

Terraprobe

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

**FOUNDATION INVESTIGATION & DESIGN REPORT
STORM SEWER INSTALLATIONS
QUEEN ELIZABETH HIGHWAY
FROM BRANT STREET TO BURLOAK DRIVE
AGREEMENT No. 2006-E-0026, W.P. 2831-02-01
MINISTRY OF TRANSPORTATION, ONTARIO
CENTRAL REGION**

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PART 1: FACTUAL INFORMATION**

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted along storm sewer alignments crossing the QEW at Sta. 15+425 (Walkers Line) and Sta. 17+609 (Appleby Line). This project is the Ministry of Transportation of Ontario undertaking to rehabilitate and widen the QEW from Brant Street to Burloak Drive.

The purpose of this investigation was to explore the subsurface conditions along the alignments and, based on the data obtained, to provide borehole location plans, records of boreholes, stratigraphic profiles, laboratory test results and descriptions of the subsurface conditions. Models of the subsurface conditions were developed from the data obtained.

Terraprobe conducted the investigation as a sub-consultant to Giffels Associates Ltd./IBI Group (Giffels), under the Ministry of Transportation Ontario (MTO) Agreement Number 2006-E-0026.

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

- Terraprobe Limited, "Foundation Investigation and Design Report, High Mast Lighting, Queen Elizabeth Highway, From Brant Street to Burloak Drive", W.P. 2831-02-01, MTO Central Region, dated August 29 2008.

For reporting purposes the investigated sections are designated as Walkers Line Crossing and Appleby Line Crossing. Further details are outlined below.

Walkers Line Crossing: A 750 mm to 825 mm diameter storm sewer crossing the East Bound and West Bound lanes of the QEW at Sta. 15+425.

Appleby Line Crossing: A 525 mm diameter storm sewer crossing the East Bound lanes of the QEW at Sta. 17+609.

2 SITE DESCRIPTION & PHYSIOGRAPHY

This project is located in the Regional Municipality of Halton, City of Burlington, Ontario, and extends a distance of approximately 8.2 km from Sta. 11+700 to Sta. 10+330. Within the project limits, this divided highway comprises of six lanes and fully paved shoulders. There is an existing storm sewer located close to the median centreline of the highway. There are four interchanges within the project limits: Guelph Line, Walkers Line, Appleby Line and Burloak Drive.



A significant feature at Walkers Line is Tuck Creek, which crosses the QEW at Sta. 15+370. When the QEW was constructed provisions were made to cross this watercourse by constructing a concrete culvert. Fill was placed in the creek valley to achieve the current grade profile of the QEW.

The site is located in the physiographic region of Southern Ontario referred to as the Iroquois Plain¹. This strip of land is approximately 3 km wide and is located between the shoreline of the former glacial lake, Lake Iroquois and Lake Ontario. The topography is flat to moderately rolling and the terrain slopes gently towards Lake Ontario.

The soils generally consist of fine grained silts and clays, underlain by silty clay glacial till. The overburden soils are further underlain by bedrock of the Queenston Formation, which is predominantly shale and is known to exist at relatively shallow depths within the project limits. Very often the basal portion of this till is distinctly red in colour from large amounts of incorporated Queenston shale.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period January 25 to 27, 2009 and consisted of drilling and sampling four boreholes each to a depth of 7.8 m below ground surface. Borehole HML-3/3A from Terraprobe's previous work was drilled on August 01, 2007 and October 04, 2007 to a depth of 9.2 m below ground surface. The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawings in Appendix C.

Solid stem auger drilling techniques 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 and the bedrock. The boreholes were also advanced approximately 3.1 m to 4.9 m into bedrock by NQ size diamond coring techniques.

Borehole S3 was drilled through the paved left shoulder of the QEW EBL. This borehole was sealed using bentonite and the pavement structure was reinstated by backfilling with granular material and patching with cold mix asphalt.

Terraprobe's staff observed the drilling and recorded the sampling, in-situ testing and rock coring operations on a full time basis. The staff logged the boreholes and processed the recovered soil samples and rock cores for transport to Terraprobe's Brampton laboratory for further examination and testing.

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

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



Table 3.1 – Piezometer Installation Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
Walkers Line Crossing (Sta. 15+425)		
S1 - P1	7.8/107.0	Piezometer with 1.5 m slotted screen installed with filter sand to 5.9 m, bentonite seal from 5.9 m to 3.7 m, filter sand from 3.7 m to 1.8 m and bentonite seal from 1.8 m to ground surface.
S1 - P2	3.7/111.1	Piezometer with 1.5 m slotted screen installed with filter sand to 1.8 m and bentonite seal from 1.8 m to ground surface.
S2 - P1	7.8/107.2	Piezometer with 1.5 m slotted screen installed with filter sand to 5.9 m, bentonite seal from 5.9 m to 3.0 m, filter sand from 3.0 m to 1.2 m and bentonite seal from 1.2 m to ground surface.
S2 - P2	3.0/111.9	Piezometer with 1.5 m slotted screen installed with filter sand to 1.2 m and bentonite seal from 1.2 m to ground surface.
Appleby Line Crossing (Sta. 17+609)		
S4 - P1	7.6/110.2	Piezometer with 1.5 m slotted screen installed with filter sand to 5.8 m, bentonite seal from 5.8 m to 4.6 m, filter sand from 4.6 m to 2.4 m, bentonite seal from 2.4 m to 0.3 m and a flush mounted casing installation from 0.3 m to ground surface.
S4 - P2	4.3/113.5	Piezometer with 1.5 m slotted screen installed with filter sand to 2.4 m, bentonite seal from 2.4 m to 0.3 m and a flush mounted casing installation from 0.3 m to ground surface.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and natural moisture content determination. Selected samples were also subjected to gradation analysis and Atterberg Limits tests. Rock core samples were subjected to unconfined compressive strength tests and unit weight tests. The results of the laboratory testing program are shown on the Record of Borehole sheets and Core Logs in Appendix A. The grain size distribution curves and plasticity charts are illustrated in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A and the Core Logs. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawings in Appendix C. An overall description of the stratigraphy at each site is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets and Core Logs governs any interpretation of the site conditions.



5.1 Walkers Line Crossing (Sta. 15+425)

In general, the site is underlain by surficial layers of topsoil and a flexible pavement followed by firm to stiff silty clay fill, stiff to hard silty clay till and hard till/shale complex. These overburden soils are further underlain by shale bedrock of the Queenston Formation.

5.1.1 Topsoil

Topsoil ranging from 230 mm to 280 mm was encountered at this site. Topsoil thickness may vary between and beyond the boreholes.

5.1.2 Flexible Pavement

Borehole HML 3/3A indicates that the inner shoulder of the QEW WBL consists of 280 mm of asphalt concrete underlain by a 720 mm thick layer of gravelly sand fill that extends to a depth of 1.0 m (Elev. 114.7 m) below ground surface.

Standard Penetration tests conducted in the granular fill gave SPT "N" values ranging from 23 blows to 29 blows for 0.3 m penetration indicating a compact relative density. The moisture content (by weight) of samples of the granular fill was 1%.

5.1.3 Fill – Silty Clay

Silty clay fill was encountered at the site extending to depths ranging from 3.7 m (Elev. 111.1 m) to 4.9 m (Elev. 110.8 m) below ground surface.

The grain size distribution curves of samples of this fill are shown in Figure B1. The results show a grain size distribution consisting of 1% to 20% gravel, 21% to 34% sand, 34% to 50% silt and 15% to 33% clay size particles.

Samples of the silty clay fill were also subjected to Atterberg Limits tests and the results are illustrated in Figure B2. The summarized index values from these tests are presented herein.

Liquid Limit:	31 to 36%
Plastic Limit:	19 to 22%
Plasticity Index:	12 to 15%
Natural Moisture Content:	8 to 19%

These values are characteristic of clayey soils of low to intermediate plasticity.

Standard Penetration tests conducted in the silty clay fill gave SPT "N" values ranging from 6 blows to 10 blows for 0.3 m penetration indicating a firm to stiff consistency. The moisture content of samples of the silty clay fill ranged from 8% to 27% by weight.



5.1.4 Silty Clay Till

Silty clay glacial till was encountered at the site extending to a depth of 1.0 m (Elev. 113.9 m) below ground surface. Till soils can also be expected to contain random cobble and boulder inclusions.

The grain size distribution curve of a sample of the silty clay till is illustrated in Figure B3. These results show a grain size distribution consisting of 3% gravel, 32% sand, 42% silt and 23% clay size particles.

A sample of the silty clay till was also subjected to an Atterberg Limits test and the results are illustrated in Figure B4. The summarized index values from this test are presented herein.

Liquid Limit:	26%
Plastic Limit:	18%
Plasticity Index:	8%
Natural Moisture Content:	15%

These values are characteristic of clayey soils of low plasticity.

Standard Penetration tests in the silty clay till gave "N" values ranging from 12 to 34 blows for 0.3 m. Based on these results the silty clay till is considered to have a stiff to hard consistency.

The moisture content of samples from this deposit ranged from 13% to 15% by weight.

5.1.5 Silty Clay Till - Till/Shale Complex

The lower portions of the glacial till above the shale bedrock are difficult to distinguish from the upper, highly weathered shale. This transition zone of material is sometimes referred to as till/shale complex. The unit may often be described as residual soil or completely weathered shale bedrock. Shale and limestone slabs may occur within this deposit.

The till/shale complex extends to depths ranging from 1.8 (Elev. 113.1 m) to 4.1 m (Elev. 110.7 m) below ground surface.

The results of a grain size distribution test conducted on a sample obtained from this deposit are shown in Figure B5. These results show a grain size distribution consisting of 14% gravel, 21% sand, 52% silt and 13% clay size particles.

A sample of the till/shale complex was also subjected to an Atterberg Limits test and the results are plotted on the plasticity chart in Figure B6.



The index values from these tests are summarized below:

Liquid Limit:	20%
Plastic Limit:	15%
Plasticity Index:	5%
Natural Moisture Content:	8%

These values are characteristic of clayey soils of low plasticity.

Standard Penetration tests in the till/shale complex gave "N" values ranging from 34 blows to more than 100 blows for 0.3 m penetration. Based on these results the till/shale complex is considered to have a hard consistency.

The moisture content of samples from this deposit ranged from 6% to 8% by weight.

5.1.6 Bedrock

The bedrock beneath the site is of the Queenston Formation, a deposit predominantly comprised of thickly bedded to massive reddish brown shale of Ordovician age. The rock contains within the shale matrix occasional layers of limestone, sandstone and siltstone, and occasionally green calcareous shale layers. There is typically a horizontal zone of weathering at the contact between the weak rock of the Queenston Formation and the glacial soil overburden. In the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects*, there is reproduced from Skempton, Davis and Chandler, *a typical weathering profile of a low durability shale*, that characterizes the shale surface into three grades of weathering and four zones described and interpreted as follows:

	Zone	Description	Notes
Fully Weathered	IVb	soil like matrix only	indistinguishable from glacial drift deposits, slightly clayey, may be fissured
Partially Weathered	IVa	soil like matrix with occasional pellets of shale less than 3 mm diameter	little or no trace of rock structure, although matrix may contain relic fissures
	III	soil like matrix with frequent angular shale particles up to 25 mm diameter	moisture content of matrix greater than the shale particles
	II	angular blocks of unweathered shale with virtually no matrix separated by weaker chemically weathered but intact shale	spheroidal chemical weathering of shale pieces emanating from relic joints and fissures, and bedding planes
Unweathered (sound)	I	shale	regular fissuring

At the base of the glacial till deposit there is sometimes found a zone of silty clay and fragmented shale that can be interpreted as the lowest portion of the till or as partially weathered rock of Zone III. The distinction is subjective and depends on the investigator.



The surface of the bedrock as indicated on the Borehole Logs from this investigation is to be consistently interpreted as the surface of Zone II in the profile.

Shale bedrock was encountered within the depth of investigation. The bedrock was penetrated by solid stem augering and samples were obtained by split spoon sampling. The bedrock was also cored approximately 3.3 m to 4.9 m using NQ-sized diamond drilling techniques.

Tabulated below are the bedrock depth and elevation at the borehole locations.

BH No.	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
S1	4.1	110.7
HML3/3A	4.9	110.8
S2	1.8	113.1

The bedrock is described as slightly to highly weathered to depths ranging from 4.1 m to 7.3 m and is unweathered below. It is generally thinly to medium bedded shale, occasionally laminated with interbeds of greenish grey to light grey limestone and dolostone. Total core recovery ranged from 98% to 100% and the RQD values generally ranged from 39% to 82% indicating poor to good quality rock. In Borehole HML3/3A an RQD value of 15% was obtained in the first run, indicating very poor rock quality.

Three unconfined compressive strength tests were conducted on core samples of the shale bedrock retrieved between Elev. 109.3 m and Elev. 111.3 m. The unconfined compressive strengths ranged between 8.3 MPa and 44.1 MPa indicating low to medium strength rock. The unit weight of the rock ranged from 21.3 kN/m³ to 27.6 kN/m³.

5.2 Appleby Line Crossing (Sta. 17+609)

In general, the site is underlain by a flexible pavement followed by stiff to hard silty clay fill, very stiff to hard silty clay till and hard till/shale complex. These overburden soils are further underlain by shale bedrock of the Queenston Formation.

5.2.1 Flexible Pavement

Borehole S3 indicates that the inner shoulder of the QEW EBL consists of 130 mm of asphalt concrete underlain by a 270 mm thick layer of sand and gravel fill that extends to a depth of 0.4 m (Elev. 118.4 m) below ground surface.

A Standard Penetration test conducted in the granular fill gave an SPT "N" value of 86 blows for 0.3 m penetration indicating a very dense relative density. The moisture content (by weight) of a sample of the granular fill was 6%.



5.2.2 Fill – Silty Clay

Silty clay fill was encountered at this site extending to depths ranging from 0.7 m (Elev. 117.1 m) to 1.8 m (Elev. 117.0 m).

Grain size distribution curves of samples of this fill material are presented in Figure B7. These results show grain size distributions consisting of 1% to 15% gravel, 7% to 23% sand, 46% to 72% silt and 16% to 20% clay size particles.

Samples of the silty clay fill were also subjected to Atterberg Limits tests and the results are illustrated in Figure B8. The summarized index values from these tests are presented herein.

Liquid Limit:	24 to 32%
Plastic Limit:	18 to 21%
Plasticity Index:	6 to 11%
Natural Moisture Content:	10 to 15%

These values are characteristic of clayey soils of low plasticity.

Standard Penetration tests in the silty clay fill material yielded "N" values ranging from 12 blows to 69 blows for 0.3 m penetration. Based on these results the fill is considered to have a stiff to hard consistency.

The moisture content of samples of this fill ranged from 10% to 15% by weight.

5.2.3 Silty Clay Till

Silty clay glacial till was encountered at the site extending to depths ranging from 1.4 m to 2.4 m below ground surface or to Elev. 116.4 m.

The grain size distribution curve of a sample of the silty clay till is illustrated in Figure B9. These results show a grain size distribution consisting of 8% gravel, 12% sand, 57% silt and 23% clay size particles. Till soils can also be expected to contain random cobble and boulder inclusions.

A sample of the silty clay till was also subjected to an Atterberg Limits test and the results are illustrated in Figure B10. The summarized index values from these tests are presented herein.

Liquid Limit:	36%
Plastic Limit:	23%
Plasticity Index:	13%
Natural Moisture Content:	14%



These values are characteristic of clayey soils of intermediate plasticity.

Standard Penetration tests in this deposit gave "N" values ranging from 24 blows to more than 100 blows for 0.3 m. Based on these results the silty clay till is considered to have a very stiff to hard consistency.

The moisture content of samples from this deposit ranged from 11% to 14% by weight.

5.2.4 Silty Clay Till - Till/Shale Complex

The lower portions of the glacial till above the shale bedrock are difficult to distinguish from the upper, highly weathered shale. This transition zone of material is sometimes referred to as till/shale complex. The unit may often be described as residual soil or completely weathered shale bedrock. Shale and limestone slabs may occur within this deposit.

