 83)

RECORD OF BOREHOLE 123A

BORING DATE: Mar.17/18, 1999

SHEET 1 OF 3
DATUM: Geodetic
PROJECT: 981-6057

N605723A.BH

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	POWER AUGER, 205 mm Dia. Hollow Stem Augers	GROUND SURFACE	246.60					
		PEAT	0.00	1	AS	-		Bentonite Grout
1		SANDY SILT, trace gravel and organics, sand seam at 0.9 m depth, Loose, Brown, Moist	245.84 / 0.76	2	50 DO	10		
		GRAVELLY SAND, trace silt, Dense, Grey, Wet	245.23 / 1.37	3A	50 DO	26		
2		SANDY SILT, trace clay, Loose to compact, Grey, Wet	244.82 / 1.78	3B	50 DO	13		Backfill
				4	50 DO	11		
3				5	50 DO	6		
4		SILTY CLAY, trace sand, Hard, Grey, Wet	242.56 / 4.04					
5				6	50 DO	45	MH	
6				7	50 DO	62		
7		SILT, trace clay and sand, Compact to dense, Grey, Wet	239.51 / 7.09					
8				8	50 DO	22		Cave
9				9	50 DO	35		
10		CONTINUED ON NEXT PAGE						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - ○
HYDRAULIC CONDUCTIVITY, k, cm/s
WATER CONTENT, PERCENT Wp —○W— Wl 10 20 30 40

DATA INPUT: mg n605723a.bh 3/99

DEPTH SCALE 1 to 50

LOGGED: MG
CHECKED:

Golder Associates

SOILM6

RECORD OF BOREHOLE 123A

BORING DATE: Mar 17/18, 1999
DATUM: Geodetic
PROJECT: 981-8057

N605723A BH

Boring method: POWER AUGER, 205 mm Dia. Hollow Stem Augers

Depth (m)	Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.3m	Water content (%)	Additional lab testing	Piezometer or standpipe installation
10	CONTINUED FROM PREVIOUS PAGE							
	SILT, trace clay and sand Compact to dense Grey Wet							Cave
			10	50 DO	48	o	MH	
11								Bentonite Seal
12			11	50 DO	24	o		
13	SAND, trace silt Compact Grey Wet	233.41 13.19						
14			12	50 DO	20	o		
15			13	50 DO	11	o		
16	SILT, trace clay and sand Compact Grey Wet Reddish-brown clay seams below 16.3 m	230.37 16.23						Cave
17			14	50 DO	19	o		
18			15	50 DO	26	o	MH	
19	SILTY SAND, trace clay and gravel Compact to dense Reddish-brown Wet (TILL)	227.32 19.28						
20			16	50 DO	47			

CONTINUED ON NEXT PAGE

Column headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O; HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl, 10 20 30 40

DEPTH SCALE
1 to 50

LOGGED: MG
CHECKED:

DATA INPUT: mg n605723a.bh 3/99

SOILM6

RECORD OF BOREHOLE 123A

W.P. 22-79-00
DIST. Stn 7+770 8.5
LOCATION: N 4844360 E 594793 (NAD 83)

BORING DATE: Mar.17/18, 1999

DATUM: Geodetic
PROJECT: 981-8057

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp —O W— Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	POWER AUGER 205 mm Dia. Hollow Stem Augers	CONTINUED FROM PREVIOUS PAGE									
		SILTY SAND, trace clay and gravel Compact to dense Reddish-brown Wet (TILL) Gravelly sand seams at 24.4 m depth			16	50 DO	47				
21											
					17	50 DO	44				Cave
22											
23					18	50 DO	15			MH	
24											Silica Sand
					19	50 DO	49				
25											Slotted Screen with Filter Cloth
26					20	50 DO	40				
		END OF BOREHOLE		219.77 26.63							
27											Water level in open borehole at 1.5 m depth during drilling
28											Borehole caved to 3.4 m depth upon completion of drilling, Mar. 18, 1999
29											Water level in piezometer at 0.8 m depth on Mar. 19, 1999
30											

DATA INPUT: mg n605723a.bh 3/99

DEPTH SCALE
1 to 50

LOGGED: MG
CHECKED:

