

**FOUNDATION INVESTIGATION AND DESIGN REPORT
SHERWOODTOWNE BOULEVARD GRADE SEPARATION STRUCTURE
MISSISSAUGA BUS RAPID TRANSIT (BRT) PROJECT
MISSISSAUGA, ONTARIO**

Geocres Number: 30M12-287

Report to

McCormick Rankin Corporation

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a grade separation structure to carry the proposed Bus Rapid Transit way (BRT) under the existing Sherwoodtowne Boulevard in Mississauga, Ontario. The proposed structure will be located on the south side of Highway 403 adjacent to the east side of Hurontario Street.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections and a written description of the subsurface conditions. A model of the subsurface conditions was developed to describe the geotechnical conditions influencing design and construction of the foundations for the structure.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation under their Sub-consultant Agreement for Project Number 7493.

2 PROJECT AND SITE DESCRIPTION

The BRT project involves a fully grade-separated, two-lane bus-only roadway located in the City of Mississauga, extending from the City Centre Station (Highway 403 at Hurontario Street) to the Renforth Drive Station (Renforth Drive at Eglinton Avenue). The total length is approximately 9.5 km.

The segment of the BRT at this site will include a grade separation structure to carry the proposed BRT under Sherwoodtowne Boulevard.

The site is located adjacent to the east side of Hurontario Street, approximately 250 m south of Highway 403 EBL and 120 m north of Rathburn Road in Mississauga, Ontario.

Lands adjacent to the site have been developed for residential and commercial uses.

Photographs of the site included in Appendix E, show the general nature of the site.

The site is situated within the South Slope physiographic region. The geology generally comprises a till plain consisting of clayey silt to silty clay (Halton Till) overlying bedrock at relatively shallow depth. The bedrock consists of grey shale, siltstone and limestone of the Georgian Bay Formation.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation was carried out on March 13 and 14, 2009. The field program consisted of drilling and sampling four boreholes (numbered 09-005 to 09-008) for the proposed grade separation structure.

All the boreholes were terminated in shale bedrock. Boreholes 09-006 and 09-007 were terminated at 6.1 m depth (Elevations 154.6 to 155.0). Boreholes 09-005 and 09-008 were advanced into the shale bedrock by coring to depths of 14.6 m and 13.6 m (Elevations 146.3 and 147.4), respectively, with 8.5 m and 9.5 m of rock core recovered.

The approximate borehole locations are shown on the Borehole Locations and Soil Strata Drawing in Appendix F. The coordinates and elevations of the boreholes are given on these drawings and on the individual Record of Borehole Sheets in Appendix A.

Prior to commencement of drilling, utility clearances were obtained for all borehole locations.

Hollow stem augers were used to advance the boreholes in the overburden and into the shale. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). NQ rock coring equipment was used to recover core samples of the underlying bedrock in selected boreholes.

A member of Thurber's engineering staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes, visually examined the recovered samples, and transported them to Thurber's laboratory for further examination and testing.

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers consisting of 19 mm PVC pipes with screens were installed in two boreholes to permit monitoring of groundwater levels. Details of the piezometer installations and other borehole completion details are as shown in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole	Piezometer Tip Depth/ Elevation (m)	Completion Details
East Abutment	09-005	None installed	Bentonite to surface.
	09-006	None installed	Bentonite to 3.0 m, auger cuttings from 3.0 m to 0.1 m, then asphalt to surface.
West Abutment	09-007	6.1/155.0	Piezometer with 1.5 m slotted screen installed with sand filter to 3.7 m, bentonite from 3.7 m to 2.1 m, auger cuttings from 2.1 m to 0.1 m, then asphalt to ground surface.
	09-008	13.0/148.0	Piezometer with 1.5 m slotted screen installed with sand filter to 9.9 m, bentonite from 9.9 m to ground surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and rock samples to geological logging. At least 25% of the recovered samples of soil were also subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg Limits testing where appropriate. Moisture content determinations were carried out on all soil samples. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B.

Core samples of the shale bedrock were carefully protected to prevent drying during transport to the laboratory. Point load tests and unconfined compression tests (UCS) were carried out on selected samples of intact shale, siltstone and limestone interbeds upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. The results of the point load and unconfined compression tests are shown on the Record of Borehole sheets, and the UCS results are also included in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

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

In general terms, the soil stratigraphy encountered at this site comprises surficial topsoil or pavement structure overlying sand and gravel fill, sand fill and silty clay fill, underlain by native silty clay till at

the north end. Shale bedrock was contacted below the fill and till deposit. More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

Pavement structure consisting of approximately 75 mm of asphalt overlying granular (sand and gravel) road base was encountered surficially in Borehole 09-006 drilled on Sherwoodtowne Boulevard. In Borehole 09-007 drilled on the boulevard, a 50 mm thick paving stone was encountered over granular base. The thickness of granular fill measured in the boreholes ranged from 625 mm to 650 mm and the underside lay at elevations 160.0 and 160.5. The moisture content of the granular material was 3 to 4%.

5.2 Topsoil

A 50-mm thick layer of topsoil was identified surficially in Borehole 09-005.

The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.3 Fill

Fill consisting of intermixed layers of various soils was encountered in all boreholes:

- Sand and gravel fill containing some clay and some silt was contacted below the topsoil in Borehole 09-005 and surficially in Borehole 09-008. The thickness of the sand and gravel fill was 650 mm and 700 mm.
- Gravelly sand fill was contacted at 1.2 m depth (Elevation 159.9) in Borehole 09-007. The thickness of the sand fill was 1.6 m.
- Silty clay fill containing some sand, trace gravel, occasional organics, occasional asphalt and limestone fragments and occasional cobbles was contacted below the granular road base in Boreholes 09-006 and 09-007 and below the sand and gravel fill in Boreholes 09-005 and 09-008. The thickness of the silty clay fill ranged from 0.5 m to 3.4 m.
- A second layer of clay fill was encountered below the sand fill in Borehole 09-007. This fill layer was 1.7 m thick.

The depths to the base of the fill ranged from 3.0 m to 4.5 m (Elevations 156.6 to 158.0).