The till/shale complex extends to depths ranging from 2.3 m (Elev. 115.5 m) to 4.2 m (Elev. 114.6 m) below ground surface.

The results of a grain size distribution test conducted on a sample obtained from this deposit are shown in Figure B11. These results show a grain size distribution consisting of 1% gravel, 16% sand, 66% silt and 17% clay size particles.

A sample of the till/shale complex was also subjected to an Atterberg Limits test and the results are plotted on the plasticity chart in Figure B12.

The index values from these tests are summarized below:

Liquid Limit:	25%
Plastic Limit:	17%
Plasticity Index:	8%
Natural Moisture Content:	6%

These values are characteristic of clayey soils of low plasticity.

Standard Penetration tests in the till/shale complex gave "N" values ranging from 30 blows to more than 100 blows for 0.3 m penetration. Based on these results the till/shale complex is considered to have a hard consistency.

The moisture content of samples from this deposit ranged from 6% to 12% by weight.

5.2.5 Bedrock

The bedrock beneath the site is of the Queenston Formation, a deposit predominantly comprised of thickly bedded to massive brick red shale of Ordovician age. The rock contains within the shale matrix occasional layers of limestone, sandstone and siltstone, and occasionally green calcareous shale layers. There is typically a horizontal zone of weathering at the contact between the weak rock of the Queenston Formation and the



glacial soil overburden. In the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects*, there is reproduced from Skempton, Davis and Chandler, *a typical weathering profile of a low durability shale*, that characterizes the shale surface into three grades of weathering and four zones described and interpreted as follows:

	Zone	Description	Notes
Fully Weathered	IVb	soil like matrix only	indistinguishable from glacial drift deposits, slightly clayey, may be fissured
Partially Weathered	IVa	soil like matrix with occasional pellets of shale less than 3 mm diameter	little or no trace of rock structure, although matrix may contain relic fissures
	III	soil like matrix with frequent angular shale particles up to 25 mm diameter	moisture content of matrix greater than the shale particles
	II	angular blocks of unweathered shale with virtually no matrix separated by weaker chemically weathered but intact shale	spheroidal chemical weathering of shale pieces emanating from relic joints and fissures, and bedding planes
Unweathered (sound)	I	shale	regular fissuring

At the base of the glacial till deposit there is sometimes found a zone of silty clay and fragmented shale that can be interpreted as the lowest portion of the till or as partially weathered rock of Zone III. The distinction is subjective and depends on the investigator. The surface of the bedrock as indicated on the Borehole Logs from this investigation is to be consistently interpreted as the surface of Zone II in the profile.

Shale bedrock was encountered within the depth of investigation. The bedrock was penetrated by solid stem augering and samples were obtained by split spoon sampling. The bedrock was also cored approximately 3.1 m using NQ-sized diamond drilling techniques.

Tabulated below are the bedrock depth and elevation at the borehole locations.

BH No.	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
S3	4.2	114.6
S4	2.3	115.5

The bedrock is described as moderately weathered to depths ranging from 4.7 m to 5.0 m and is unweathered below. It is generally thinly bedded to medium bedded shale that is occasionally laminated with interbeds of light greenish grey to light grey limestone and dolostone. Total core recovery ranged from 99% to 100% and the RQD values ranged from 46% to 93% indicating poor to excellent quality rock.



Two unconfined compressive strength tests were conducted on the shale bedrock at elevations ranging between 112.2 m and 113 m. The results ranged between 21.4 MPa and 22.7 MPa indicating medium strength rock. The unit weight of the rock ranged from 26.4 kN/m³ to 27.9 kN/m³.

5.3 Water Levels

Standpipe piezometers were installed in selected boreholes and water level readings were taken on separate visits made after the completion of drilling. The water level records are presented in Table 5.3.

Table 5.3 – Water Level Measurements

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
Walkers Line Crossing (Sta. 15+425)			
S1-P1*	January 30, 2009	4.3	110.5
	February 03, 2009	4.3	110.5
S1-P2**	January 30, 2009	3.4	111.4
	February 03, 2009	3.6	111.2
S2-P1*	January 30, 2009	3.9	111.0
	February 03, 2009	4.3	110.6
S2-P2**	January 30, 2009	2.1	112.8
	February 03, 2009	2.0	112.9
Appleby Line Crossing (Sta. 17+609)			
S4-P1*	January 30, 2009	3.2	114.6
	February 03, 2009	3.1	114.7
S4-P2	January 30, 2009	2.6	115.2
	February 03, 2009	3.1	114.7

* Standpipe piezometer installed and sealed in the bedrock

** Standpipe piezometer installed in the overburden soils

At the Walkers Line Crossing the recorded water levels in the bedrock range between Elev. 110.5 m and Elev. 111.0 m. The water level readings in the overburden soils indicate that the groundwater level is likely to exist at elevations ranging between ±111.2 m and ±112.9 m.

At the Appleby Line Crossing the recorded water level in the bedrock ranges between Elev. 114.6 m and Elev. 114.7 m. The estimated groundwater level in the overburden is Elev. ±117 m.

All groundwater observations at the two sites are short term and the levels are expected to fluctuate seasonally and after severe weather events. The ground water level at the Walkers Line Crossing will also be controlled by the free water level in Tuck Creek.



5.4 Miscellaneous

The borehole locations, their coordinates and geodetic elevations, were established in the field by surveyors from Strada Survey Inc. of Vaughan, Ontario based on drawings provided by Giffels,

The drilling, sampling and in-situ testing operations were conducted using both truck-mounted and track-mounted drill rigs owned and operated Geo-Environmental Drilling Inc. of Milton, Ontario.

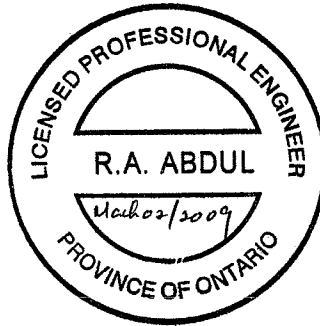
The utility locates, fieldwork planning and its coordination were undertaken by Mr. H. Ahmed, P.Eng. Drilling and sampling operations were observed and recorded on a full time basis by Mr. K. Singh, P.Eng., Mr. B. Racher, C.E.T, and Mr. P. Khuu. The supervisors logged the boreholes and processed the recovered soil samples and rock cores for transport to Terraprobe's Brampton laboratory for further examination and testing.

The report was written by Mr. Rehman Abdul, P.Eng. and reviewed by Mr. Michael Tanos, P.Eng.





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



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



**FOUNDATION DESIGN REPORT
STORM SEWER INSTALLATIONS
QUEEN ELIZABETH HIGHWAY
FROM BRANT STREET TO BURLOAK DRIVE
MINISTRY OF TRANSPORTATION, ONTARIO
AGREEMENT No. 2006-E-0026, W.P. 2831-02-01
PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

6 GENERAL

This report presents interpretation of the geotechnical data in the factual report and provides geotechnical design recommendations for the proposed storm sewer installations at Walkers Line (Sta. 15+425) and Appleby Line (Sta. 17+609).

The Walkers Line sewer will be installed in a 1050 mm diameter steel casing and will extend below the East Bound and West Bound lanes of the QEW over a distance of about 62.5 m. This crossing is located at Sta. 15+425.

The Appleby Line sewer will be installed in a 900 mm diameter steel casing below the East Bound lanes of the QEW and will extend a distance of about 36.5 m. This crossing is located at Sta. 17+609.

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 OPEN EXCAVATIONS

7.1 General

Discussions and recommendations related to general open cut installation are presented in this section of the report. Recommendations related to tunnelling below the QEW are presented in Section 8.

7.2 Walkers Line Crossing (Sta. 15+425)

7.2.1 Vertical Alignment

Based on the proposed invert elevations along the alignment and the data from the Record of Borehole Logs the trench bottom will lie in stiff silty clay fill and weathered shale bedrock. These soils will generally provide good to excellent support to the sewer. Soft/loose soils that are encountered in the trench bottom will have to be sub-excavated and replaced with compacted granular fill in order to ensure reliable pipe support.



The borehole data indicates that the groundwater table in the soil and bedrock ranges from Elev. 110.5± m to Elev. 112.9± m. Therefore, the groundwater table will generally be above the depth of excavation. This aspect must be taken into consideration when undertaking excavations at this site.

7.2.2 Excavation

The soils described at this site are considered to be suitable for excavation using trenching and excavating equipment, such as backhoes normally used by contractors for this type of utility installation. Excavations should be undertaken in accordance with OPSS 514 and OPSS 515.

Excavations at some sections along the alignment (especially in the area of Borehole S2) will encounter shale bedrock that may require excavation. The bedrock is of the Queenston Formation and is a rippable rock that can be removed with large conventional excavating equipment once it has been displaced by a ripper tooth. Harder layers (e.g. limestone) within the shale matrix that are not feasible or practical to map are normally broken with hoe mounted hydraulic rams prior to excavation.

Till soils inherently contain cobbles and boulders and the contract documents must identify this fact to bidders. The frequency of boulders is unlikely to be high enough to prevent the use of suitable trenching and excavating equipment. However, the contract documents should include a NSSP alerting bidders to the fact that cobbles and boulders and shale bedrock may be encountered. Suggested wording for this NSSP is included in Appendix E.

7.2.3 OHSA Soil Classification

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 1 soils. The silty clay fill may be classified as a Type 3 soil.

Excavations above and below the water table in the overburden soils may be sloped at 1.5H:1V provided unwatering is carried out as described below. Excavations made in rock would be nominally self supporting in a vertical face provided the rock bedding is horizontally oriented.

7.2.4 Groundwater Control

At this site the groundwater table in the bedrock ranges between Elev. 110.5 m and Elev. 111.0 m. The groundwater level in the overburden soils ranges between Elev. 111.2 m and Elev. 112.9 m.

The Contractor must implement suitable groundwater control and ground support systems as required to install the sewer in a safe, stable, unwatered excavation. The design of the unwatering system should be the responsibility of the Contractor.



Groundwater seepage into excavations made through the silty clay fill, silty clay till, till/shale complex and the shale bedrock, should be minimal due to the relatively low permeability of these soils and the rock. It is believed that this seepage can be controlled by gravity drainage and pumping from strategically located filtered sumps as and where required.

7.2.5 Bedding & Backfill

The bedding for the sewer must conform to the requirements of OPSD 802.030, 802.031 and 802.033 as appropriate (rigid pipe) or OPSD 802.010 and OPSD 802.013 (flexible pipe) as appropriate.

It is recommended that the bedding material consist of OPSS Granular "A". Additional bedding requirements that may be imposed by the supplier must also be followed.

Any accumulation of water at the base of the excavation and any soft/loose soils should be removed prior to placing and compacting the pipe bedding. Placement of the pipe bedding must be done in the dry.

The backfill may consist of the excavated soil compacted to 95% SPMDD at a moisture content within $\pm 2\%$ of the optimum value. Shale bedrock is not recommended for use as backfill material. Trenching, backfilling and compaction operations should be in accordance with OPSS 514.

7.3 Appleby Line Crossing (Sta. 17+609)

7.3.1 Vertical Alignment

Based on the proposed invert elevations along the alignment and the data from the Record of Borehole Logs, the trench bottom will lie in hard till/shale complex. This soil will provide excellent support to the sewer. Soft/loose soils that are encountered in the trench bottom will have to be sub-excavated and replaced with compacted granular fill in order to ensure reliable pipe support.

The groundwater table in the overburden soil is estimated to be at Elev. 117 \pm m and will be above the depth of excavation. This aspect must be taken into consideration when undertaking excavations at this site.

7.3.2 Excavation

The soils described at this site are considered to be suitable for excavation using trenching and excavating equipment, such as backhoes normally used by contractors for this type of utility installation. Excavations should be undertaken in accordance with OPSS 514 and OPSS 515.

The shale bedrock exists at relatively shallow depth across this site and excavations may encounter bedrock. The shale bedrock is of the Queenston Formation and is a ripplable rock that can be removed with large conventional excavating equipment once it has been



displaced by a ripper tooth. Harder layers (e.g. limestone) within the shale matrix that are not feasible or practical to map are normally broken with hoe mounted hydraulic rams prior to excavation.

Till soils inherently contain cobbles and boulders and the contract documents must identify this fact to bidders. The frequency of boulders is unlikely to be high enough to prevent the use of suitable trenching and excavating equipment. However, the contract documents should include a NSSP alerting bidders to the fact that cobbles and boulders and shale bedrock may be encountered. Suggested wording for this NSSP is included in Appendix E.

7.3.3 OHSA Soil Classification

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 1 soils. The fill material may be classified as a Type 3 soil.

Excavations above and below the water table in the overburden soils may be sloped at 1.5H:1V provided unwatering is carried out as described below. Any excavations made in rock would be nominally self supporting in a vertical face provided the rock bedding is horizontally oriented.

7.3.4 Groundwater Control

The groundwater table in the overburden soils is estimated at Elev. 117± m. Groundwater will be encountered in the excavations.

The Contractor must implement suitable groundwater control and ground support systems as required to install the sewer in a safe, stable, unwatered excavation. The design of the unwatering system should be the responsibility of the Contractor.

Groundwater seepage into excavations made through the silty clay fill, silty clay till and the till/shale complex, should be minimal due to the relatively low permeability of these soils. It is believed that this seepage can be controlled by gravity drainage and pumping from strategically located filtered sumps as and where required.

7.3.5 Bedding & Backfill

The bedding for the sewer must conform to the requirements of OPSD 802.030, 802.031 and 802.033 as appropriate (rigid pipe) or OPSD 802.010 and OPSD 802.013 (flexible pipe) as appropriate.

It is recommended that the bedding material consist of OPSS Granular "A". Additional bedding requirements that may be imposed by the supplier must also be followed.

Any accumulation of water at the base of the excavation and any soft/loose soils should be removed prior to placing and compacting the pipe bedding. Placement of the pipe bedding must be done in the dry.



The backfill may consist of the excavated soil compacted to 95% SPMDD at a moisture content within $\pm 2\%$ of the optimum value. Shale bedrock is not recommended for use as backfill material. Trenching, backfilling and compaction operations should be in accordance with OPSS 514.

8 SEWER CROSSINGS BELOW QEW (WALKERS LINE AND APPLEBY LINE)

8.1 Walkers Line Crossing (15+425)

A 1050 mm diameter and 62.5 m long steel casing will be installed below the QEW at this site. Based on the vertical alignment of the sewer it is envisaged that the tunnel drive will head from the south side of the QEW to the north side.

The diameter, length and anticipated subsurface conditions limit the range of trenchless installation techniques that would be economically viable at this site. Each method considered has advantages; disadvantages or limitations and these are discussed. The methods that are considered viable are:

1. Horizontal Auger Boring
2. Micro Tunnelling

Tunnelling shall be undertaken in accordance with OPSS 415 and 416. The choice of equipment and the method of tunnelling is the Contractor's responsibility.

8.1.1 Horizontal Auger Boring

Horizontal auger boring requires the use of an auger boring machine that is used to bore horizontally through soil or rock with a cutting head and auger. The auger boring machine can accept many types of cutting attachments ranging from backhoe teeth cutters for excavating soil to small boring units equipped with mini disc cutters for excavating rock. Small boring units can be steered to maintain line and grade. The liner pipe is normally installed by a pipe jacking operation.

Based on the vertical alignment of the sewer it is envisaged that tunnelling will commence from the south to north side of the freeway. The borehole data indicates that the tunnelling operation will be made initially in hard till/shale complex and weathered shale bedrock. As the alignment extends further north the drive will encounter stiff to hard silty clay till and hard till/shale complex. Further north the drive will enter the old Tuck Creek Valley where firm to stiff silty clay fill material will be encountered.