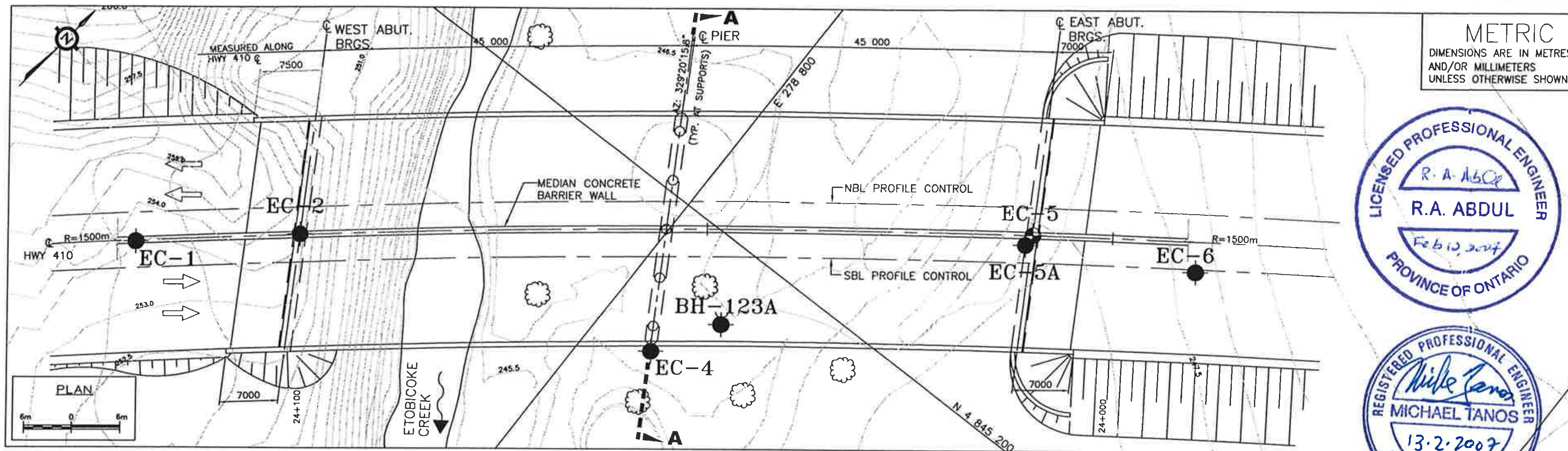
Golder Associates

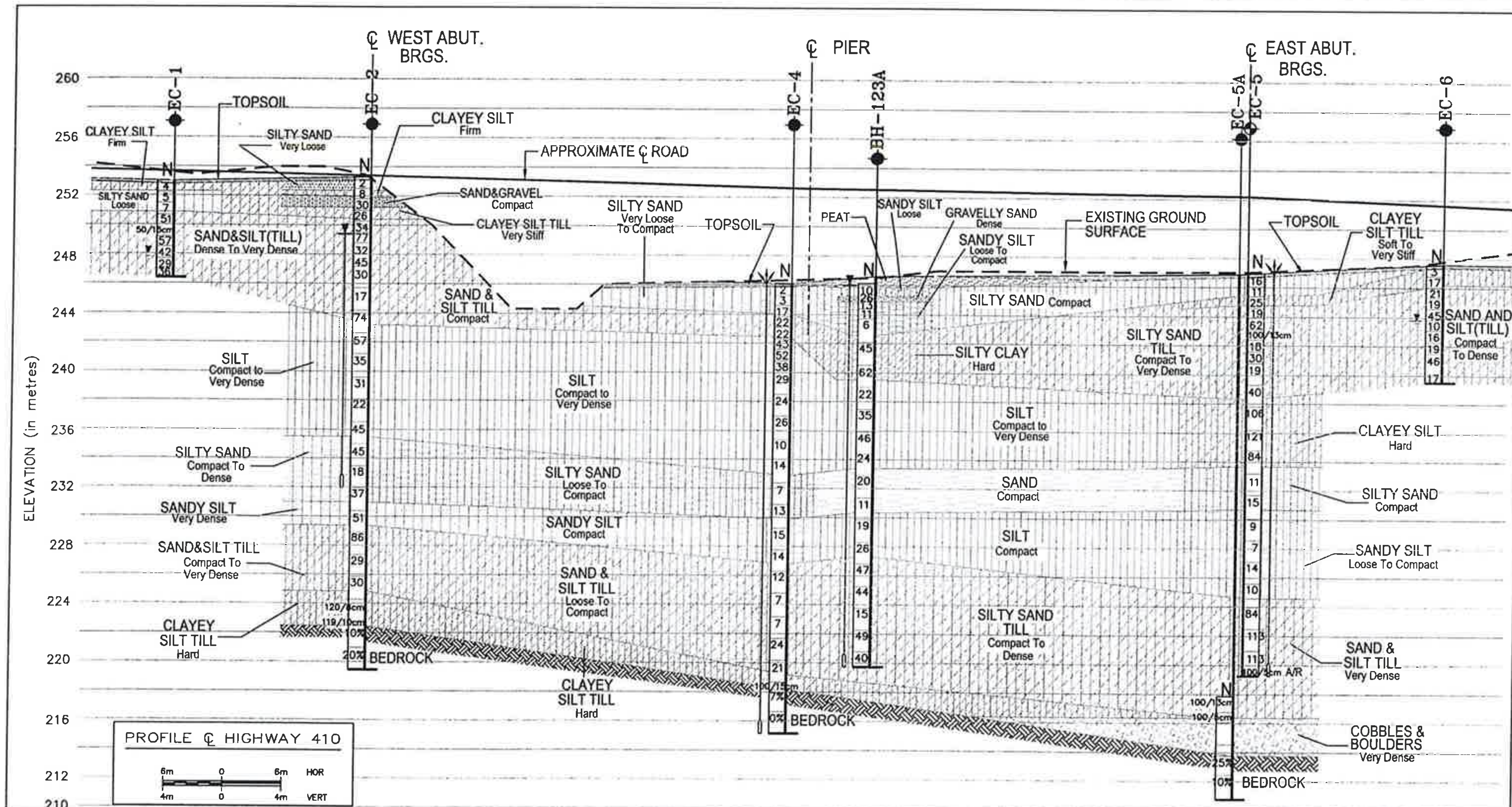
SOILM6