Based on SPT 'N' values typically ranging from 6 to 25 blows for 0.3 m of penetration, the silty clay fill is generally described as firm to very stiff in consistency. SPT 'N' values of 42 blows per 0.3 m of penetration to 80 blows per 0.2 m (hard) recovered in the fill may reflect the presence of rock fragments or other obstructions. SPT 'N' values of 24 blows per 0.3 m of penetration and 75 blows per 0.225 m were measured within the sand fill, indicating a compact to very dense relative density.

The natural moisture contents of the fill samples recovered ranged from 4% to 21%.

Grain size distribution curves for two samples of sand/gravel fill and two samples of silty clay fill are presented on the Record of Borehole sheets and on Figures B1 and B2 of Appendix B. Atterberg Limit test results for the clay fill are presented on Figure B4 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	Silty Clay Fill	Sand/Gravel Fill
Gravel %	3 to 4	25 to 52
Sand %	19 to 26	31 to 52
Silt %	44 to 48	6 to 17
Clay %	27 to 27	

Liquid Limit	39 to 44
Plastic Limit	20 to 22

The above results show that the silty clay fill is typically of medium plasticity with a group symbol of CI.

5.4 Silty Clay Till

Native brown silty clay till containing some sand, trace gravel and occasional limestone and shale fragments was contacted below the fill in Boreholes 09-006 and 09-008. The thickness of the silty clay till was 0.8 m and 0.4 m.

The depth to the base of the silty clay till was 3.8 m and 3.4 m (Elevation 156.9 and 157.6) in Boreholes 09-006 and 09-008, respectively.

Based on SPT 'N' values of 25 and 35 blows for 0.3 m of penetration, the silty clay till is described as very stiff to hard in consistency.

The natural moisture contents of the samples recovered from the silty clay till layer ranged from 8% to 10%.

Grain size distribution curves for two samples of silty clay till are presented on the Record of Borehole sheets and on Figure B3 of Appendix B. Atterberg Limit test results are presented on Figure B5 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	4 to 5
Sand	14 to 16
Silt	54 to 61
Clay	21 to 25

Liquid Limit	41
Plastic Limit	22

The above results show that the silty clay till is typically of medium plasticity with a group symbol of CI.

Glacial tills inherently contain cobbles and boulders and the lower part of the till may contain pieces and slabs of bedrock which may account for some high blow counts.

5.5 Bedrock

The soils described above were underlain by shale bedrock. The shale encountered in the boreholes is described as thinly bedded and contains numerous hard interbedded siltstone and limestone layers. The shale bedrock is highly to moderately weathered within the upper 1.5 m to 2.5 m below which it becomes less weathered and stronger with depth. SPT 'N' values obtained in the upper part of the shale bedrock were 50 to 70 blows per 0.025 m to 0.175 m of penetration. Moisture contents ranged from 4% to 12%. The depths and elevations of the top of weathered bedrock are shown in Table 5.1.

Table 5.1 – Depths and Elevations of Top of Weathered Bedrock

Foundation Unit	Borehole	Depth to Weathered Bedrock (m)	Top of Weathered Bedrock Elevation (m)
East Abutment	09-005	4.1*	156.7
	09-006	3.8	156.9
West Abutment	09-007	4.5	156.6
	09-008	3.4*	157.6

* Proved by coring below augered depth.

Bedrock cores were collected using NQ sized coring equipment. Total core recovery (TCR) in the bedrock ranged from 90% to 100% in all core runs.

RQD values recorded in the core runs typically ranged from 83% to 100%, indicating a good to excellent rock quality. RQD values of 0% were recorded in Borehole 09-005 Run 1 and Borehole 09-008 Runs 1 to 3, indicating a very poor rock quality in the upper part of the shale. An RQD value of 50% was measured in Borehole 09-008 Run 4, indicating a fair to poor rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to greater than 25. Broken/rubble zones and clay seams were observed within the cores at several depths.

The unconfined compressive strength of the rock assessed from Point Load testing and Unconfined Compression tests conducted on recovered rock cores are presented on the Record of Borehole sheets in Appendix A. The UCS test reports are included in Appendix B. The compressive strength results ranged from 3 to 31 MPa (very weak to medium strong) for the shale and 26 to 189 MPa (medium strong to very strong) for the limestone interbeds.

It must be noted that point load tests were possible only on less weathered shale or higher strength limestone interbed samples as the more typical weathered shale cores tended to be not suitable for point load testing.

The shale bedrock typically contains layers of siltstone and limestone that can be significantly harder than the shale itself. The distribution, thickness and strength of these layers vary from location to location, and these layers typically exhibit less pronounced weathering than the shale. The records of boreholes indicate that within the depths investigated, these hard interbeds range up to 160 mm in thickness. Sampling and interpretation from small diameter boreholes may underestimate the frequency, thickness and strength of the strong layers and therefore geological expertise and past experience must be applied in any decision making process regarding the bedrock.

5.6 Water Levels

Water levels were observed in the boreholes during and upon completion of drilling. Standpipe piezometers were installed in two boreholes to monitor water levels after completion of drilling. The water levels measured in the piezometers are summarized in Table 5.2.

Table 5.2 – Measured Groundwater Levels

Foundation Unit	Borehole	Date (2009)	Water Level (m)		Comment
			Depth (m)	Elevation (m)	
West Abutment	09-007	April 14	5.9	155.2	In piezometer
		May 5	5.5	155.6	
		May 21	5.7	155.4	
	09-008	April 14	9.3	151.7	In piezometer
		May 5	6.6	154.4	
		May 21	8.5	152.5	

The most recent groundwater levels measured in the piezometers range from elevations 152.5 to 155.4.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall. Further, perched water may be encountered at higher levels in pockets or zones of more permeable sands and silts within the heterogeneous tills, or within the fill.

6 MISCELLANEOUS

Borehole locations and ground surface elevations were supplied to Thurber by McCormick Rankin Corporation.

The drilling and sampling equipment was supplied and operated by DBW Drilling of Ajax, Ontario and Eastern Ontario Diamond Drilling Ltd. of Hawkesbury, Ontario. The field work was supervised on a full time basis by Mr. Luke Gilarski and Ms. Eckie Siu of Thurber Engineering Ltd. under the direction of Mr. Murray R. Anderson, P.Eng and Mr. Mark Farrant, P. Eng.

Laboratory testing was carried out at Thurber's Laboratory in Oakville, Ontario.