Since mixed face conditions will be encountered at this site the tunnelling contractor must ensure that the tunnelling equipment is suitably designed to deal with these varying conditions while ensuring that proper alignment is maintained during the tunnelling operation.



The silty clay fill, silty clay till, till/shale complex and weathered shale bedrock possesses sufficient cohesion and/or cementation and are expected to have a stand-up time of several hours depending on the composition of the soil matrix. Nevertheless, when tunnelling through these strata the cutting head should follow closely behind the casing to minimize settlements.

When excavation is halted, the casing should be in close contact with the cutting head in order to maintain stability. Ground closure around the liner is expected to be minimal. The application of bentonite slurry under pressure may be required to reduce frictional resistance.

Settlement at the ground surface is estimated to be negligible. This estimate is based on the assumption that the work will be carried out by experienced tunnellers with great care and good workmanship. Under "normal" tunnelling operations, ground loss can be limited to acceptable levels. However, excessive ground loss, and settlement can occur when unusual conditions are encountered.

Possible problems when tunnelling within the silty clay till include boulders and water-bearing sand lenses, which are common in glacial deposits.

The silty clay fill, silty clay till, till/shale complex and shale bedrock are relatively low permeability soils and groundwater seepage is expected to be in small quantities at a slow rate. This seepage can be handled by gravity drainage into the entry shaft from where it could be removed by pumping from filtered sumps.

8.1.2 Microtunnelling

The technique for installing the liner pipe is generally similar to horizontal auger boring i.e. the liner pipe is jacked horizontally into the ground behind a shield. The soil is excavated using a remote controlled cutting head that can be designed to excavate soil and rock.

Microtunnelling is a very precise method of tunnelling and with the suitable choice of cutting tools a wide soil spectrum as well as rock can be excavated. Additionally, there is relatively little settlement with this method if handled properly.

This method is feasible for consideration at this site. However, due to the specialized type of machinery required it might be prohibitively expensive for this relatively short run.

8.2 Appleby Line Crossing (Sta. 17+609)

A 900 mm diameter steel casing will be installed below the QEW. The length of the crossing is approximately 36.5 m extending from Manhole 387 to Manhole 379 below the QEW EBL.

The diameter, length and anticipated subsurface conditions limit the range of trenchless installation techniques that would be economically viable at this site. Each method



considered has advantages; disadvantages or limitations and these are discussed. The methods that are considered viable are:

1. Horizontal Auger Boring
2. Micro Tunnelling

Tunnelling shall be undertaken in accordance with OPSS 415 and 416. The choice of equipment and the method of tunnelling is the Contractor's responsibility.

8.2.1 Horizontal Auger Boring

Horizontal auger boring requires the use of an auger boring machine that is used to bore horizontally through soil or rock with a cutting head and auger. The auger boring machine can accept many types of cutting attachments ranging from backhoe teeth cutters for excavating soil to small boring units equipped with mini disc cutters for excavating rock. Small boring units can be steered to maintain line and grade. The liner pipe is normally installed by a pipe jacking operation.

Based on the borehole data, the casing will be installed in very stiff to hard silty clay till and hard till/shale complex. These soils possess sufficient cohesion and/or cementation and are expected to have a stand-up time of several hours depending on the composition of the soil matrix. Nevertheless, when tunnelling through these strata the cutting head should follow closely behind the casing to minimize settlements.

When excavation is halted, the casing should be in close contact with the cutting head in order to maintain stability. Ground closure around the casing is expected to be minimal. The application of bentonite slurry under pressure may be required to reduce frictional resistance.

Settlement at the ground surface is estimated to be negligible. This estimate is based on the assumption that the work will be carried out by experienced tunnellers with great care and good workmanship. Under "normal" tunnelling operations, ground loss can be limited to acceptable levels. However, excessive ground loss, and settlement can occur when unusual conditions are encountered.

Possible problems when tunnelling within the silty clay till and the till/shale complex include boulders and water-bearing sand lenses, which are common in glacial deposits. A great deal of care is required under these conditions.

The silty clay till and till/shale complex are relatively low permeability soils and groundwater seepage is expected to be in small quantities at a slow rate. This seepage can be handled by gravity drainage into the entry shaft from where it could be removed by pumping from filtered sumps.



8.2.2 Microtunnelling

The technique for installing the liner pipe is generally similar to horizontal auger boring i.e. the liner pipe is jacked horizontally into the ground behind a shield. The soil is excavated using a remote controlled cutting head that can be designed to excavate soil and rock.

Microtunnelling is a very precise method of tunnelling and with the suitable choice of cutting tools a wide soil spectrum and rock can be excavated. Additionally, there is relatively little settlement with this method if handled properly.

This method is feasible for consideration at this site. However, due to the specialized type of machinery required it might be prohibitively expensive for this relatively short run.

9 TUNNEL SUPPORT

In the completed tunnel the maximum residual stress would be expressed in the spring-line of the tunnel diameter where the unbalanced horizontal stress is a maximum. The horizontal and tangential pressure on the permanent tunnel lining is a function of the vertical in situ pressure, which is given by:

$$P_h = \gamma (h - h_w) + \gamma' h_w + h_w \gamma_w$$

γ = bulk unit weight of soil

γ_w = unit weight of water (9.81 kN/m³)

h = depth below surface (m)

h_w = depth below the groundwater level (m)

For design purposes assume a unit weight of 21 kN/m³ for the soil and rock overlying the springline of the tunnel.

10 EARTH PRESSURE

The entry and exit shafts will have to be supported by a shoring system. The shape of the soil pressure distribution diagram behind the shoring system depends upon the type of soil to be encountered and the amount of movement that can be permitted. The sequence of work may also alter the shape of the pressure diagram during the various construction phases.

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Temporary shoring should be designed by a licensed Professional Engineer experienced in shoring design.

Earth pressure computations must also take into account the groundwater level. Above the groundwater level, earth pressure is computed using the bulk unit weight of the retained soil. Below the groundwater level, the earth pressures are computed using the submerged unit weight of the soil. A hydrostatic pressure is also applied if the retained soil is not fully drained.



The appropriate values of the parameters for use in the design of structures subject to unbalanced earth pressures are given in Table 10.1 and Table 10.2.

Table 10.1 – Earth Pressure Coefficients (Walkers Line Crossing Sta. 15+425)

Stratum	ϕ	γ	K_a	K_o	K_p
Fill – Silty Clay	25	18	0.40	0.58	2.46
Silty Clay Till	30	20	0.33	0.50	3.00
Silty Clay Till (Till-Shale Complex)	35	21	0.27	0.43	3.70

Table 10.2 – Earth Pressure Coefficients (Appleby Line Crossing Sta. 17+609)

Stratum	ϕ	γ	K_a	K_o	K_p
Fill – Sand and Gravel	32	21	0.31	0.47	3.25
Fill – Silty Clay	25	18	0.40	0.58	2.46
Silty Clay Till	30	20	0.33	0.50	3.00
Silty Clay Till (Till-Shale Complex)	35	21	0.27	0.43	3.70

The factors in the table above are “ultimate” values and require certain movements for the active and passive 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, 2006.

Flexible shoring should be designed on the basis of the active earth pressure coefficient (K_a). In this case, the performance level should be Level 2 – Angular Distortion 1:200 but shall not be more than 25 mm. Where limited shoring movement (less than performance Level 1) is required, such as the shaft located in the left shoulder of the QEW EBL at the Appleby Line Crossing; the design should be based on the at rest earth pressure coefficient (K_o). For “kick out” design the lateral resistance should be computed on the basis of the passive earth pressure coefficient (K_p).

Where the excavation penetrates the bedrock, the rock excavation is nominally self supporting in a vertical face, provided the rock bedding is horizontally oriented. The rock induces no pressure on shoring systems that require structural support. The requirement for lagging support of partially weathered rock depends on the cleanliness of the excavation break.

Where shoring systems are made perched in the rock above the excavation base, great care and consideration must be given to providing protection and support for the rock in the area of influence directly beneath the base of the soldier pile toe as appropriate. It has become accepted practice in the shoring design community to leave a minimum metre wide shelf to carry soldier pile toes perched above the level of the excavation base.

Soldier piles socketted into the sound bedrock can be designed based on a lateral resistance of 1 MPa.



11 BASAL STABILITY

Tunnelling will require the construction of entry and exit shafts on both alignments. The borehole data shows that the excavation bases will be made in firm to stiff silty clay fill, hard silty clay till, hard till/shale complex and shale bedrock. The base of the excavations will be stable with respect to bottom heave.

12 MONITORING

The contract documents should require the contractor to monitor the roadway surface before, during and after the trenchless installation. A precondition survey is also required prior to tunnelling. A recommended settlement monitoring guideline is included in Appendix F.

It is also necessary to check the amount of spoil removal to determine if there is over excavation and if there are any possible voids outside of the casing. Voids must be grouted with approved grouting materials using approved methods.

13 CONSTRUCTION CONCERNS

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

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

- the possibility of encountering boulders or other obstructions in the overburden soils when tunnelling.
- the possibility of encountering limestone layers in the till/shale complex and shale bedrock which can potentially cause problems with vertical control during tunnelling.
- the potential for groundwater levels to be higher at the time of construction than those recorded in this report.



Rehman Abdul



Engineering Analysis and Report Preparation by:
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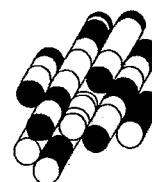
Michael Tanos

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 is 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
$\epsilon_1, \epsilon_2, \epsilon_3$	%	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_r	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ² /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p) / I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ²	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

EXPLANATION OF CORE LOG TERMS

Column Number

1. Elevation of borehole collar.
2. Depth of geotechnical boundary in borehole
3. Geologic symbol for rock or soil material
4. General description of geotechnical unit - qualitative description, including rock type(s), percentage rock types, frequency and sizes of interbeds, colour, texture.

Joint (discontinuity) Characteristics

5. Number of joint sets: a rock mass can be intersected by a number of joint sets of varying orientations.
6. Joint type: B = Bedding joint C = Cross joint
7. Orientation: only variations in dip can be identified in core; dip direction is from field mapping or oriented core:
 F = Flat = 0 - 20° D = Dipping = 20 - 50° V = Vertical = 50 - 90°
8. Joint spacing: this is an approximate measure of spacing between joints in specific joint sets.

SPACING	>3 m	1 m - 3 m	0.3 m - 1 m	50 mm - 300 mm	<50 mm
	VERY WIDE	WIDE	MODERATE	CLOSE	VERY CLOSE

9. Roughness:

RU = Rough Undulating RP = Rough Planar
 SU = Smooth Undulating SP = Smooth Planar
 LU = Slickensided Undulating LP = Slickensided Planar

10. Filling:

Approximate ϕ

T = Tight, hard, non-softened
 O = Oxidation surface staining only 25 - 35
 SA = Slightly altered; clay-free 25 - 30
 S = Sandy particles; clay-free 25 - 30
 Si = Sandy and silty, minor clay 20 - 25
 NC = Non-softening Clays; 5mm 16 - 24
 SC = Swelling Clay fillings; 5mm 6 - 12

11. Aperture: estimated size of joint opening.

12. Degree of weathered rock material:

DEGREE	DESCRIPTION				
UNWEATHERED	NO SIGNS OF DISCOLOURATION OR OXIDIZATION				
SLIGHTLY WEATHERED	PARTIAL DISCOLOURATION; FRACTURES (JOINTS), TYPICALLY OXIDIZED				
MODERATELY WEATHERED	TOTAL DISCOLOURATION				
HIGHLY WEATHERED	TOTAL DISCOLOURATION; TYPICALLY FRIABLE AND PITTED				
COMPLETELY WEATHERED	RESEMBLES A SOIL; ROCK STRUCTURE - USUALLY PRESERVED				

13. Strength of rock material:

		MPa				
VERY HIGH STRENGTH	SPECIMEN CAN ONLY BE CHIPPED BY GEOLOGICAL HAMMER	> 200				
HIGH STRENGTH	SPECIMEN REQUIRES A NUMBER OF BLOWS OF A GEOLOGICAL HAMMER TO FRACTURE IT; CANNOT BE SCRAPED WITH POCKET KNIFE	50 - 200				
MEDIUM STRENGTH	SPECIMEN CANNOT BE FRACTURED BY A SINGLE, FIRM BLOW OF GEOLOGICAL HAMMER; CAN BE SCRAPED WITH POCKET KNIFE, NOT PEELED	15 - 50				
LOW STRENGTH	SHALLOW INDENTATIONS MADE BY FIRM BLOW WITH POINT OF GEOLOGICAL HAMMER; CAN BE PEELED WITH POCKET KNIFE WITH DIFFICULTY	4 - 15				
VERY LOW STRENGTH	CRUMBLES UNDER FIRM BLOW WITH POINT OF GEOLOGICAL HAMMER; CAN BE PEELED	1 - 4				

14. Fracture frequency: number of natural joints occurring over a meter length of core. All natural joints are counted irrespective of the number of joint sets.

FRACTURE FREQUENCY	JOINT SPACING	LENGTH				
0.3m	VERY WIDE	> 3m				
0.3 - 1m	WIDE	1m - 3m				
1 - 3m	MODERATE	0.03m - 1m				
3 - 20m	CLOSE	0.005m TO 0.03m				
20m	VERY CLOSE	< 0.005m				

15. Run number and Total Core Recovery

- (i) Drill run number
- (ii) Total Core Recovery is the total length of core pieces, irrespective of their individual lengths obtained in a core run, and expressed as a percentage of the length of that core run.

16. Rock Quantity Designation (RQD): The total length of those pieces of sound core which are 0.01 metres or greater in length in a core run, expressed as a percentage of the total length of that core run. Sound pieces of rock are those pieces separated by natural breaks and not machine breaks or subsequent artificial breaks.