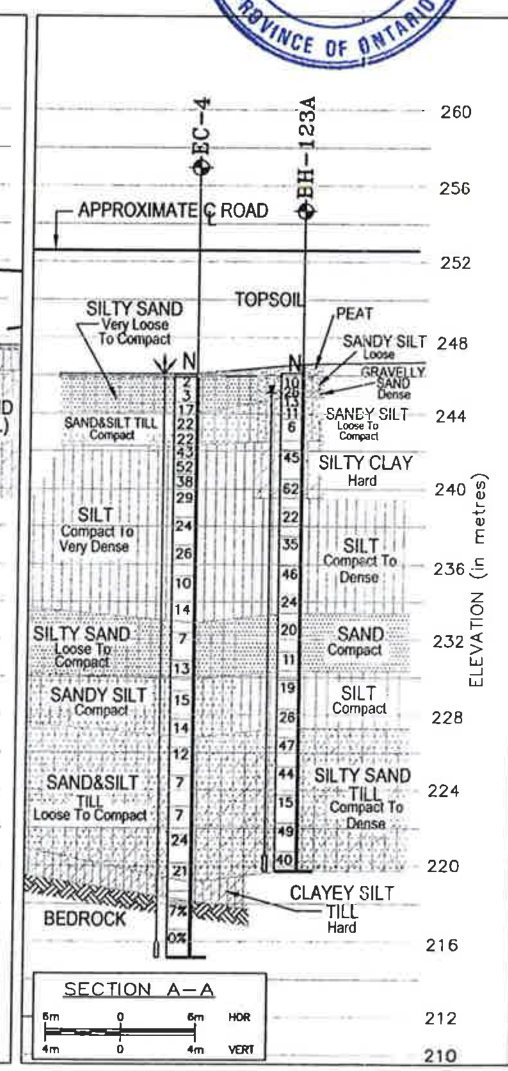
APPENDIX D

Drawing titled
"Borehole Locations and Soil Strata"