Overall supervision of the field program was conducted by Mr. Murray R. Anderson, P.Eng. and Mr. M. Farrant, P. Eng. Interpretation of the data and preparation of the report were carried out by Mr. Murray R. Anderson, P.Eng and Ms. R. Palomeque Reyna, P.Eng.

Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects, reviewed the report.

THURBER ENGINEERING LTD.

Rocio Palomeque Reyna, P.Eng.
Geotechnical Engineer



Murray R. Anderson, P.Eng.
Senior Foundations Engineer



P.K. Chatterji, P.Eng.
Review Principal



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PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

7 INTRODUCTION

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

It is understood that the proposed structure is to carry the proposed BRT under the existing Sherwoodtowne Boulevard in Mississauga, Ontario.

Based on the preliminary General Arrangement (GA) drawing provided by McCormick Rankin Corporation, a single-span, concrete structure is planned. The span will be approximately 14.8 m (parallel to existing Sherwoodtowne Boulevard alignment) and the width will be approximately 16.6 m.

The existing ground surface at Sherwoodtowne Boulevard is near elevation 161.0. The proposed BRT alignment will be in a cut with the base near elevation 154.7. A cut of approximately 6.3 m depth will therefore be required to pass the proposed BRT under Sherwoodtowne Boulevard.

Retaining walls will extend parallel to the proposed BRT route adjacent to each corner of the structure. The current design calls for the wall at the southeast corner of the structure to be a contiguous caisson wall and the remaining walls to be RSS walls.

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 investigation.

8 FOUNDATION DESIGN

Consideration was given to various alternate foundations systems, taking account of the site stratigraphy and the structure General Arrangement. In general terms, the stratigraphy encountered at the site consists of topsoil or asphalt overlying sand and gravel fill, compact to very dense sand fill and firm to hard silty clay fill, underlain by native stiff to hard silty clay till at the north end. Weathered shale bedrock was contacted below the fill and silty clay till deposit at depths varying from 3.4 m to 4.5 m (Elevations 156.6 to 157.6). Piezometers installed in Boreholes 09-007 and 09-008 at the west abutment revealed that the groundwater level is anticipated to be near 5.5 m to 8.5 m depth (Elevations 155.6 to 152.5) although perched water may be encountered at higher levels within the fill.

Initial consideration was given to the following foundation types:

- Spread footings on native silty clay till
- Spread footings on shale bedrock
- Spread footings on engineered fill
- Augered Caissons (drilled shafts)
- Driven steel H-piles

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

From a geotechnical perspective and based on the subsurface conditions, spread footings founded on shale bedrock is considered the most cost effective foundation option for supporting the structure at this site. The base of the proposed busway cut lies within the shale bedrock and therefore a high bearing resistance is available for footings founded on bedrock.

Use of deep foundations such as piles and caissons is not considered cost effective at this site due to the presence of shallow bedrock. If pile foundations are to be used, the piles would need to be extended well below the bedrock surface by pre-drilling, rock coring or rock excavation to achieve adequate pile embedment into the shale. Further, a high bearing resistance is available for design of spread footings founded on the shale bedrock at the level of the cut base. Consequently, deep foundations are not recommended and detailed design recommendations for deep foundations were not developed.

8.1 Spread Footings on Native Silty Clay Till

The native clay till deposit was not encountered at the south end of the site. The native silty clay till was relatively thin at the north end, and has a variable consistency. Further, the till unit is located above the base of the busway cut. Therefore, the native clay till is not a suitable stratum for support of the structure.

8.2 Spread Footings on Shale Bedrock

The proposed busway grade at the structure location will be near elevation 154.7 m.

Spread footings bearing on undisturbed weathered shale bedrock at or below elevation 154.7 m (the base of the cut) may be designed for the following geotechnical resistance:

- Factored geotechnical resistance of 1,000 kPa at Ultimate Limit States (ULS)

The SLS condition will not govern design for footings founded on shale bedrock.

This resistance value is for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The lateral resistance of the footings founded on shale may be computed using an unfactored friction factor of 0.5. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

The bearing surface should be prepared by removing all loose/disturbed material and shattered rock. The exposed shale surface should be protected from deterioration by placing a minimum 100 mm thick working mat of concrete of the same class as the footing within 4 hours of completing excavation. Areas requiring subexcavation beneath the underside of footing should be backfilled with the same class of concrete as used in the footing.

8.3 Spread Footings on Engineered Fill

As the base of the subway cut extends into shale bedrock, placement of engineered fill to support footprints is not considered to be a suitable option at this site.

8.4 Frost Protection

The design depth of frost penetration at this site through overburden soils is 1.2 m.

Although the shale is geologically defined as bedrock, it is susceptible to frost action. Therefore, all footings must be provided with a minimum of 1.2 m of earth cover as frost protection. It is possible to reduce the thickness of earth cover by the substitution of synthetic insulation and typically 25 mm of Styrofoam is equivalent to 600 mm of earth cover. Synthetic insulation must be covered to provide protection where it is used.

8.5 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, spread footings founded on bedrock is considered the most cost effective foundation option for supporting the grade separation structure at this site.

9 PERMANENT CUT

Permanent earth and rock cuts are required to construct the grade separation structure at this site. The earth cut will be formed through about 3.4 m to 4.5 m of existing fill, native silty clay till and then in shale bedrock. At the maximum depth, permanent cut slopes through the cohesive and cohesionless fills and native soils are expected to be stable at inclinations not steeper than 2H : 1V.

The shale will generally be stable with near-vertical faces. However, the shale is prone to weathering and on the longer term steep slopes are expected to slough and ravel. Taking account of the properties of the shale, it is recommended that permanent slopes in shale also be formed no steeper 2H:1V. It is anticipated that the base of the cut will consist of shale bedrock and will be stable.

Based on water levels measured in the piezometers, the ground water level is expected to be near or just above the base of the cut. In general the bedrock at the cut base is considered to be of low permeability and consequently seepage from the sides and base of the cut is expected to be of low volume. Longitudinal subdrains along the sides of the BRT outletting in the storm drainage system should be provided for permanent drainage of the cut. The need for lateral drains connected to the longitudinal subdrains in the structure area should be assessed as part of the pavement design requirements.