Rock Mass Classification (after Deere)

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
DESCRIPTION	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

17. Core and Casing sizes: changes of core and casing sizes are indicated.

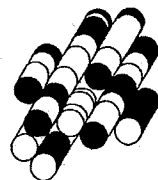
18. Water recovery, level and tests:

- (i) percentage drill water recovery
- (ii) water level depth
- (iii) positions and results of tests, e.g., permeability and packer tests

APPENDIX A

Record of Borehole Sheets, Core Logs and Core Photos

Terraprobe Limited



RECORD OF BOREHOLE No S1

1 OF 1

METRIC

W.P. 2831-02-01 LOCATION Coords: N:4802943.9 E:281382.9 ORIGINATED BY PK
DIST HWY QEW BOREHOLE TYPE Solid Stem Augers & NQ Coring COMPILED BY DB
DATUM Geodetic DATE 25.01.09 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)	
								○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL						× LAB VANE	
114.8	Ground Surface							20	40	60	80	100					GR SA SI CL		
0.0 114.5 0.3	280mm TOPSOIL		1	SS	6														
	FILL - Silty Clay, trace sand to sandy, trace to some gravel, trace rootlets, occasional cobbles, occasional shale inclusions, firm to stiff, brown, damp to moist		2	SS	10		114										13 34 34 19		
			3	SS	9		113												
	topsoil stained		4	SS	10		112												
			5	SS	8												1 21 45 33		
111.1 3.7 110.7 4.1	SILTY CLAY TILL - with shale, hard, reddish brown, dry (TILL-SHALE COMPLEX)		6	SS	147/ 20cm		111												
	SHALE BEDROCK		1	RUN	NQ		110										RUN#1 TCR=100% RQD=39%		
	Dark reddish brown, slightly to moderately weathered to 7.0m, then unweathered, thinly to medium bedded, occasionally laminated, low to medium strength, with interbeds of light greenish grey to light grey, medium to high strength limestone / dolostone.		2	RUN	NQ		109										RUN#2 TCR=100% RQD=74%		
	(Queenston Formation)		3	RUN	NQ		108										RUN#3 TCR=100% RQD=39%		
107.0 7.8	End of Borehole																		
	Commence rock coring at 4.4m. See Core Log S1 for detailed information.																		
	Piezometer Installation consists of 19mm diameter, schedule 40 PVC pipe with a 1.52m slotted screen.																		
	Water Level Readings (P1):																		
	Date Depth(m) Elevation(m)																		
	Jan.30.09 4.3 110.5																		
	Feb.03.09 4.3 110.5																		
	Water Level Readings (P2):																		
	Date Depth(m) Elevation(m)																		
	Jan.30.09 3.4 111.4																		
	Feb.03.09 3.6 111.2																		

ONTARIO MOT 1-09-4007 QEW SANITARY SEWER.GPJ ONTARIO MOT.GDT 11/02/09

CORE LOG



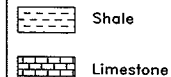
Project	QEW Sanitary Sewer	Orientation	Vertical	Ground Elevation	114.8m	Datum	Geodetic	Borehole No.	S1
Location	Burlington, Ontario	Date Started	January 26, 2009	Completed	January 26, 2009	Logged By	P.K.	Sheet	1 of 1
Client	Ministry of Transportation, Ontario	Drilling Agency	GeoEnvironmental	Drill Type	Bombardier	Core Barrel & Bit Design	NQ	Project No.	1-09-4007

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics								WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (kN/m ³)	
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE										
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19		
	4.0		Overburden, see Borehole Log S1																	
110.4	4.4		QUEENSTON FORMATION – Shale with interbedded limestone. Shale Reddish brown, slightly to moderately weathered to 7.0m (Elev. 107.8m), then unweatherd, thinly to medium bedded, occasionally laminated, low to medium strength. Vertical joint occurs at: 4.7m (Elev. 110.1m). High frequency fractures occur at: 6.5–7.0m (Elev. 108.3–107.8m) 7.2–7.4m (Elev. 107.6–107.4m) Limestone/Dolostone Light greenish grey to light grey, laminated to thinly bedded, medium to high strength. Run 1 Shale = 100% Run 2 Shale = 85% Limestone = 15% Run 3 Shale = 92% Limestone = 8%	2	BC	FD	VC	SP	NC	0 to 1					#1	100%	39%	NQ		
110.3	4.5			2	BC	FV	VC	SP	T							#2				
109.8	5.0			2	BC	FV	VC	SP	NC							100%	74%			
109.3	5.5			2	BC	FVD	VC	SP	T	0 to 1								NQ	44.1	21.3
				1	B	F	VC	SP	T											
108.8	6.0			2	BC	FV	VC	SP	NC											
108.3	6.5			2	BC	FV	VC	SP	NC											
				2	BC	FVD	VC	RP	T	0 to 1									NQ	
107.8	7.0			2	BC	FVD	VC	SP	NC											
107.3	7.5			2	BC	FVD	VC	SP	T											
107.0	7.8			1	B	F	VC	SP	T											
106.8	8.0		End of Core Log																	
106.3	8.5																			
105.8	9.0																			
105.3	9.5																			
104.8	10.0																			

Remarks:

Only limestone layers thicker than 25mm are reported in column 3.

LEGEND:



RECORD OF BOREHOLE No S2

1 OF 1

METRIC

W.P. 2831-02-01 LOCATION Coords: N:4802901.9 E:281438.7 ORIGINATED BY PK
DIST HWY QEW BOREHOLE TYPE Solid Stem Augers & NQ Coring COMPILED BY DB
DATUM Geodetic DATE 27.01.09 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
114.9	Ground Surface							20 40 60 80 100						
0.0 114.7	230mm TOPSOIL							20 40 60 80 100						
0.2	SILTY CLAY sandy, trace gravel, stiff to hard, brown, damp to moist (GLACIAL TILL)		1	SS	12		114						23.0	3 32 42 23
113.9			2	SS	34		114							
1.0	SILTY CLAY TILL - with shale and limestone inclusions, hard, reddish brown, dry (TILL-SHALE COMPLEX)		3	SS	117		113							14 21 52 13
113.1			4	SS	100/ 13cm		112							
1.8	SHALES BEDROCK Dark reddish brown, moderately to highly weathered to 4.1m, then unweathered, thinly to medium bedded, occasionally laminated, low to medium strength, with interbeds of light greenish grey, medium to high strength limestone / dolostone. (Queenston Formation)		1	RUN	NQ		111							RUN#1 TCR=100% RQD=51%
			2	RUN	NQ		110							RUN#2 TCR=100% RQD=82%
			3	RUN	NQ		109							RUN#3 TCR=100% RQD=69%
			4	RUN	NQ		108							RUN#4 TCR=100% RQD=68%
107.2	End of Borehole													
7.8	Commence rock coring at 2.9m. See Core Log S2 for detailed information. Piezometer installation consists of 19mm diameter, schedule 40 PVC pipe with a 1.52m slotted screen. Water Level Readings (P1): Date Depth(m) Elevation(m) Jan.30.09 3.9 111.0 Feb.03.09 4.3 110.6 Water Level Readings (P2): Date Depth(m) Elevation(m) Jan.30.09 2.1 112.8 Feb.03.09 2.0 112.9													

ONTARIO MOT 1-09-4007 QEW SANITARY SEWER.GPJ ONTARIO MOT.GDT 11/02/09

CORE LOG



Terraprobe

Project	QEW Sanitary Sewer	Orientation	Vertical	Ground Elevation	114.9m	Datum	Geodetic	Borehole No.	S2
Location	Burlington, Ontario	Date Started	January 27, 2009	Completed	January 27, 2009	Logged By	P.K.	Sheet	1 of 1
Client	Ministry of Transportation, Ontario	Drilling Agency	GeoEnvironmental	Drill Type	Bombardier	Core Barrel & Bit Design	NQ	Project No.	1-09-4007

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics							WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO.	CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa	UNIT WEIGHT (kN/m ³)
				NO. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE									
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
	2.5		Overburden, see Borehole Log S2																
112.0	2.9		QUEENSTON FORMATION – Shale with interbedded limestone. <u>Shale</u> Reddish brown, moderately to highly weathered to 4.1m (Elev. 110.8m), then unweathered, thinly to medium bedded, occasionally laminated, low to medium strength. High frequency fractures occur at: 3.5–3.9m (Elev. 111.4–111.0m) <u>Limestone/Dolostone</u> Light greenish grey to light grey, laminated to thinly bedded, medium to high strength. <u>Run 1</u> Shale = 85% Limestone = 15% <u>Run 2</u> Shale = 98% Limestone = 2% <u>Run 3</u> Shale = 65% Limestone = 35% <u>Run 4</u> Shale = 80% Limestone = 20%	1	B	F	C	SP	NC	0 to 1				#1					
111.9	3.0			1	B	F	VC	SP	NC		1				100%	51%	NQ		
111.4	3.5				2	BC	FV	VC	SP	NC					#2			8.3	23.0
110.9	4.0				1	B	F	VC	SP	NC	0 to 1				100%	82%			
					2	BC	FV	VC	SP	NC							NQ		
110.4	4.5				1	B	F	VC	SP	T									
109.9	5.0														#3				
															100%	69%			
109.4	5.5				1	B	F	VC	SP	T	0 to 1						NQ		
108.9	6.0																		
108.4	6.5													#4					
														100%	68%				
107.9	7.0			1	B	F	VC	SP	T	0 to 1						NQ			
107.4	7.5																		
107.1	7.8																		
106.9	8.0		End of Core Log																
106.4	8.5																		

Remarks:

Only limestone layers thicker than 25mm are reported in column 3.

LEGEND:



Shale



Limestone

RECORD OF BOREHOLE No HML-3

1 OF 1

METRIC

W.P. 2831-02-01 LOCATION Coords: N:4802915.0 E:281399.1 ORIGINATED BY SK
 DIST HWY QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY DB
 DATUM Geodetic DATE 01.08.07 - 02.08.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
								● QUICK TRIAXIAL	× LAB VANE						
115.7	Ground Surface														
0.0 115.4	280mm ASPHALT														
0.3	FILL - Gravelly Sand, some silt, compact, brown, dry		1	SS	29										
114.7			2	SS	23		115								
1.0	FILL - Silty Clay, trace to some sand, trace gravel, firm to stiff, brown, damp to moist		3	SS	10		114	75kPa						6 21 50 23	
			4	SS	7		113	100kPa							
	trace shale fragments		5	SS	7		112	125kPa							
							111	125kPa						20 27 38 15	
110.8 4.9	reddish brown		6	SS	65		110								
	SHALE BEDROCK reddish brown (Queenston Formation)		7	SS	100/ 10cm		109								
			8	AS	-		108								
106.5 9.2	End of Borehole		9	SS	100/ 2.5cm		107								
	Auger Refusal at 9.2m Borehole was open and unstabilized water level at 8.5m upon completion of drilling.														

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

ONTARIO MOT 1-09-4007 QEW SANITARY SEWER GPJ ONTARIO MOT.GDT 11/02/09

RECORD OF BOREHOLE No HML-3A

1 OF 1

METRIC

W.P. 2831-02-01 LOCATION Coords: N:4802915.0 E:281399.1 ORIGINATED BY HA
 DIST HWY QEW BOREHOLE TYPE Solid Stem Augers & NQ Coring COMPILED BY DB
 DATUM Geodetic DATE 04.10.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
115.7 0.0	Ground Surface						20	40	60	80	100									
	Augered to 5.6m, refer to BH HML-3 for inferred soil stratigraphy.																			
110.1 5.6	SHALE BEDROCK																			
	Reddish brown, slightly weathered to 7.3m, then unweathered, medium to thickly bedded, low to medium strength shale with occasional interbeds of medium to high strength greenish grey limestone. Irregular, stained subvertical joints at 7.3, 8.2m.																			
	Shale = 80% Limestone = 20% (Queenston Formation)																			
			1	RUN	NQ															
			2	RUN	NQ															

ONTARIO MOT 1-09-4007 QEW SANITARY SEWER GPJ ONTARIO MOT GDT 11/02/09

CORE LOG



Terraprobe

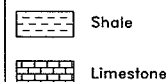
Project	QEW - Bront Street to Burlock Drive Agreement No. 2006-E-0026: W.P. 2831-02-01	Orientation	Vertical	Ground Elevation	115.7m	Datum	Geodetic	Borehole No.	HML-3A
Location	Burlington, Ontario	Date Started	October 4, 2007	Completed	October 4, 2007	Logged By	H.A.	Sheet	1 of 1
Client	Ministry of Transportation, Ontario	Drilling Agency	DBW	Drill Type	Truck-Mount	Core Barrel & Bit Design	NQ	Project No.	1-09-4007

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics								WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO.	CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (kN/m³)		
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE												
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19				
110.7	5.0		Overburden, see Borehole Log HML-3A																			
110.2	5.5																					
110.1	5.6		QUEENSTON FORMATION – Shale with interbedded limestone. Shale Reddish brown, slightly weathered to 7.3m (Elev. 108.4m), then unweatherd, thinly to medium bedded, occasionally laminated, low to medium strength. Multiple vertical and subvertical joints occur at: 7.8–8.4m (Elev. 107.9–107.3m). High frequency fractures occur at: 6.2m (Elev. 109.5m) Limestone/Dolostone Light greenish grey to light grey, laminated to thinly bedded, medium to high strength. Run 1 Shale = 73% Limestone = 27% Run 2 Shale = 92% Limestone = 8%	2	BC	FVD	VC	RP	NC					#1	98%	15%	NQ	17.8	27.6			
108.7	6.0			2	BC	FV	VC	RP	T													
109.2	6.5			1	B	F	VC	RP	T	0 to 1												
				2	BC	FV	VC	RP	NC													
108.7	7.0			2	BC	FV	VC	RP	T													
				1	B	F	VC	SP	T													
				2	BC	FV	VC	RP	NC													
108.2	7.5														#2							
107.7	8.0				2	BC	FV	VC	RP	T	0 to 1				100%	49%						
																				NQ		
107.2	8.5				2	BC	FVD	VC	SP	T												
106.8	8.9				1	B	F	VC	SP	T												
106.7	9.0		End of Core Log																			
106.2	9.5																					
105.7	10.0																					
105.2	10.5																					
104.7	11.0																					

Remarks:

Only limestone layers thicker than 25mm are reported in column 3.

LEGEND:



RECORD OF BOREHOLE No S3

1 OF 1

METRIC

W.P. 2831-02-01 LOCATION Coords: N:4804619.9 E:282779.9 ORIGINATED BY KS
 DIST HWY QEW BOREHOLE TYPE Solid Stem Augers & NQ Coring COMPILED BY DB
 DATUM Geodetic DATE 25.01.09 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							× LAB VANE
118.8	Ground Surface					20	40	60	80	100					GR SA SI CL		
0.0	130mm ASPHALT																
0.1	FILL - Sand and Gravel, silty, very dense, brown, moist		1	SS	86												
118.4	FILL - Silty Clay, trace sand, trace gravel, with shale inclusions, very stiff to hard, brown, damp to moist		2	SS	69										1 7 72 20		
0.4	topsoil stained		3	SS	24												
117.0	SILTY CLAY - trace sand, trace gravel, very stiff to hard, brown, damp (GLACIAL TILL)																
116.4			4	SS	118/ 22cm										1 16 66 17		
2.4	SILTY CLAY TILL with shale, some sand, hard, reddish brown, dry to damp (TILL-SHALE COMPLEX)		5	SS	100/ 14cm												
			6	SS	30												
114.6			7	SS	100/ 11cm												
4.2	SHALE BEDROCK Dark reddish brown, unweathered, thinly to medium bedded, occasionally laminated, low to medium strength, with interbeds of light greenish grey, medium to high strength limestone / dolostone. (Queenston Formation)		1	RUN	NQ										RUN#1 TCR=99% RQD=77%		
			2	RUN	NQ										RUN#2 TCR=100% RQD=80%		
111.0	End of Borehole																
7.8	Borehole was open and dry upon completion of drilling. Commence rock coring at 4.7m. See Core Log S3 for detailed information.																

ONTARIO MOT 1-09-4007 QEW SANITARY SEWER GPJ ONTARIO MOT.GDT 11/02/09

ONTARIO MOT 1-09-4007 QEW SANITARY SEWER.GPJ ONTARIO MOT.GDT 11/02/09

CORE LOG



Terraprobe

Project	QEW Sanitary Sewer	Orientation	Vertical	Ground Elevation	118.8m	Datum	Geodetic	Borehole No.	S3
Location	Burlington, Ontario	Date Started	January 26, 2009	Completed	January 26, 2009	Logged By	K.S.	Sheet	1 of 1
Client	Ministry of Transportation, Ontario	Drilling Agency	GeoEnvironmental	Drill Type	Bombardier	Core Barrel & Bit Design	NQ	Project No.	1-09-4007

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics							WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (kN/m³)	
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE									
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
114.8	4.0		Overburden, see Borehole Log S3																
114.3	4.5																		
114.0	4.8		QUEENSTON FORMATION – Shale with interbedded limestone. <u>Shale</u> Reddish brown, unweathered, thinly to medium bedded, occasionally laminated, medium strength. Multiple vertical and subvertical joints occur at: 4.8–6.0m (Elev. 114.0–112.8m). 6.3–6.5m (Elev. 112.5–112.3m). <u>Limestone/Dolostone</u> Light greenish grey to light grey, laminated to thinly bedded, high strength. <u>Run 1</u> Shale = 93% Limestone = 7% <u>Run 2</u> Shale = 79% Limestone = 21%	2	BC	FVD	VC	SP	T					#1					
113.8	5.0			1	B	F	VC	SP	T					99%	77%				
113.3	5.5			2	BC	FVD	VC	SP	T	0 to 1							NQ	21.4	27.9
112.8	6.0			1	B	F	VC	SP	T										
112.3	6.5			2	BC	FVD	VC	SP	T						#2				
111.8	7.0			1	B	F	VC	SP	T	0 to 1							NQ		
111.3	7.5														100%	80%			
111.0	7.8																		
110.8	8.0			End of Core Log															
110.3	8.5																		
109.8	9.0																		
109.3	9.5																		
108.8	10.0																		

Remarks:

Only limestone layers thicker than 25mm are reported in column 3.