Terraprobe Limited







METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
Feb 13 2007
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
MICHAEL TANOS
13·2·2007
PROVINCE OF ONTARIO

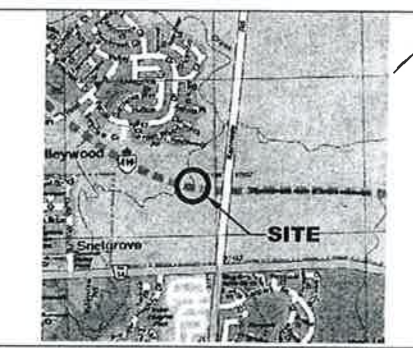
CONT. W.P. 107-00-01

HIGHWAY 410 – PHASE III
ETOBICOKE CREEK CROSSING
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 342

Giffels
An Ingenium Group Company

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing



KEY PLAN

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test
- Bore Hole And Cone
- 'N' Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60' Cone, 475 J/blow)
- WL at Time of Investigation
- WL in Piezometer 99 02
- Piezometer
- 90% Rock Quality Designation
- A/R Auger Refusal

No	ELEVATION	COORDINATES NORTHING	COORDINATES EASTING
EC-1	253.0	4845155.1	278749.2
EC-2	253.3	4845168.1	278764.5
EC-4	246.1	4845183.0	278807.6
EC-5	246.9	4845223.0	278835.8
EC-5A	246.9	4845221.5	278835.8
EC-6	247.6	4845231.7	278854.4
BH-123A	246.6	4845190.8	278812.3