Drainage ditches and storm sewers can be used to control surface runoff and precipitation.

In view of the depth and extent of the cuts required for this project, it is recommended that a Permit to Take Water (PTTW) be obtained from the Ministry of Environment prior to commencement of construction.

Vegetative cover should be established on all exposed earth and rock slopes to protect against surficial erosion. Reference may be made to special provision SP572S01 for more detailed requirements, where applicable.

10 TEMPORARY EXCAVATION

Earth excavations required at this site will penetrate through a variety of overburden soils including sand and gravel fill, sand fill, silty clay fill and native silty clay till. The soils, especially the tills, may contain cobbles, boulders and slabs of rock. Rock excavation is also required.

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the soils within the likely depth of excavation at this site may be classed as Type 3 soils and temporary cut slopes should be inclined not steeper than 1H:1V. Near vertical cut slopes should be stable over a short term in shale.

Flatter temporary slopes may be required at locations where the soils are less competent than what is assumed during design or where water seepage affects surficial stability.

The selection of the method of excavation is the responsibility of the contractor and the contractor is solely responsible for assessing the type, size and power rating of the required equipment. However,

from the point of view of assessing constructability, the following points should be taken into consideration:

- It is anticipated that a hydraulic excavator will be suitable for the overburden soils. Arduous excavation may be experienced in the hard till materials.
- The silty clay till grades into weathered bedrock and there is often not a distinct boundary between the two. Accordingly, excavation of the lower portions of the clay till may be similar to the upper, more weathered layers of the bedrock.
- Provision must be made for the handling of pavement materials, potential obstructions in the fill, and possible cobbles and boulders.
- Bidders must be alerted to the fact that the shale bedrock gets stronger with depth and contains frequent very strong interbeds, and these stronger layers may occur immediately below the bedrock surface.
- Excavation of the bedrock will become more arduous with increasing depth into the deposit and the contractor may have to employ specialized methods such as ripping and pneumatic breaking to dislodge the rock.
- The shale is not durable and will deteriorate rapidly once exposed on the cut face. Therefore, sidewalls in shale bedrock should be provided with protection, such as a shotcrete coating to prevent ravelling and progressive failure, if left exposed for more than 2 to 3 weeks.

The contract documents should contain a Non Standard Special Provision (NSSP) alerting the contract bidders to the possible presence of cobbles and boulders in the till deposits as well as the bedrock conditions. Suggested wording for this NSSP is provided in Appendix E.

The requirements for unwatering during excavation are discussed in Section 12.

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

Excavations should be inspected regularly for evidence of instability if they have been left open for extended periods of time and following periods of heavy rain or thawing. If required, remedial actions must be taken to ensure the stability of the excavation and the safety of workers.

11 ROADWAY PROTECTION

It is anticipated that roadway protection will be required during construction. An item titled "Protection System" as per SP 105S19 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.02.01 and the alignment of the shoring be specified on the contract drawings.

The design of roadway protection should be the responsibility of the Contractor. However, one option that is considered to be suitable for use as temporary shoring at this site is a soldier pile and lagging wall. It is anticipated that the soldier piles will need to be driven or drilled into bedrock to develop the required toe resistance.

All shoring systems should be designed by a Professional Engineer experienced in such designs.

12 UNWATERING

Temporary excavations for construction of footings founded on bedrock are anticipated to be near the groundwater level. Considering the consistency and low permeability of the shale, groundwater control measures such as perimeter ditches and pumping from filtered sumps should be implemented to remove any accumulation of water from the footing base prior to placing concrete. The possibility exists that additional pumps may be required if localized zones of perched water are encountered in the fill or if concentrated seepage is experienced from seams or fractures in the shale bedrock. If footings are founded on shale bedrock, they must be constructed in the dry as shale is prone to rapid deterioration upon exposure to water and air.

The design of the unwatering systems is the responsibility of the Contractor.

13 APPROACH EMBANKMENTS

The proposed BRT will be constructed in a cut at this location and therefore approach embankments to the structure are not required.

14 BACKFILL TO ABUTMENTS

Backfill to the abutments should consist of Granular A or Granular B Type II material meeting the requirements of Special Provision 110F13 "Amendment to OPSS 1010, March 1993". The backfill must be in accordance with OPSS 902 as amended by Special Provision 902S01, and placed to the extents shown in OPSD 3101.150.

Excavated shale is prone to deterioration and is difficult to compact adequately. Therefore, excavated shale must not be used as backfill to the abutments.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with SP105S10. The design of the abutment must include a subdrain as shown in OPSD 3102.100.

15 STATIC EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K(\gamma h + q)$$

Where:

p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 15.1)

γ = unit weight of retained soil (see Table 15.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 15.1.

Table 15.1 – Earth Pressure Coefficients (K)

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.27	0.40*	0.31	0.46*
At rest (Restrained Wall)	0.43	-	0.43	-	0.47	-
Passive	3.7	-	3.7	-	3.3	-

* For wing walls.

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

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

The factors in Table 16.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

16 RETAINING WALLS

Retaining walls are required along the busway cut adjacent to all corners of the structure. The type of retaining system has not been finalized, however we understand that Retained Soil System (RSS) walls and concrete cantilever walls are contemplated. In addition, a contiguous (secant) cantilever caisson wall may be employed on the southeast corner of the structure where space is limited due to the presence of an existing four-storey office building.

Recommendations for design of the proposed wall types are presented in the following sections.

16.1 Cantilever Walls

Design of concrete cantilever walls should be carried out using the foundation design and earth pressure parameters presented for the structure and abutments (Sections 8, 14 and 15).

If lateral movement of the wall is not permissible and/or the wall is restrained from lateral yielding, the at-rest pressure coefficient, K_o , should be used to assess the earth pressures acting on the wall. If the wall design allows lateral yielding (non rigid structure), the active earth pressure coefficient, K_a , may be used.

16.2 Retained Soil Systems

RSS walls used in conjunction with new bridge abutments must be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

The performance of a RSS is dependent on, among other factors, the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. It is critical that the RSS walls are not subject to settlement due to compression of the foundation soils and embankment fill. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

The borehole information indicates that the foundation subgrade at the wall locations will consist of weathered shale bedrock. The shale bedrock is considered suitable for the support of RSS walls.