LEGEND:

	Shale
	Limestone
	Alternating Shale and Limestone Layers

RECORD OF BOREHOLE No S4

1 OF 1

METRIC

W.P. 2831-02-01 LOCATION Coords: N:4804600.0 E:282809.0 ORIGINATED BY BR
 DIST HWY QEW BOREHOLE TYPE Solid Stem Augers & NQ Coring COMPILED BY DB
 DATUM Geodetic DATE 27.01.09 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
117.8	Ground Surface																GR SA SI CL			
0.0	FILL - Silty Clay, sandy, some gravel, topsoil stained, stiff, moist		1	SS	12									○			15 23 46 16			
117.1																				
0.7	SILTY CLAY - some sand, trace gravel, occasional shale inclusions, hard, brown, damp to moist (GLACIAL TILL)		2	SS	43									○			8 12 57 23			
116.4																				
1.4	SILTY CLAY TILL - with shale, hard, reddish brown, dry to damp (TILL-SHALE COMPLEX)		3	SS	58									○						
115.5																				
2.3	SHALE BEDROCK Dark reddish brown, moderately weathered to 5.0m, then unweathered, thinly to medium bedded, occasionally laminated, low to medium strength, with interbeds of light greenish grey to light grey, medium to high strength limestone / dolostone. (Queenston Formation)		4	SS	100/ 15cm									○						
			5	SS	100/ 15cm									○						
			6	SS	100/ 15cm									○						
			7	SS	100/ 10cm															
			1	RUN	NQ												RUN#1 TCR=100% RQD=46%			
			2	RUN	NQ												RUN#2 TCR=100% RQD=93%			
110.1																				
7.8	End of Borehole Commence rock coring at 4.7m. See Core Log S4 for detailed information. Piezometer Installation consists of 19mm diameter, schedule 40 PVC pipe with a 1.52m slotted screen. Water Level Readings (P1): Date Depth(m) Elevation(m) Jan.30.09 3.2 114.6 Feb.03.09 3.1 114.7 Water Level Readings (P2): Date Depth(m) Elevation(m) Jan.30.09 2.6 115.2 Feb.03.09 3.1 114.7																			

ONTARIO MOT 1-08-4007 QEW SANITARY SEWER.GPJ ONTARIO MOT.GDT 11/02/09

CORE LOG



Terraprobe

Project	QEW Sanitary Sewer	Orientation	Vertical	Ground Elevation	117.8m	Datum	Geodetic	Borehole No.	S4
Location	Burlington, Ontario	Date Started	January 26, 2009	Completed	January 26, 2009	Logged By	K.S.	Sheet	1 of 1
Client	Ministry of Transportation, Ontario	Drilling Agency	GeoEnvironmental	Drill Type	Bombardier	Core Barrel & Bit Design	NQ	Project No.	1-09-4007

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics								WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (kN/m³)
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE									
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
113.8	4.0																		
113.3	4.5		Overburden, see Borehole Log S4																
113.1	4.7																		
112.8	5.0		QUEENSTON FORMATION – Shale with interbedded limestone.	2	BC	FVD	VC	RP	T					#1					
			Shale Reddish brown, moderately weathered to 5.0m (Elev. 112.8m), then unweathered, thinly to medium bedded, occasionally laminated, low to medium strength.	1	B	F	VC	SP	T					100%	46%				
112.3	5.5		High frequency fractures occur at: 4.8m (Elev. 113.0m) 5.8m (Elev. 112.0m)	2	BC	FVD	VC	RP	T	0 to 1						NQ	22.7	26.4	
				2	BC	FV	VC	RP	T										
111.8	6.0			1	B	F	VC	SP	T										
			Limestone/Dolostone Light greenish grey to light grey, laminated to thinly bedded, medium to high strength.																
111.3	6.5													#2					
			Run 1 Shale = 86% Limestone = 14%	1	B	F	VC	SP	T	0 to 1				100%	93%				
110.8	7.0		Run 2 Shale = 73% Limestone = 27%													NQ			
110.3	7.5																		
110.0	7.8																		
109.8	8.0		End of Core Log																
109.3	8.5																		
108.8	9.0																		
108.3	9.5																		
107.8	10.0																		

Remarks:

Only limestone layers thicker than 25mm are reported in column 3.

LEGEND:



Shale



Limestone

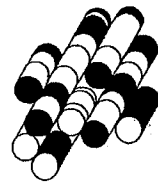


Alternating Shale and Limestone Layers

APPENDIX B

Laboratory Test Results

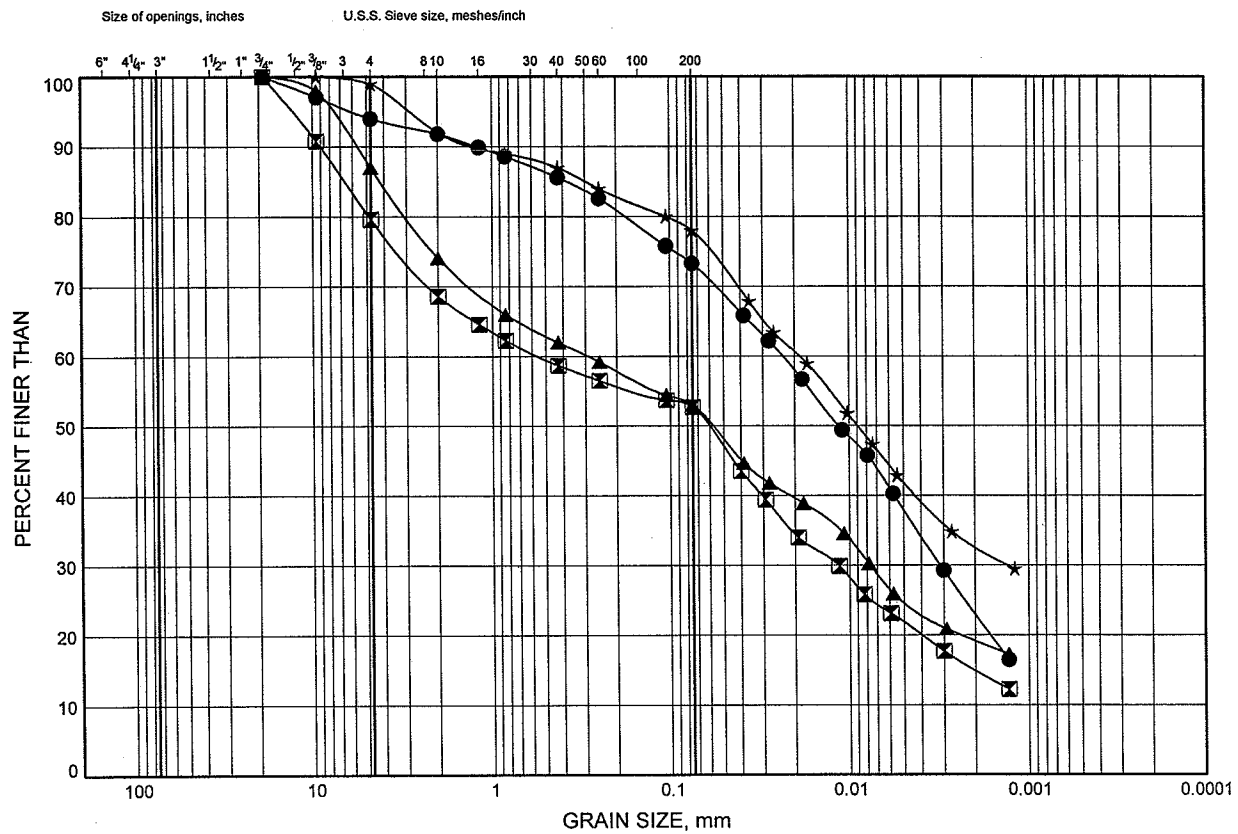
Terraprobe Limited



GRAIN SIZE DISTRIBUTION

FIGURE B1

FILL - Silty Clay

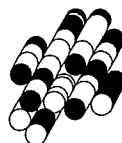


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	HML-3	1.7	114.0
□	HML-3	4.7	111.0
▲	S1	1.0	113.8
★	S1	3.2	111.6

Date February 2009.....

Project 2831-02-01...



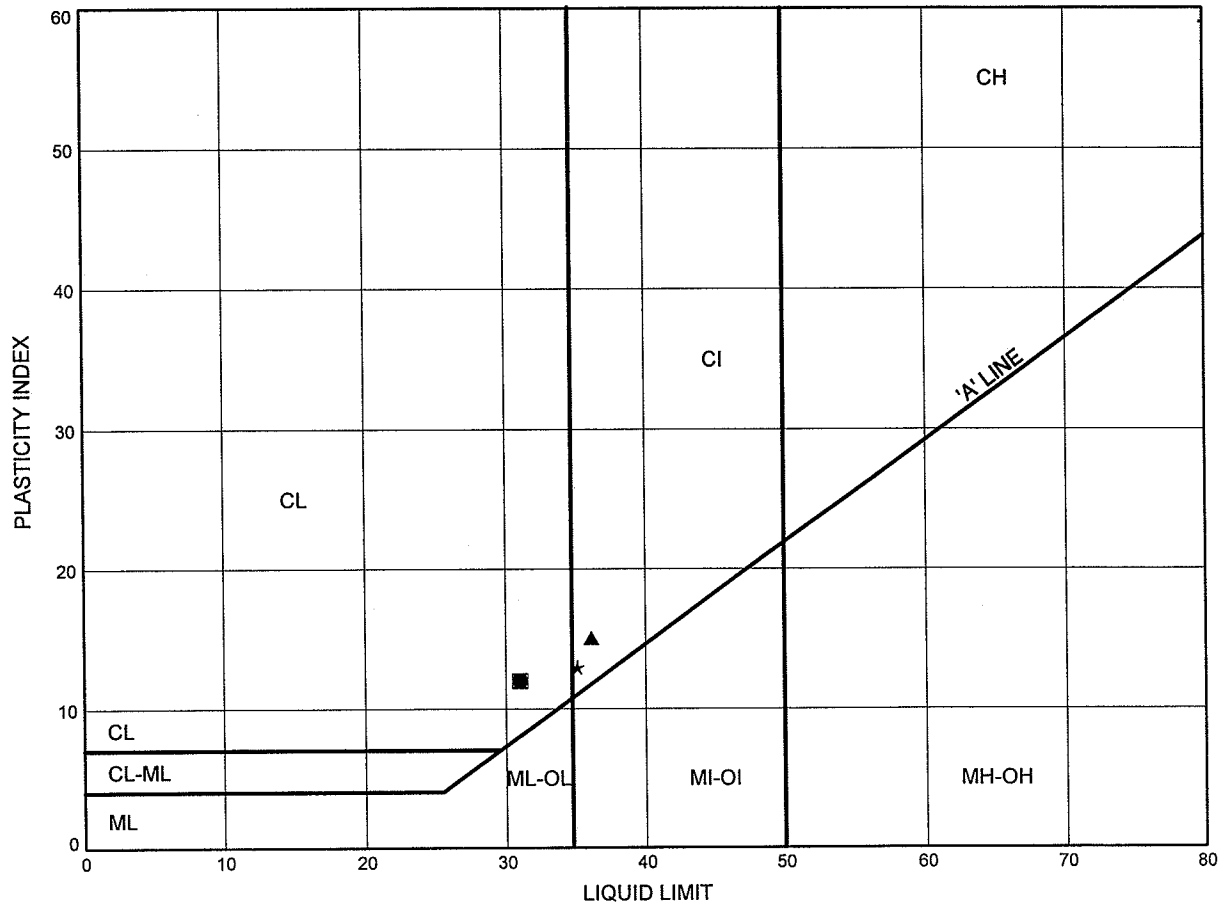
Prep'd DB.....

Chkd. RA.....

ATTERBERG LIMITS TEST RESULTS

FIGURE B2

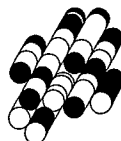
FILL - Silty Clay



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	HML-3	1.7	114.0
⊠	HML-3	4.7	111.0
▲	S1	1.0	113.8
★	S1	3.2	111.6

Date February 2009

Project 2831-02-01



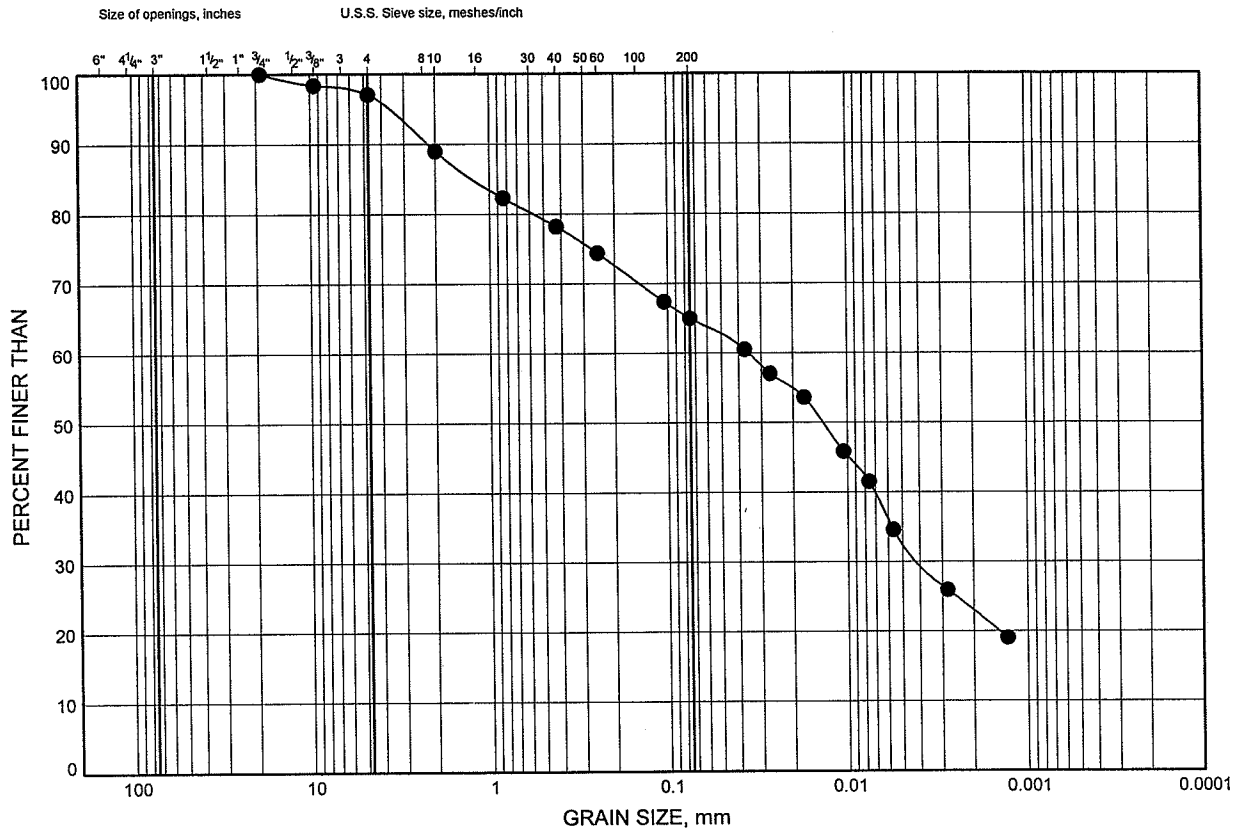
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B3