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

REVISIONS

DATE	BY	DESCRIPTION

DESIGN R.A	CODE CHBDC2000	LOAD	DATE AUG.2005
DRAWN P.S	CHK R.A	SITE: 24-744	STRUCT / DWG 2

APPENDIX E

Foundation Comparison

Terraprobe Limited

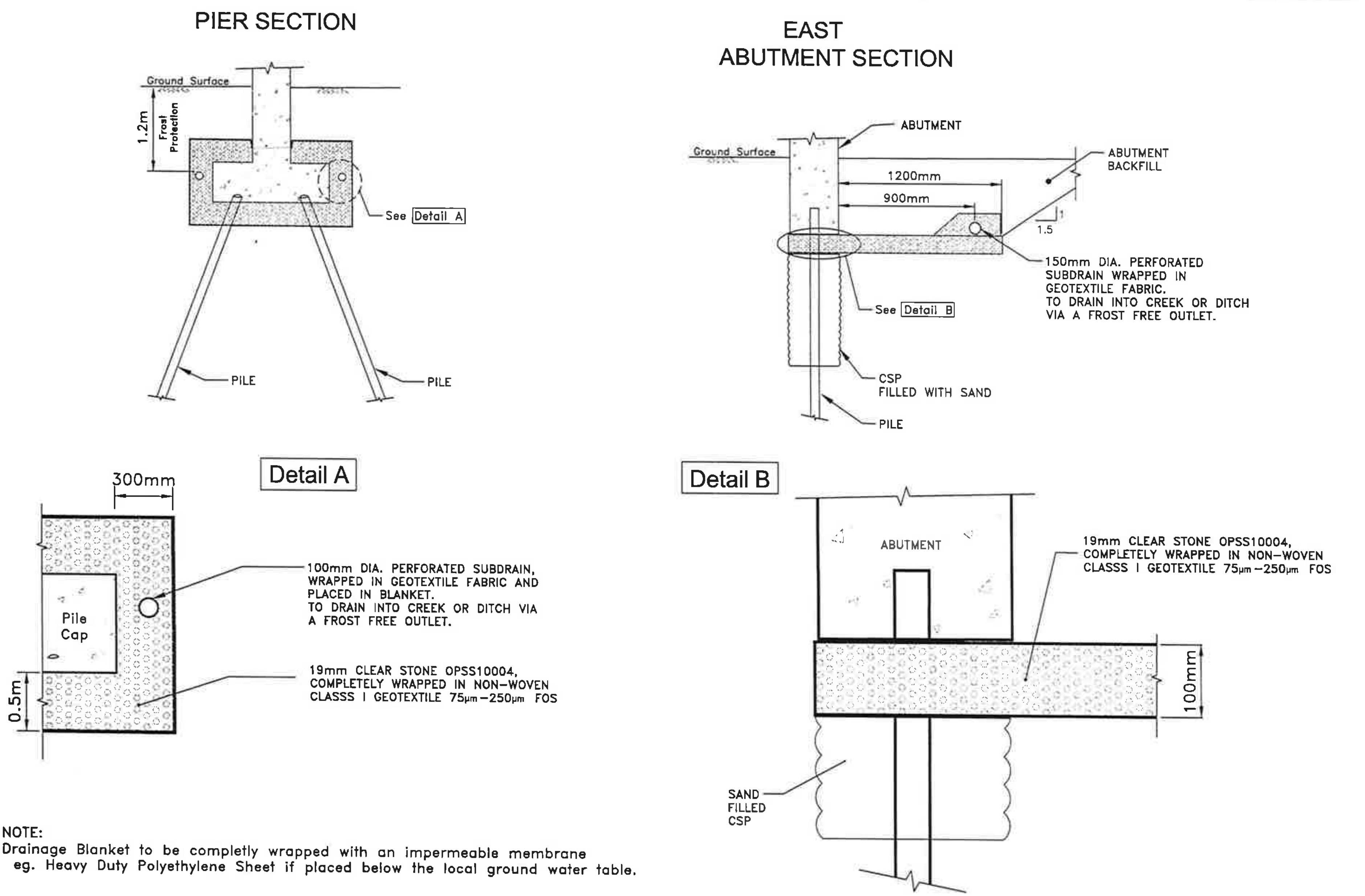
COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Driven Piles	Augered Caissons	Footing on Native Soil	Footing on Engineered Fill
East and West Abutments	***Advantages:*** i. High geotechnical resistances available by driving piles to bedrock or very dense sand and silt till. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed ***Disadvantages:*** i. Higher unit cost compared to footings. ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.	***Advantages:*** N/A ***Disadvantages:*** i. Higher unit cost compared to other footing options such as driven piles. ii. Relatively high construction effort required to install caissons to bedrock compared to driven piles	***Advantages:*** i. Less costly compared to driven piles. ***Disadvantages:*** i. Precludes consideration of an integral abutment structure. ii. Comparatively longer abutment stem required at east abutment.	***Advantages:*** N/A
Pier	***Advantages:*** i. High geotechnical resistances available by driving piles to bedrock. ii. Readily installed ***Disadvantages:*** i. Higher unit cost compared to footings. ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.	***Advantages:*** N/A ***Disadvantages:*** i. Higher unit cost compared to other footing options such as driven piles. ii. Relatively high construction effort required to install caissons to bedrock compared to driven piles.	***Advantages:*** i. Less costly compared to driven piles. ***Disadvantages:*** i. Low geotechnical resistance when submerged will result in an uneconomically large footing size. ii. Potential for unacceptable settlements and differential settlement.	***Advantages:*** i. Less costly compared to driven piles. ***Disadvantages:*** i. Low geotechnical resistance when submerged will result in an uneconomically large footing size.

APPENDIX F

Figures

Terraprobe Limited



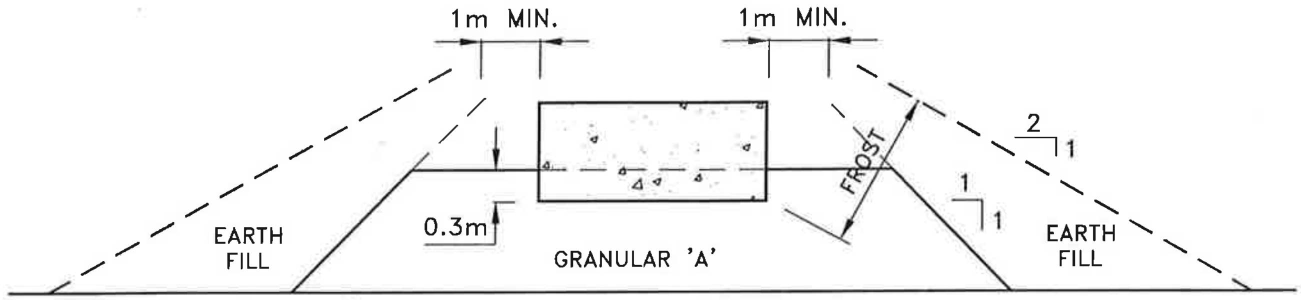
NOTE:
Drainage Blanket to be completly wrapped with an impermeable membrane eg. Heavy Duty Polyethylene Sheet if placed below the local ground water table.