A wall founded on weathered shale near the level of the cut base (elevation 154.7) should be designed using a factored geotechnical resistance of 750 kPa at ULS. The SLS condition will not govern design of a RSS wall founded on shale bedrock.

The geotechnical resistance provided above is for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2006 Section 6.7.

The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall on shale bedrock may be estimated using an ultimate friction coefficient of 0.5.

The proprietary RSS system must meet the Ministry's specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design. The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

The global stability of the RSS wall is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment. Typically, global stability should not be a major concern for a RSS wall founded on bedrock.

16.3 Caisson Walls

We understand that space adjacent to the cut on the southeast corner of the structure is limited due to the presence of an existing four-storey office building, and therefore construction of an RSS or a conventional concrete cantilever wall with granular backfill may not be a favoured option. A contiguous (secant) cantilever caisson wall may be employed at this location.

For caisson wall design, the lateral earth pressures acting on the wall will be governed by the characteristics of the existing fill, clay till and shale bedrock behind the wall. The earth pressures should be computed using the equation presented in Section 15, using the following geotechnical parameters:

Table 16.1 – Earth Pressures Parameter for Existing Materials

Parameter	Retained Material		
	Silty Clay Fill	Silty Clay Till	Weathered Shale
Friction Angle (degrees)	28	30	30
Unit Weight (kN/m ³)	19	20	25
Active Pressure Coefficient	0.36	0.33	0.33
At-Rest Pressure Coefficient	0.53	0.50	0.50
Passive Pressure Coefficient	2.8	3.0	3.0

To minimize displacements behind the caisson wall, it is recommended that the earth pressure coefficient at-rest be employed to compute the lateral pressure acting on the back of the wall.

Undisturbed shale bedrock in the area has been known to possess horizontal stresses considerably higher than the vertical overburden pressure present in the rock. In addition, shale bedrock is often susceptible to swell induced by creep and changing salinity of the groundwater. These factors may impose significant pressures on a caisson wall supporting an excavation extending below the weathered zone of the shale. At this site however, horizontal

stresses and swell pressures are considered unlikely to be an issue in view of the weathered condition of the rock within the excavation depths, the presence of previous nearby excavations, and the low swell potential identified by laboratory testing of shale samples elsewhere on the BRT project. Therefore, design based on the nominal pressures computed using a triangular pressure distribution as outlined above (Table 17.1), is considered adequate.

If the design includes a sloping ground surface behind the wall, the earth pressure parameters will require modification. Thurber should be contacted to provide appropriate coefficients in this case.

Resistance to lateral movement of a caisson wall will be provided by the passive pressure developed on the face of the primary caisson sockets embedded in the shale bedrock below the frost depth. The lateral resistance that can be mobilized in front of the caisson socket, assuming a primary caisson spacing of one caisson diameter, may be computed using the coefficient of horizontal subgrade reaction k_s and ultimate lateral resistance p_{ult} estimated as follows:

$$\begin{aligned} k_s &= 30,000 \text{ kN/m}^3 \text{ at the bedrock surface, increasing linearly to} \\ &\quad 100,000 \text{ kN/m}^3 \text{ at a depth of 3 caisson diameters and below} \\ p_{ult} &= 400 \text{ kPa at the bedrock surface, increasing linearly to} \\ &\quad 2,000 \text{ kPa at a depth of 3 caisson diameters and below} \end{aligned}$$

The above equations and recommended parameters may be used to analyze the interaction between a caisson and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K , for analysis may be obtained by the expression, $K = k_s * L * D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the caisson diameter (m) and L is the length (m) of the caisson segment or element used in the analysis. The ultimate lateral resistance on any one segment of caisson, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} * L * D$. This represents the ultimate load at which the caisson fails and will not support any additional load at greater displacements.

The lateral displacements of the cantilever caisson wall should be analysed to confirm that the movements behind the wall can be tolerated by the nearby building foundations and any adjacent utilities. The building foundation design, including the foundation type, founding level, founding stratum, and distance from the wall must be determined for this analysis. If ground movements exceed tolerable levels at the facility location, measures such as anchors or permanent overhead struts should be employed to reduce the displacements.

To achieve proper interlock and minimize displacements behind the caisson wall during installation, the following procedures are recommended for construction of the contiguous (secant) caisson wall:

- i. The infill (intermediate) caissons should be installed first. These caissons should be extended a nominal distance of approximately 0.6 m below the rock surface at the base of the excavation.
- ii. The primary caissons (“king piles”) should then be installed within one to three days of the adjoining infill caissons, timed such that the concrete in the infill caissons has attained adequate strength to remain intact yet be easily drilled.
- iii. The caissons should be filled with concrete the same day as drilling to minimize displacements in the adjacent soils and avoid deterioration of the shale socket.
- iv. To minimize the potential for instability or lateral movement of the soils adjacent to the wall, caisson installation should be staged so that all caissons in a section are not left open simultaneously, effectively forming a trench.

The glacial till above the shale bedrock is very stiff to hard at this site and augering for caisson installation may be laboured. The caisson installation equipment should be able to dislodge and remove any obstructions or cobbles and boulders in the fill or clay till.

The shale becomes stronger with depth and contains hard interbeds of limestone, siltstone or calcareous shale that may slow drill production and/or require the use of specialized equipment to penetrate. The use of coring or breaking equipment may be required to penetrate hard interbedded limestone layers while advancing the socket in the shale bedrock.

17 SEISMIC CONSIDERATIONS

17.1 Seismic Design Parameters

The following seismic parameters should be used for design:

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

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

17.2 Liquefaction Potential

The foundation soils at the site are assessed as not being prone to liquefaction.

17.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active and passive earth pressure coefficients that incorporate the effects of earthquake loading.

For the design of retaining walls, the coefficients of horizontal earth pressure in Table 17.1 provided at the end of the report text may be used.

18 BURIED UTILITIES

The BRT alignment follows transportation and utility corridors containing roadways, bridges, transmission towers, underground utilities and pipelines. It is recommended that the exact locations and elevations of these utilities be established by the designer. The impact of excavation and resulting ground deformations on facilities located in close proximity to or crossing the busway must be assessed and measures to control impacts included in design. In general, structures and utilities located above a line extending upwards from the outer edges of the base of the excavation at an inclination of 1.25H:1V should be considered.