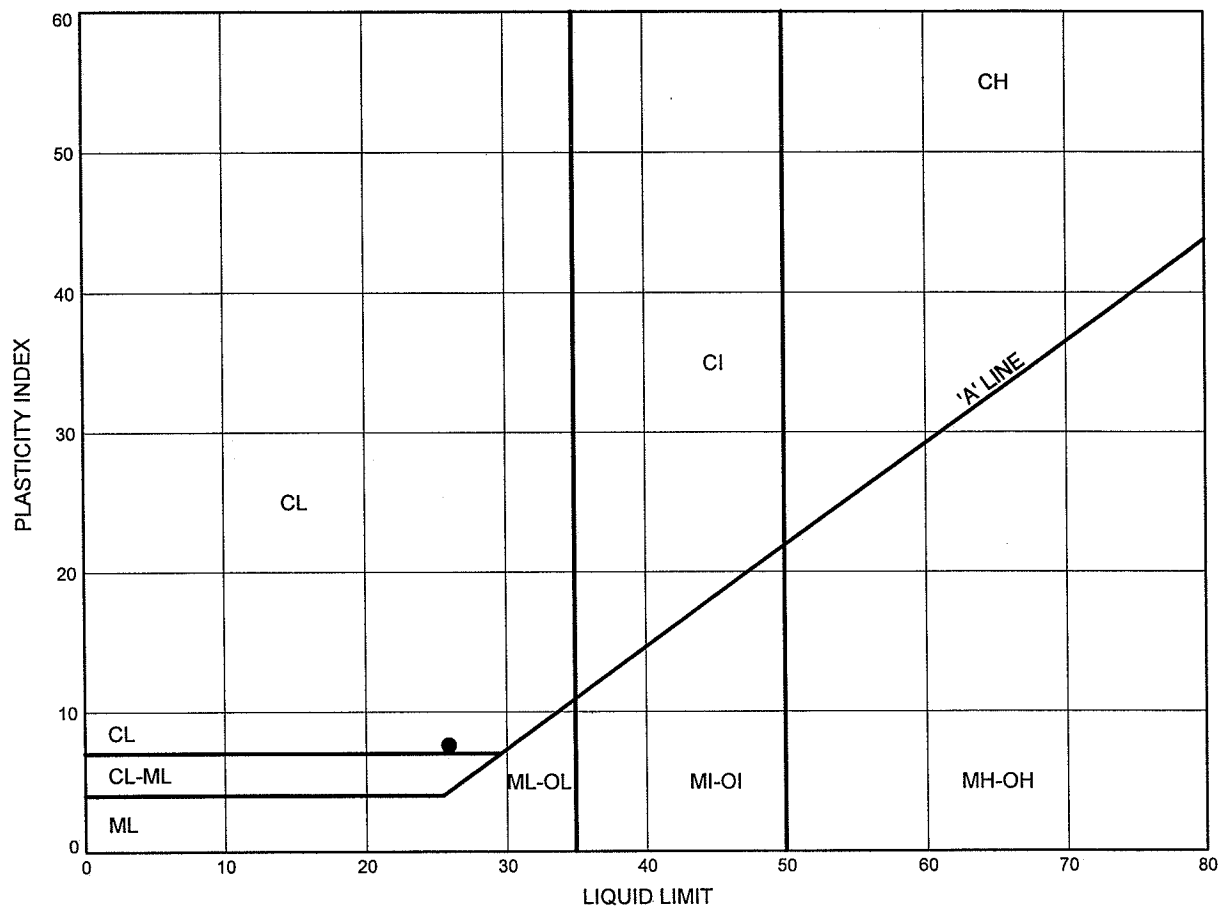
SILTY CLAY TILL



ATTERBERG LIMITS TEST RESULTS

FIGURE B4

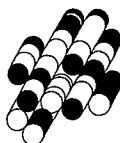
SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	S2	0.3	114.6

Date February 2009

Project 2831-02-01



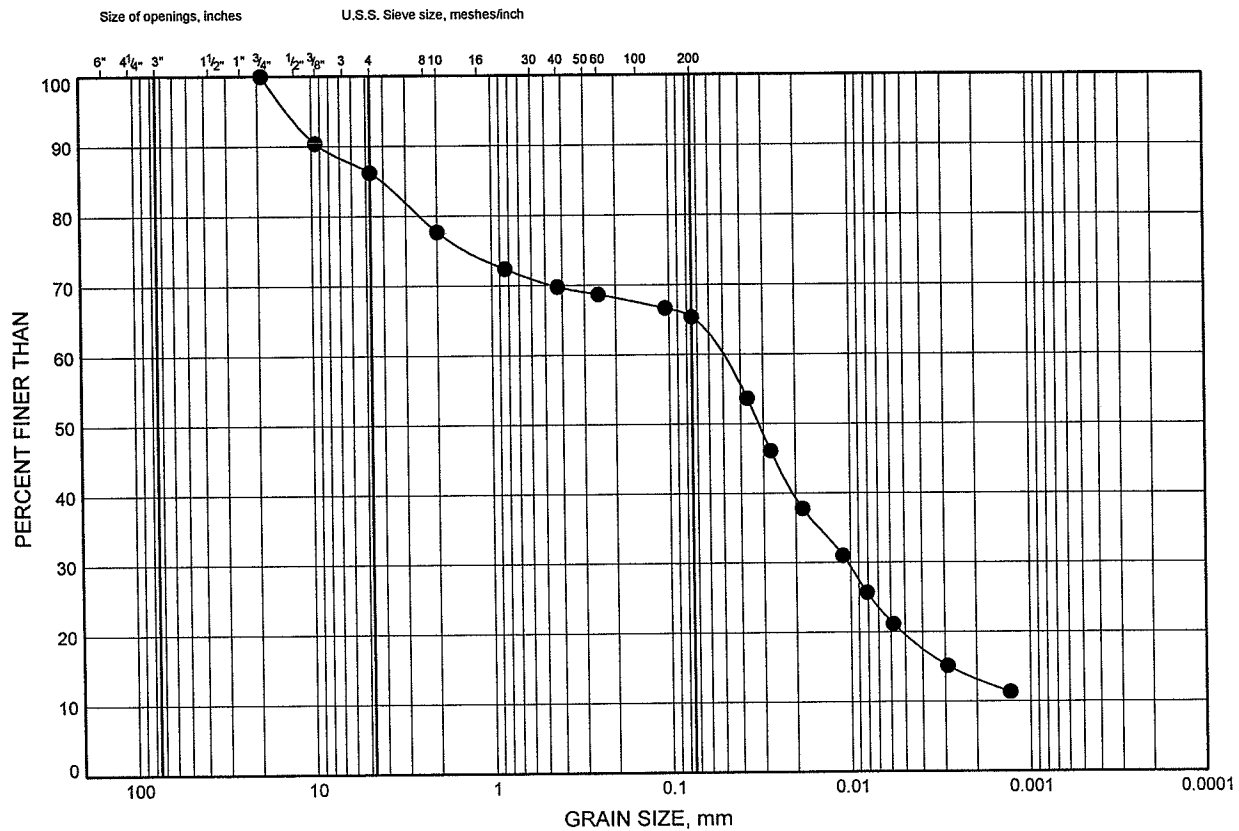
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B5

TILL SHALE COMPLEX

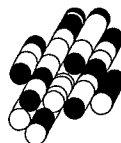


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	S2	1.6	113.3

Date February 2009

Project 2831-02-01



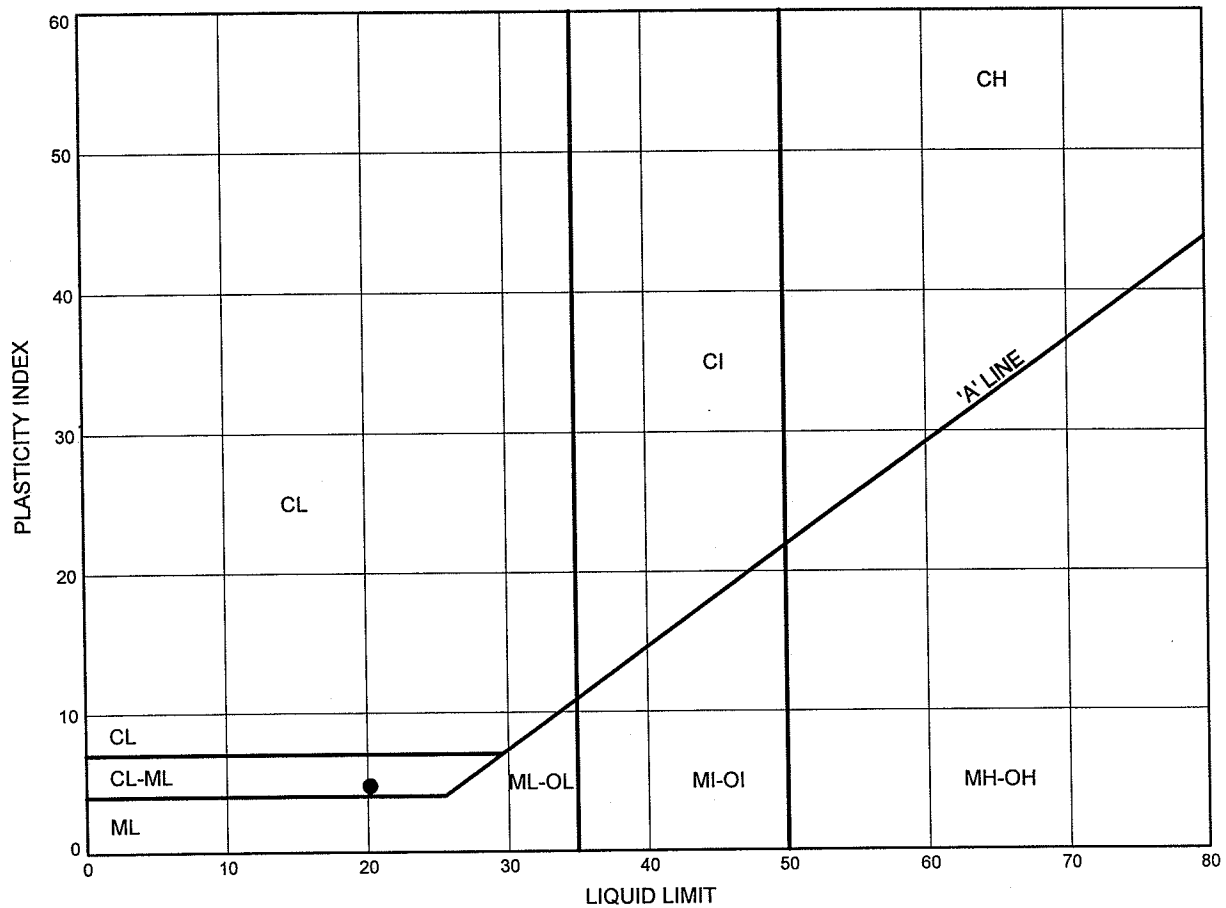
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

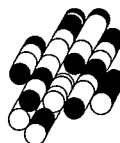
TILL SHALE COMPLEX



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	S2	1.6	113.3

Date February 2009

Project 2831-02-01



Prep'd DB

Chkd. RA

FIGURE B7

Size of openings, inches

U.S.S. Sieve size, meshes/inch

PERCENT FINER THAN

GRAIN SIZE, mm

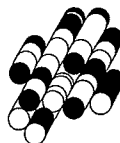
Grain Size (mm)	Percent Finer Than (Solid Circles)	Percent Finer Than (Open Squares)
100	100	100
10	100	92
5	100	85
2	99	79
1	98	73
0.5	97	68
0.25	96	64
0.125	94	62
0.075	77	48
0.06	68	43
0.0425	57	37
0.03	47	32
0.025	39	28
0.02	33	25
0.015	24	19
0.0125	17	14

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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	S3	1.0	117.8
⊠	S4	0.3	117.5

Prep'dDB.....

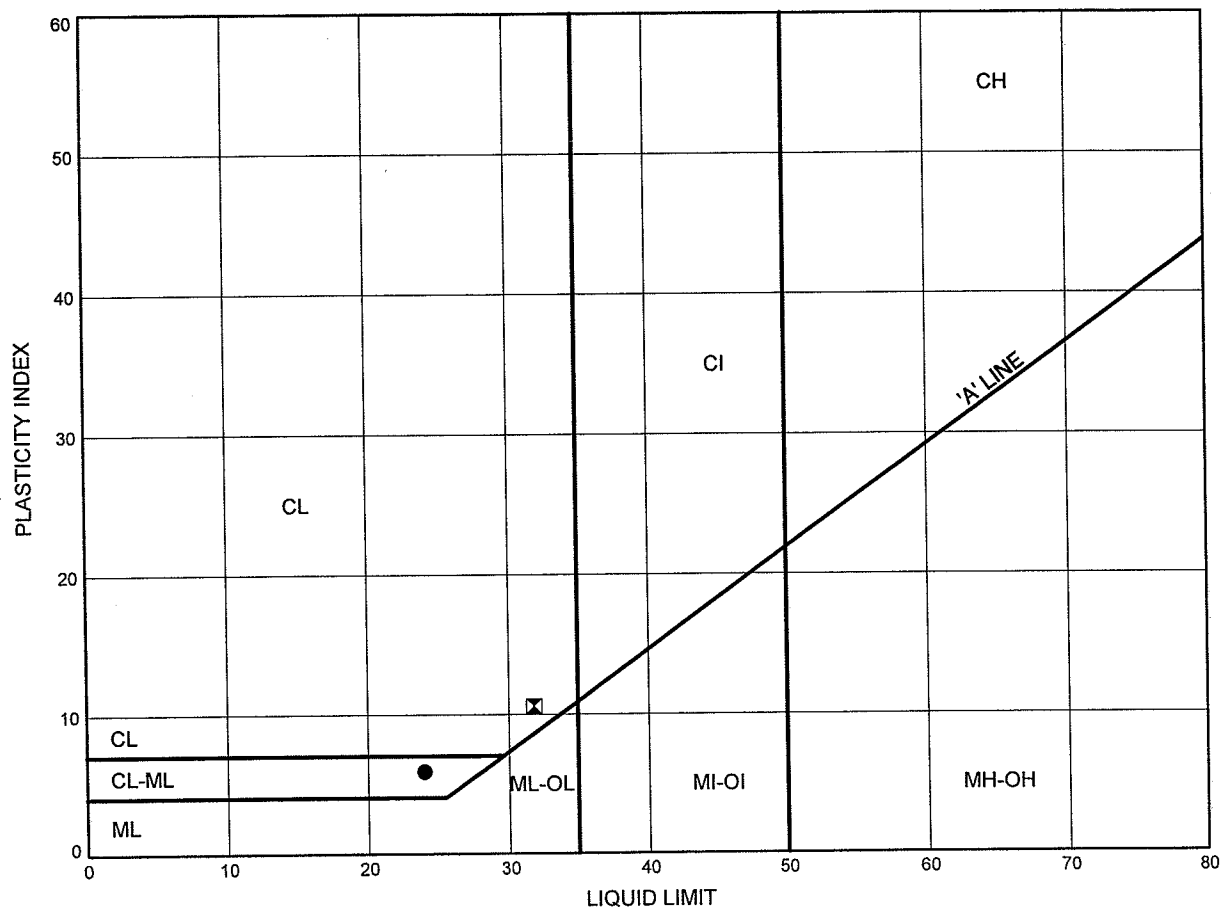
Chkd. RA



ATTERBERG LIMITS TEST RESULTS

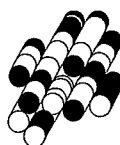
FIGURE B8

FILL - Silty Clay



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	S3	1.0	117.8
⊠	S4	0.3	117.5