PILE CAP DRAINAGE SCHEME

TERRAPROBE

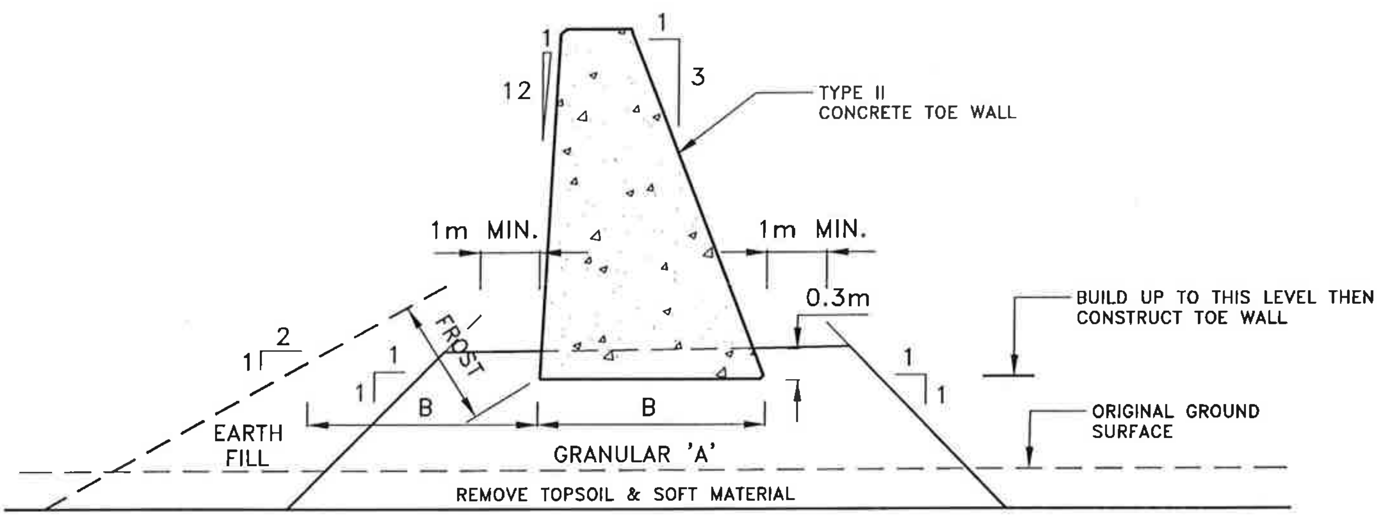
File No. 1–00–0350

FIGURE F1



CROSS-SECTION

N.T.S.



LONGITUDINAL SECTION

N.T.S.

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO OPSS 501.
3. CONSTRUCT CONCRETE TOE WALL.
4. PLACE REMAINDER OF GRANULAR "A" AND EARTH FILL AS REQUIRED.

CONCRETE TOE WALL (Type II) on COMPACTED FILL showing GRANULAR 'A' CORE

APPENDIX G

Suggested NSSP Wording

Terraprobe Limited

In this report reference is made to the following Provincial Standard:

- SP 903S01

The contract documents should contain a NSSP containing the following wording:

Cobbles and Boulders

"The Contractor is informed that the soils at this site may contain cobbles and boulders that could impede the progress of pile driving operations. The soil conditions are described in the Foundation Investigation Report prepared for this site".

If a pile encounters refusal on cobbles and boulders the QVE should terminate driving before the pile is damaged by overdriving. If the required resistance according to the Hiley Formula is not achieved and further driving is likely to cause damage to the pile, pile driving should cease and the contract administrator should be notified.