The settlement and displacement/rotation tolerances of these structures and utilities should also be obtained by the designer from the owner/designer of these structures and utilities.

If buried utilities or any existing structure are present within the zone of influence of the BRT excavation, it is recommended that the following be carried out prior to the commencement of construction:

- Carry out pre-construction condition survey including documentation of any existing distress associated with the structure or the utilities. Any distress should be reported to and discussed with the utility or structure owner.
- Implement an instrumentation and monitoring program to include vibration and settlement monitoring during construction. Establish review and alert level criteria for allowable settlement and lateral movement following discussions with the owner of the structure or utility. Establish and agree on remedial action, if required, with the utility or structure owner prior to start of construction.
- Carry out post-construction condition survey of the utilities or structure.

Relocation of, and/or special protective measures for, some of these affected utilities may be required.

19 CONSTRUCTION CONCERNS

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

1. Groundwater control

Perched water may be encountered within the fill soils. Seepage may be experienced from seams in the till soils or fractures in the shale bedrock. Proper groundwater and surface water control measures must be in place prior to commencing excavation. All footings must be placed in the dry.

2. Excavations

Cobbles, boulders, slabs of shale and hard limestone rock should be expected within the till layer during foundation excavation.

The shale becomes stronger with depth and contains hard interbeds of limestone, siltstone or calcareous shale that may slow production and/or require the use of specialized equipment to penetrate. Excavation of the shale bedrock may require the use of rock excavation methods such as pneumatic rock breakers to penetrate hard limestone interbeds.

3. Shale bedrock protection

Care must be exercised during excavation to avoid disturbing the founding subgrade. The exposed subgrade rock should be expeditiously inspected, approved and protected from disturbance. To prevent softening and degradation of the shale, exposed bearing surfaces must be protected by placement of a mud slab within 24 hours of completion.

4. Caisson installation

The glacial till above the shale bedrock is very stiff to hard at this site and augering for caisson installation may be laboured. The caisson installation equipment should be able to dislodge and remove any obstructions or cobbles and boulders in the fill or clay till.

In addition to drilling, the use of coring or breaking equipment may be required to penetrate hard interbedded limestone layers while advancing the socket in the shale bedrock.

5. Existing underground utilities and roadways

Care must be taken during footing excavation to avoid disturbing and undermining travelled lanes of the roadways, existing underground utilities and nearby structure foundations.

20 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Murray R. Anderson, P.Eng and Ms. R. Palomeque Reyna, P.Eng.

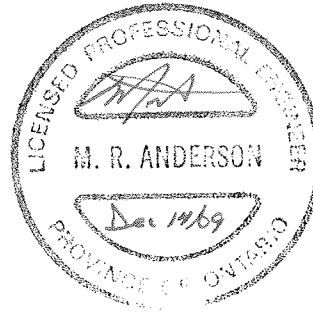
The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

Thurber Engineering Ltd.
Rocio Palomeque Reyna, P.Eng.
Geotechnical Engineer



Murray R. Anderson, P.Eng.
Senior Foundations Engineer



P.K. Chatterji, P.Eng.
Review Principal

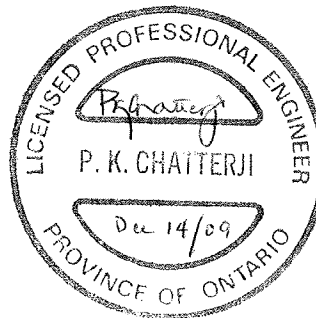


Table 17.1 – Earth Pressure Coefficient for Earthquake Loading

Wall Condition	Earth Pressure Coefficient (K) for Earthquake Loading					
	Granular A or Granular B Type II $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$		Silty Clay Fill $\phi = 28^\circ; \gamma = 19 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.28	0.47	0.32	0.59	0.37	0.84
Passive (K_{PE})	3.6	-	3.1	-	2.7	-
At Rest (K_{OE})**	0.54	-	0.58	-	0.64	-
					0.34	0.79
					2.9	-
					0.61	-

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

** After Woods

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$



Water Level

C_{pen}


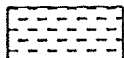
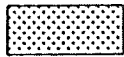


Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS			
Fresh (FR)	No visible signs of weathering.				
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE		
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE		
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE		
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL		
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)		
DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
TERMS		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.				
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				



RECORD OF BOREHOLE No 09-005

1 OF 2

METRIC

G.W.P. 19-1351-160 LOCATION N 4 828 307.9 E 609 577.0 ORIGINATED BY ES
 HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN
 DATUM DATE 2009.03.13 - 2009.03.13 CHECKED BY LT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			
						\circ UNCONFINED + FIELD VANE \bullet QUICK TRIAXIAL x LAB VANE	WATER CONTENT (%) w _p w w _L						
160.8													
0.9	TOPSOIL, trace roots and rootlets (50mm)		1	AS									
160.1	SAND and GRAVEL, some clay Brown Moist (FILL)												
0.7	Silty CLAY, some sand, trace gravel, occasional organics Firm to Stiff Brown (FILL) Layer of sand (200mm) Occasional brick fragments		1	SS	10								
			2	SS	6								
			3	SS	9								
			4	SS	12								
	Sandy zone		5	SS	80/ .200								
156.7	SHALE, highly weathered, thinly bedded, very weak to medium strong, with medium strong to very strong limestone interbeds Grey		6	SS	50/ .100								
4.1			7	SS	50/ .100								
	Limestone layer (greater than 50mm): 50mm at 6.6m 65mm at 6.9m Rubble zone (100mm) at 6.4m		1	RUN									
	Becoming slightly weathered		2	RUN									
	Clay seam ((25mm) at 8.1m		3	RUN									