Date February 2009
Project 2831-02-01

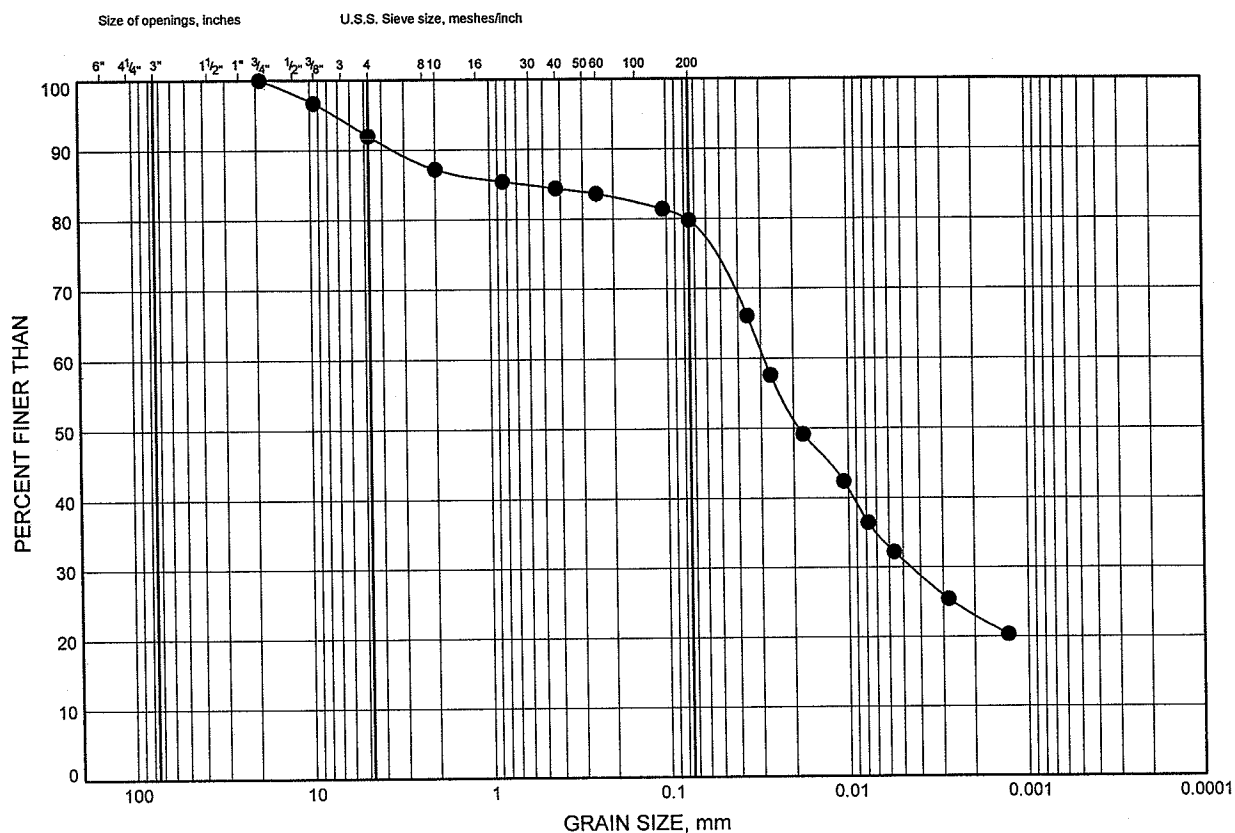


Prep'd DB
Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B9

SILTY CLAY TILL



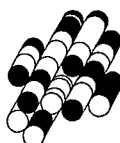
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

● S4 1.0 116.8

Date February 2009

Project 2831-02-01



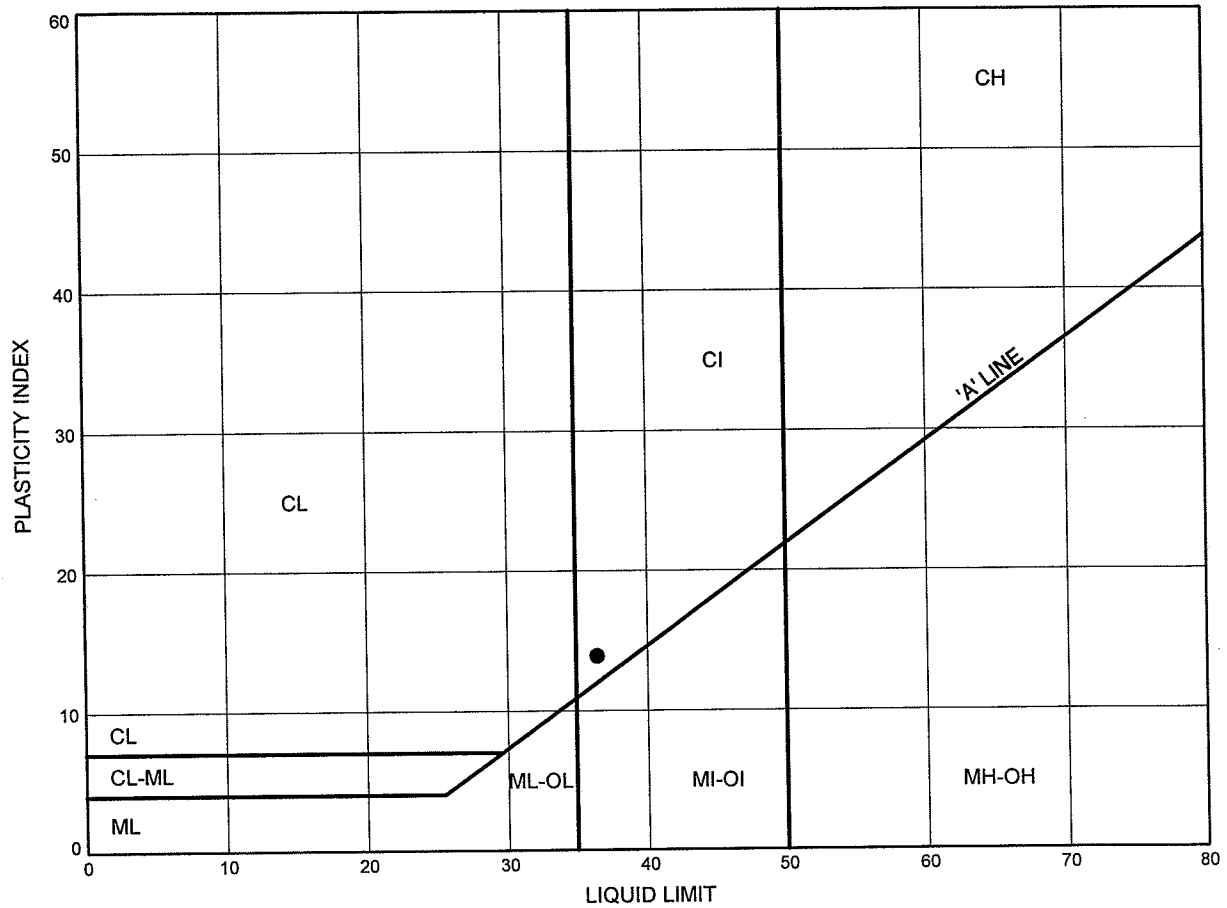
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

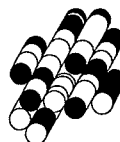
FIGURE B10

SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	S4	1.0	116.8

Date February 2009
Project 2831-02-01...

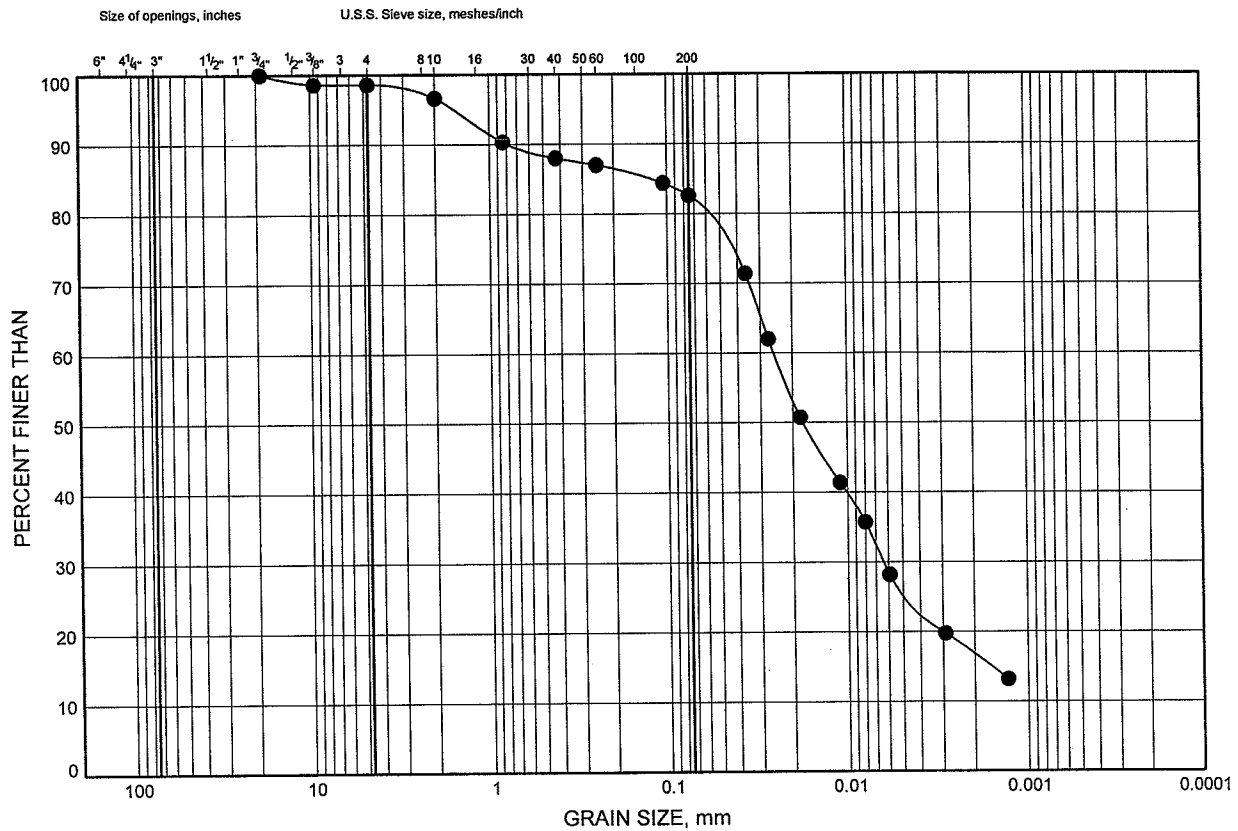


Prep'd DB
Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B11

TILL SHALE COMPLEX

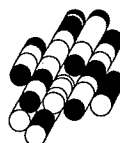


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	S3	3.2	115.6

Date February 2009

Project 2831-02-01



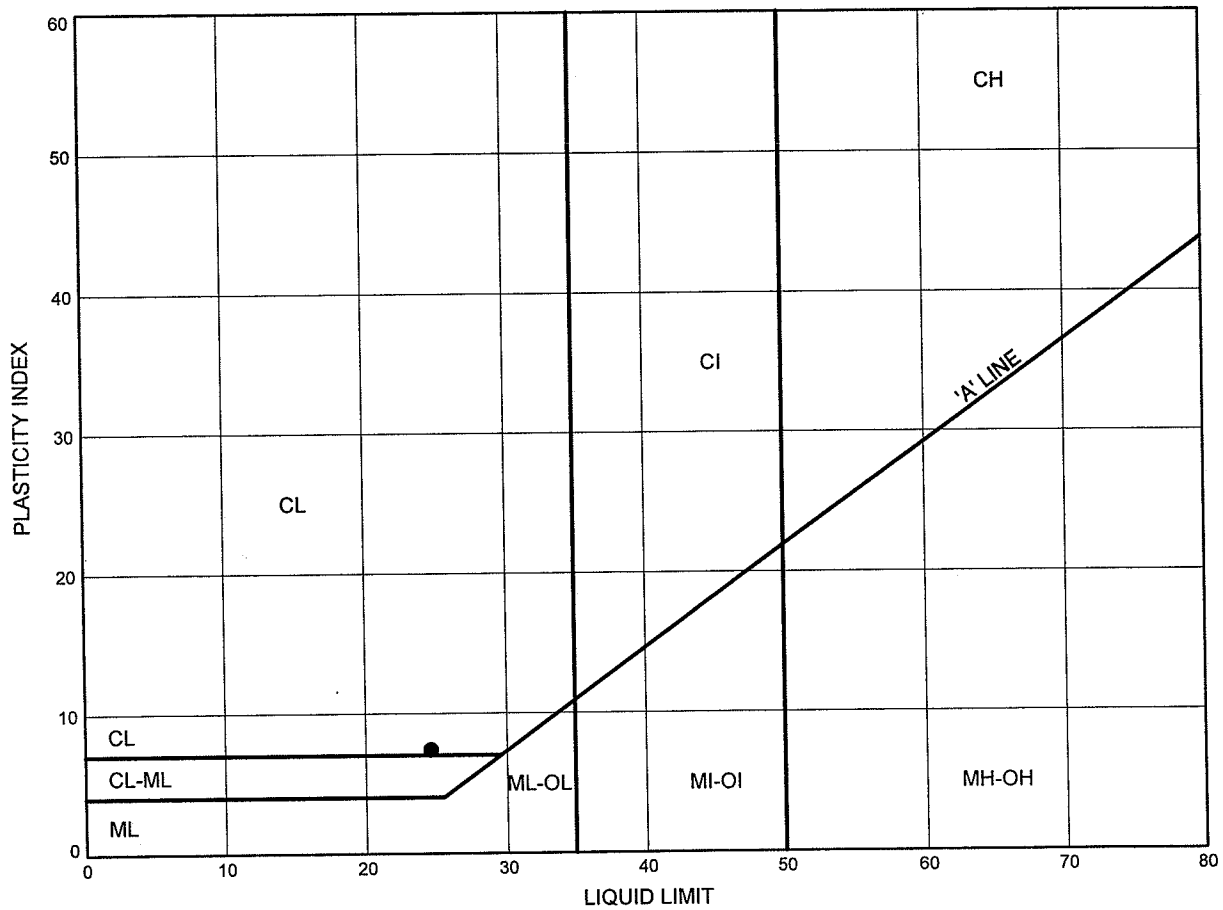
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

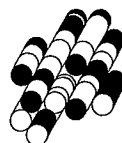
FIGURE B12

TILL SHALE COMPLEX



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	S3	3.2	115.6

Date February 2009
Project 2831-02-01

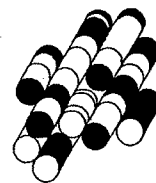


Prep'd DB
Chkd. RA

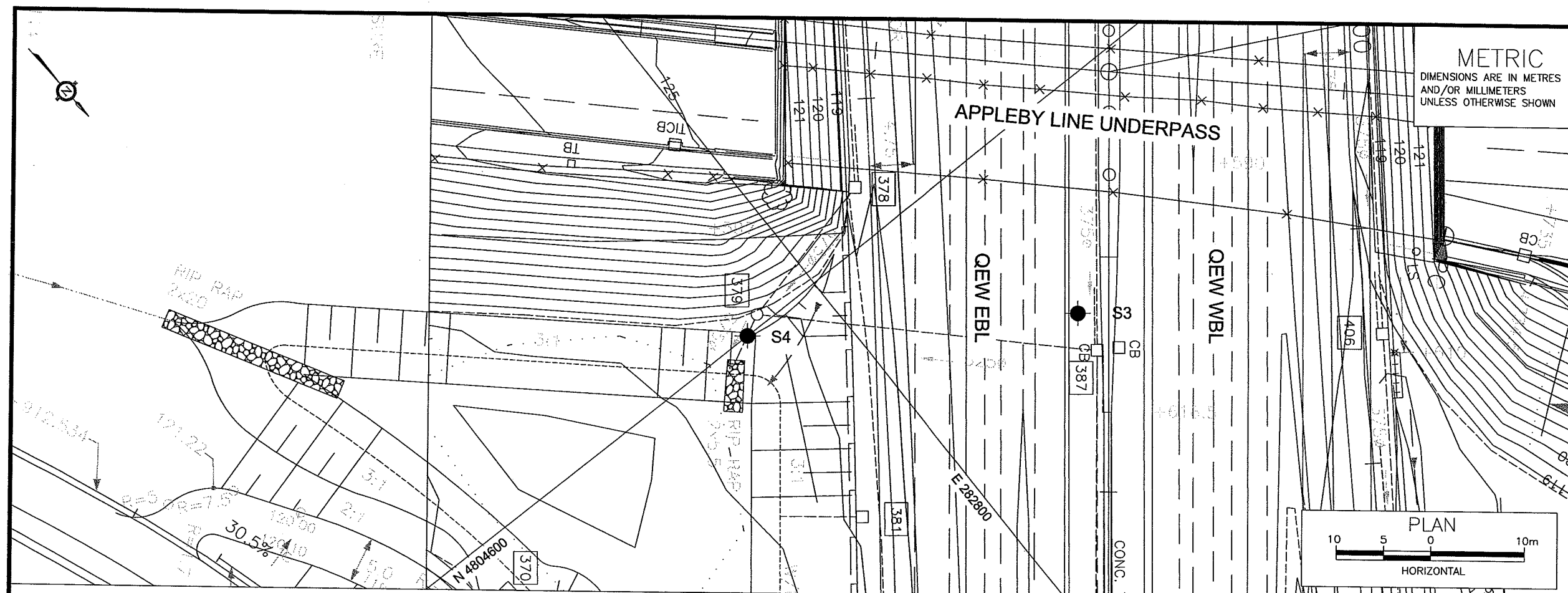
APPENDIX C

**Drawing titled “Borehole
Locations and Soil Strata”**

Terraprobe Limited



C:\Documents and Settings\Admin\My Documents\1 AUTOCAD 2008 FILES\1 Projects 2009 Jun\1 05-4027 QEW\Drawings\1 05-4027 Working.dwg, User B