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15
10


(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-006

1 OF 1

METRIC

G.W.P. 19-1351-160 LOCATION N 4 828 327.0 E 609 564.9 ORIGINATED BY ES
 HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM DATE 2009.03.13 - 2009.03.13 CHECKED BY LT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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								
160.7 0.0 0.1	ASPHALT (75mm)		1	AS												
160.0 0.7	SAND and GRAVEL, some silt Brown Moist (FILL)		1	SS	25											
	Silty CLAY, some sand, trace gravel Very Stiff to Stiff Brown (FILL)		2	SS	12											
	Occasional asphalt fragments		3	SS	11											
157.7 3.0	Silty CLAY, some sand, trace gravel, trace limestone and shale fragments, occasional oxide lenses Hard Brown (TILL)(Cl)		4	SS	35										5 16 54 25	
156.9 3.8	SHALE, highly weathered, occasional limestone interbeds Grey		5	SS	70/ .175											
			6	SS	50/ .075											
154.6 6.1	END OF BOREHOLE AT 6.1m. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE TO 3.0m, THEN CUTTINGS TO 0.1m THEN ASPHALT TO SURFACE.		7	SS	50/ .025											

ONTMT4S 1.160(MTO).GPJ 6/12/09

RECORD OF BOREHOLE No 09-007

1 OF 1

METRIC

G.W.P. 19-1351-160 LOCATION N 4 828 304.2 E 609 558.7 ORIGINATED BY ES
 HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM DATE 2009.03.13 - 2009.03.13 CHECKED BY LT

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			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
161.1															
0.0	PAVING STONE (50mm)														
160.5	SAND and GRAVEL, some silt Brown Moist (FILL)		1	AS											
0.7															
159.9	Silty CLAY, some sand, trace gravel, occasional cobbles Very Stiff Brown (FILL)		1	SS	70/ .225										
1.2															
	SAND, trace gravel, occasional cobbles Very Dense to Compact Brown Moist (FILL)		2	SS	50/ .075										
158.3			3	SS	24										27 52 15 6
2.8	Silty CLAY, sandy, trace gravel Very Stiff to Hard Brown (FILL)														
			4	SS	20										
	Occasional oxide staining														
			5	SS	72										0 13 58 29
156.7															
4.5	SHALE, highly weathered, thinly bedded, occasional limestone interbeds Grey		6	SS	56/ .150										
155.0			7	SS	50/ .050										
6.1	END OF BOREHOLE AT 6.15m. BOREHOLE OPEN AND DRY UPON COMPLETION. Piezometer installation consists of 19mm diameter pipe. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2009.04.14 5.9 155.2 2009.05.05 5.5 155.6 2009.05.21 5.7 155.4														

+³ . X³ : Numbers refer to
Sensitivity

20
15
10



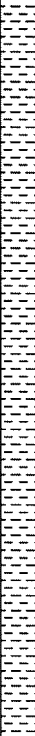
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-008

1 OF 2

METRIC

G.W.P. 19-1351-160 LOCATION N 4 828 323.4 E 609 551.5 ORIGINATED BY LG
 HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN
 DATUM DATE 2009.03.14 - 2009.03.14 CHECKED BY LT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L			
161.0						20	40	60	80	100									
0.0	SAND and GRAVEL , some silt Brown Moist (FILL)		1	AS															
160.3																			
0.7			Silty CLAY , some sand, trace gravel, occasional limestone fragments Stiff to Hard Brown (FILL)	1	SS	12													
	2	SS		42															
			3	SS	13														
158.0																			
3.0	Silty CLAY , some sand, trace gravel, occasional limestone and shale fragments Very Stiff to Hard Brown (TILL)		4	SS	25														
			5	SS	50/														
156.9																			
4.1	SHALE , highly weathered, thinly bedded, weak to medium strong, with medium strong to very strong limestone interbeds, grey Rubble zones: 300mm at 4.7m 50mm at 7.4m 50mm at 8.9m Clay seams: 50mm at 5.3m 100mm at 5.5m 50mm at 5.8m 50mm at 7.1m Becoming moderately weathered Limestone layers (greater than 50mm): 150mm at 7.5m 50mm at 9.3m		1	RUN															
			2	RUN															
			3	RUN															
			4	RUN															
			5	RUN															

Continued Next Page

+³, x³: Numbers refer to Sensitivity
 20
 15 5
 10 (%) STRAIN AT FAILURE

ONTMT4S 1160(MTO).GPJ 6/12/09

RECORD OF BOREHOLE No 09-008

2 OF 2

METRIC

G.W.P. 19-1351-160 LOCATION N 4 828 323.4 E 609 551.5 ORIGINATED BY LG
HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN
DATUM DATE 2009.03.14 - 2009.03.14 CHECKED BY LT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
	Continued From Previous Page							20 40 60 80 100							
	SHALE, highly weathered, thinly bedded, weak to medium strong, grey							○ UNCONFINED + FIELD VANE							
	Clay zone (50mm) at 10.4m							● QUICK TRIAXIAL × LAB VANE							
	Limestone layers (greater than 50mm):							WATER CONTENT (%)							
	250mm at 10.5m							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT							
	50mm at 11.4m							W P W W L							
	50mm at 11.8m														
	150mm at 11.9m														
	250mm at 12.9m														
	220mm at 13.3m														
	50mm at 13.6m														
147.4			7	RUN			151							1	
														4	
														3	RUN 7#
							150							3	TCR=90%,
														0	SCR=90%,
														1	RQD=90%,
														0	UCS=26 to
							149							1	116MPa
														0	(Limestone)
														1	
			8	RUN										0	RUN 8#
														1	TCR=100%,
														0	SCR=100%,
														1	RQD=100%,
														1	UCS=3MPa
							148							1	(Shale)
														1	
13.6	END OF BOREHOLE AT 13.6m. Piezometer installation consists of 19mm diameter pipe. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2009.04.14 9.3 151.7 2009.05.05 6.6 154.4 2009.05.21 8.5 152.5														