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

CONT No 2008-XXXX
WP No 2831-02-01

QUEEN ELIZABETH HIGHWAY
BRANT STREET TO BURLOAK DRIVE
APPLEBY LINE STORM SEWER

SHEET

Giffels Associates Limited
Consulting Engineers and Architects
An IBI Group Company

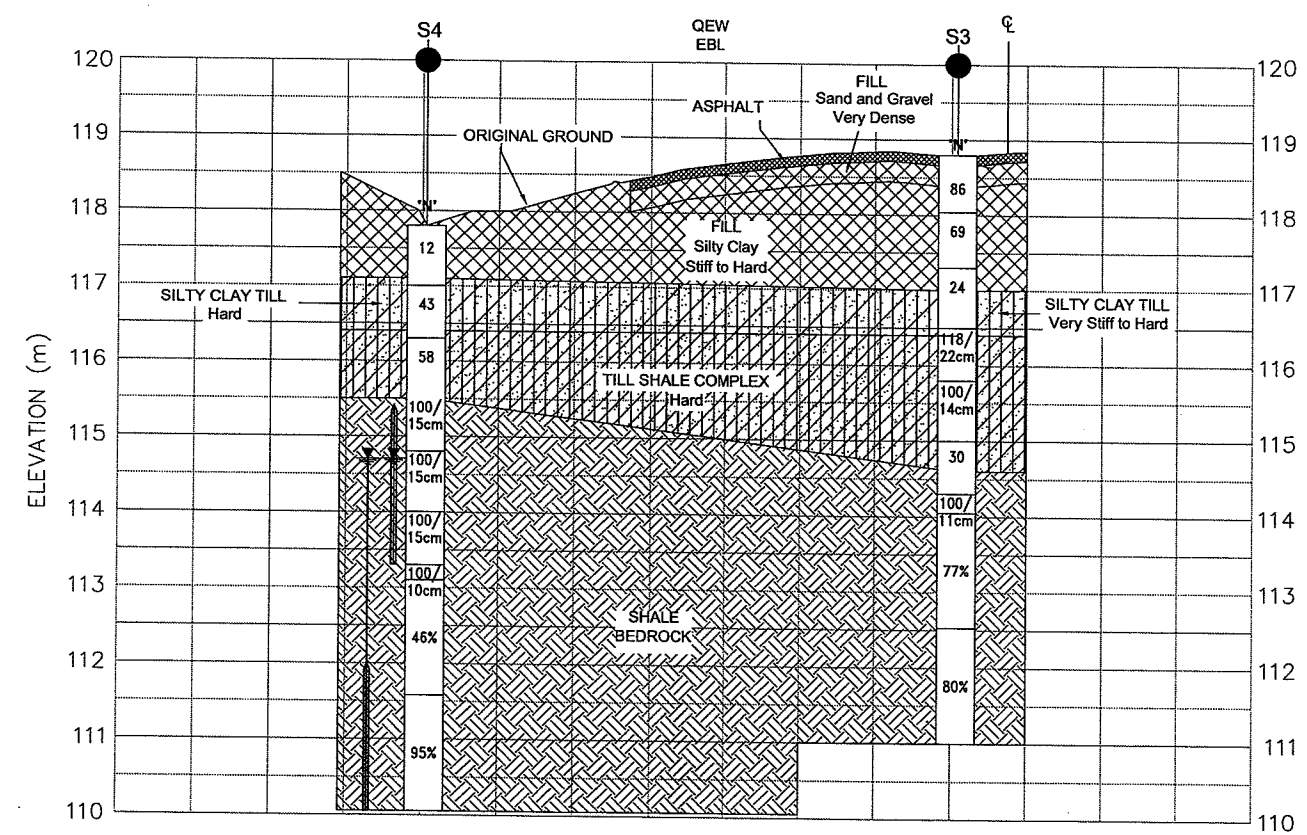
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing

WALKERS LINE

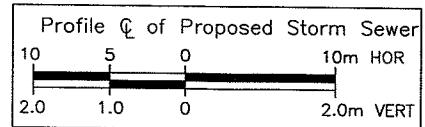
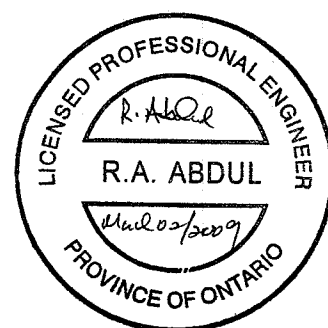
APPLEBY LINE

0 500 1000m

KEY PLAN



PROFILE FOR SEWER
CROSSING QEW AT STA 17+609



LEGEND			
	Bore Hole		
	Dynamic Cone Penetration Test (Cone)		
	Bore Hole & Cone		
	Blows/0.3m (Std Pen Test, 475 J/blow)		
	Blows/0.3m (60° Cone, 475 J/blow)		
	WL at Time of Investigation		
	WL in Piezometer (FEB 2009)		
	Piezometer		
	90% Rock Quality Designation		
	Auger Refusal		
No	ELEVATION	COORDINATES	
		NORTHING	EASTING
S3	118.8m	4 804 619.9	282 779.9
S4	117.8m	4 804 600.0	282 809.0

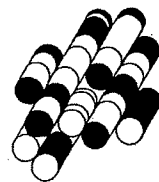
NOTE			
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.			
REVISIONS			
DATE	BY	DESCRIPTION	
DESIGN R.A.CODE	LOAD	DATE	FEB 2009
DRAWN L.B.CHK R.A.	STRUCT		

DRAWING NOT TO BE SCALED

APPENDIX D

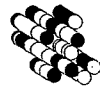
Comparison of Installation Methods

Terraprobe Limited



COMPARISON OF TRENCHLESS INSTALLATION METHODS

Microtunnelling	Horizontal Auger Boring
<p>Advantages:</p> <ul style="list-style-type: none"> i. Avoids open cut excavation, highway closure and traffic diversion. ii. Well tested technology. iii. Smaller jacking and receiving pits compared to Pipe Jacking and Horizontal Auger Boring. iv. Accuracy/Tolerance ± 25 mm. v. Relatively good control of potential settlement. vi. Can be used for tunnelling in both rock and soil and would therefore be suitable for both alignments. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Equipment may not be readily available. ii. More expensive than Horizontal Auger Boring. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Avoids open cut excavation, highway closure and traffic diversion. ii. Readily available equipment/technology. iii. More economical than microtunnelling. iv. Relatively good control of potential settlement. v. Accuracy/Tolerance ± 25 mm. vi. Can be used for tunnelling in both rock and soil and would therefore be suitable for both alignments. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. More expensive than open cut excavation. ii. Requires good care and workmanship by experienced tunnellers in order to reduce ground settlement above the existing freeway. iii. Potential problems such as boulders and water bearing sand lenses as well as rock loss due to vertical and subvertical joints can cause ground loss during tunnelling which can result in excessive ground settlement. iv. Accuracy/Tolerance ± 25 mm v. Relatively large area required to accommodate bore pit and to lay out pipe. vi. May require a small boring unit for rock excavation depending on rock strength.

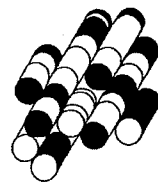


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APPENDIX E

Suggested NSSP Wording

Terraprobe Limited



In this report reference is made to the following Provincial Standards

- OPSS 514, November 2005.
- OPSS 515, November 2005

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 may impede the progress of trenching and trenchless installation. The soil conditions are described in the Foundation Investigation Report prepared for this site. Reference should be made to this report for a description of the soil conditions.”

Shale Bedrock

“The Contractor is informed that shale bedrock of the Queenston Formation will be encountered at this site. The rock conditions are described in the Foundation Investigation Report prepared for this site. Reference should be made to this report for a description of the rock conditions. Appropriate equipment should be selected to deal with the shale excavation as well as the harder limestone layers that are likely to be encountered when excavating the rock.”

Mixed Face Conditions

“The Contractor is informed that mixed face conditions will be encountered when tunnelling the Walkers Line Crossing (QEW Station 15+425). The contractor must ensure that the selected equipment can deal with these varying conditions as well as maintain proper alignment during the tunnelling operation.”

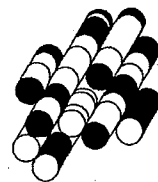
“The Contractor is required to submit a detailed work plan. The work plan must provide details on the proposed construction sequence and methodology and must also address how the Contractor intends to deal with any ground loss that may occur.”



APPENDIX F

Settlement Monitoring Guideline

Terraprobe Limited



SETTLEMENT MONITORING GUIDELINE

Instruments

Two types of settlement monitoring points are required:

- Surface points are placed within the asphalt portion of the highway
- In-ground points, approximately 2 m deep, are proposed next to the inner and outer shoulders of the QEW EBL and WBL. The in-ground points are important for detecting settlements before they are transferred to the surface.

Instrumentation Arrays

In-Ground Monitoring Points

The lateral extent of the monitoring array shall cover a distance on both sides of the tunnel alignment as defined by a 45 degree line extending from one radius of the centerline at the invert level to the ground surface.

As a minimum, five (5) instrument arrays shall be utilized, three at the Walkers Line Crossing and two at the Appleby Line Crossing. At the Walkers Line Crossing an array is to be installed perpendicular to the proposed alignment next to the east bound and west bound shoulders and near the median centre line of the QEW. At the Appleby Line Crossing an array is to be installed perpendicular to the proposed alignment next to the inner and outer shoulders of the QEW EBL. At each location the array of in-ground monitoring points should consist of a minimum of five in-ground monitors located as follows:

Walkers Line Crossing: one point directly over the centerline of the tunnel, and one point each at approximately 1.5 m and 5.5 m on either side of the tunnel.

Appleby Line Crossing: one point directly over the centerline of the tunnel, and one point each at approximately 2 m and 5.5 m on either side of the tunnel.

Surface Monitoring Points

Surface monitoring points will be installed on the pavements.

At the Walkers Line Crossing surface monitoring points will be located on each traveled lane as well as the paved inner and outer shoulder of the QEW EBL and WBL. At the Appleby Line Crossing surface monitoring points will be located on each traveled lane as well as the paved inner and outer shoulder of the QEW EBL. The surface monitoring points will be identified using paint marks on the pavement.

The final instrumentation plan should be finalized when the Contractor's proposed construction method is available.

Condition Survey



A condition survey of the pavement will be carried out prior to commencement of construction and documented for the purpose of requiring restoration, if necessary. The condition survey will be carried out using the surface monitoring points installed on each travelled lane. This surface survey will be completed when the in-ground monitors and settlement points are installed and again when the tunnel has been completed. Interim surveys will be required should movement be detected in the in-ground monitoring points.

Reading Frequency

In-ground and surface monitoring points shall be read and the data recorded continually by the Contractor during the construction period. Readings shall continue to be made after construction to a time at which all parties agree that there is no further movement.

It is recommended that at least three (3) sets of readings be taken during each shift, provided that movements are within anticipated limits. Otherwise, the frequencies should increase according to a pre-planned interval.

Monitoring of movements is required during work stoppages, such as during a non-operation period (off-shifts) or weekends. At least three (3) sets of readings should be taken daily.

Data Collection and Data Transfer

A procedure is required to be established in consultation with MTO so that the monitoring data and the interpreted data will reach all parties as soon as necessary. The responsible prime Consultant and the Contractor should interpret monitoring data as needed for the purpose of on-going construction. The Geotechnical Engineer should be contacted for technical support to the prime Consultant in the interpretation of ground movements and review of the Contractor's response when Review and Alert Levels are reached.

Criteria for Assessment

The suggested acceptable surface settlement (or heave) is 12 mm, or at criteria specified by MTO. The baseline reading, alert level and review level should be established with input from MTO.

Baseline Reading – A baseline reading of the instrumentation shall be taken prior to commencement of the work. All parties should recognize and accept the baseline level in writing.

Review Level – A maximum value of 6 mm relative to the baseline readings is suggested for this project. If this level is reached, the method, rate or sequence of construction, or ground stabilization measures should be reviewed or modified to mitigate further ground displacements.

Alert Level – A maximum value of 10 mm relative to the baseline readings is suggested for this project. If this level is reached, the Contractor shall cease construction operations and execute pre-planned measures to secure the site, to mitigate further movements and to assure safety of the public and maintain uninterrupted traffic flow.



Review of Contractor's Proposed Method

The Contractor's proposed method of construction should be reviewed by MTO, the Proponent's prime consultant and Geotechnical Engineer. The proposed method should include a description of the potential loss of ground, calculation of the maximum settlement in relation to the Contractor's procedure and equipment, alternative/remedial measures if the review level of measurement is reached; and contingency/remedial measures if the alert level of measurement is reached.

Contractor's Responsibility For Restoration and Warranty Provision

Notwithstanding the monitoring program to assess the adequacy of the tunnelling construction method to control potential ground movements and groundwater, the Contractor is responsible for reinstatement (such as surface paving) should movements or other surface distresses occur. The Contract is also required to provide a reasonable warranty period for the works acceptable to MTO.

Construction Monitoring

The Proponent shall retain a qualified Geotechnical Consultant to supervise the installation of surface and subsurface settlement points on site and to provide direction, technical input and field inspection on this project.



Foundation Investigation Report
Storm Sewer Installations
Queen Elizabeth Way from Brant Street to Burloak Drive
Agreement N 2006-E-0026, W.P. 2831-02-01



Bedrock Core Sample
Borehole S1
Runs 1, 2 & 3
Depth 4.4m - 7.8m



Foundation Investigation Report
Storm Sewer Installations
Queen Elizabeth Way from Brant Street to Burloak Drive
Agreement N 2006-E-0026, W.P. 2831-02-01



Bedrock Core Sample
Borehole S2
Runs 1, 2, 3 & 4
Depth 2.9m - 7.8m



Foundation Investigation Report
Storm Sewer Installations
Queen Elizabeth Way from Brant Street to Burloak Drive
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Bedrock Core Sample
Borehole S3
Runs 1 & 2
Depth 4.8m - 7.8m



Foundation Investigation Report
Storm Sewer Installations
Queen Elizabeth Way from Brant Street to Burloak Drive
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Bedrock Core Sample
Borehole S4
Runs 1 & 2
Depth 4.7m - 7.8m



Foundation Investigation Report
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Bedrock Core Sample
Borehole HML 3A
Runs 1 & 2
Depth 5.6m - 8.9m