ONTMT4S 1160(MTO),GPJ 12/14/09

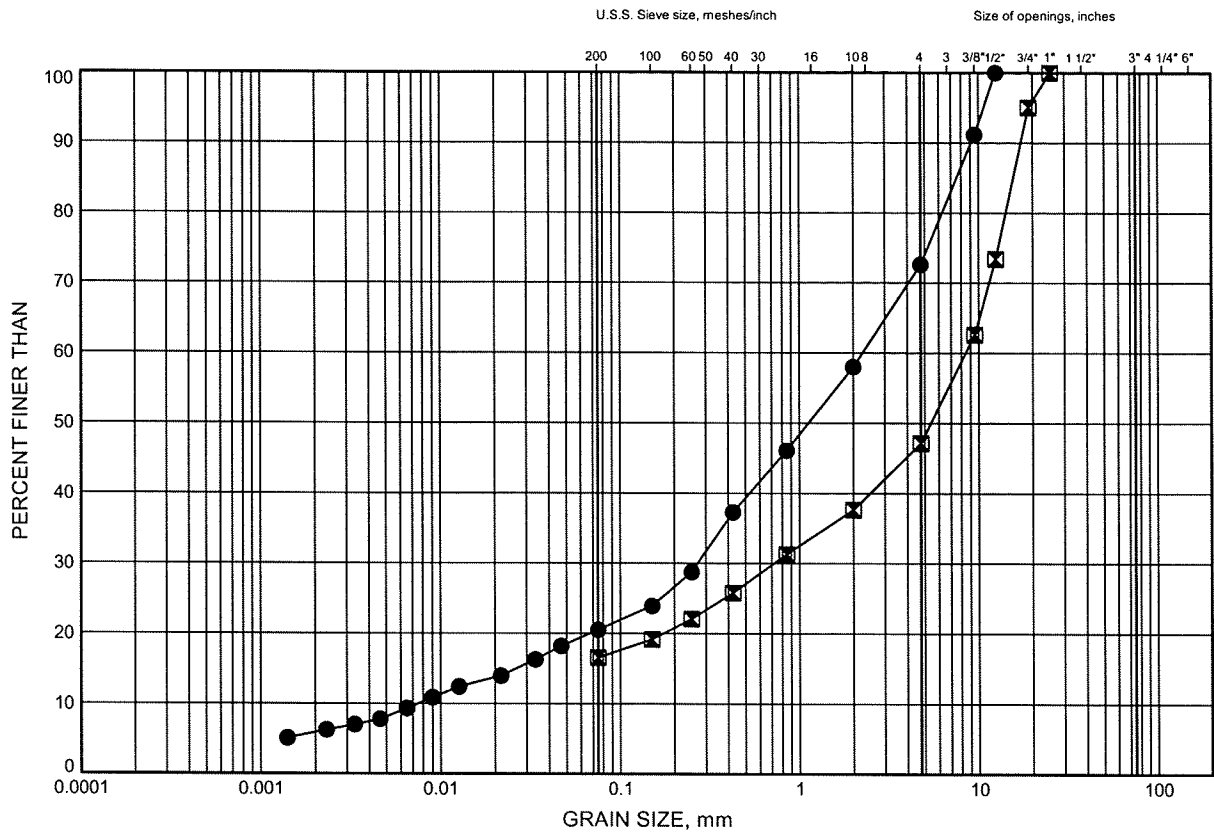
Appendix B

Laboratory Test Results

Mississauga BRT East
GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND/GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-007	2.55	158.60
☒	09-008	0.30	160.68

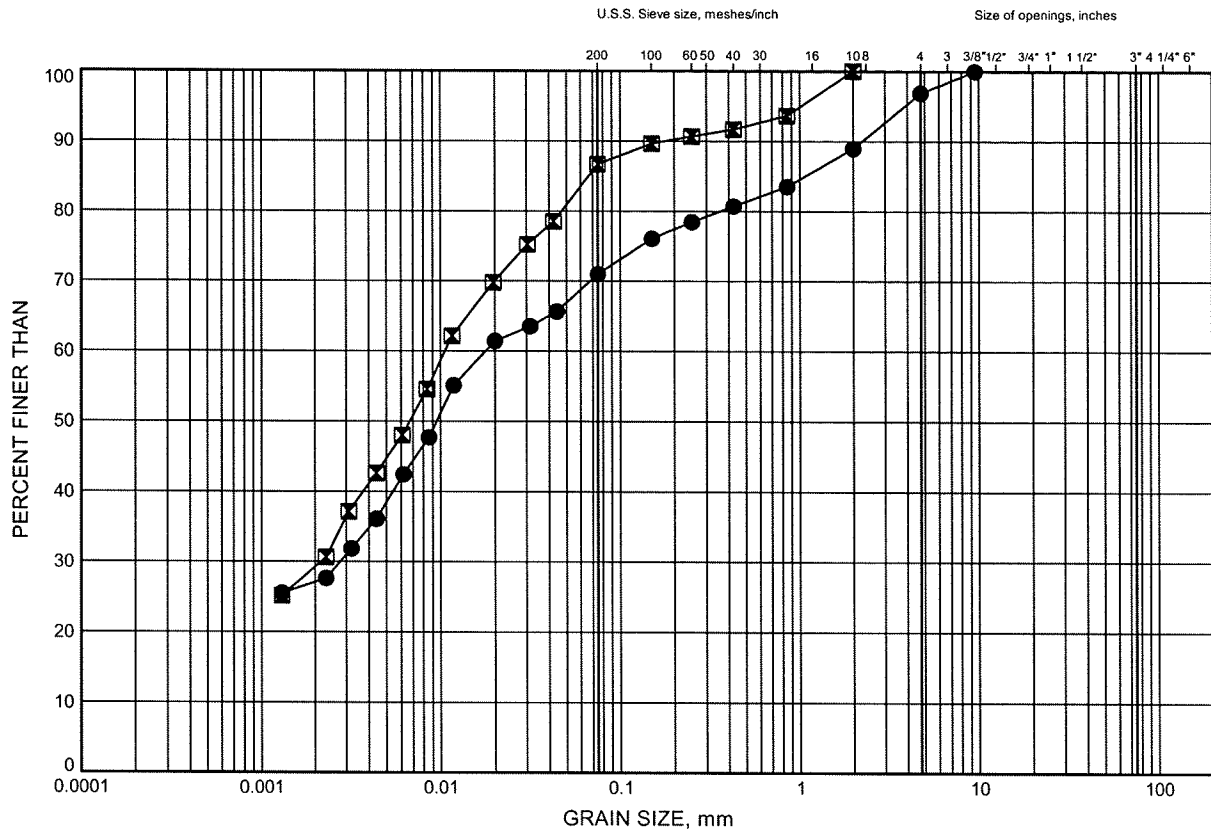


W.P.# 19-1351-160.....
Prepared By MFA.....
Checked By MRA.....

Mississauga BRT East GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty CLAY FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-005	3.96	156.87
⊠	09-007	4.04	157.11

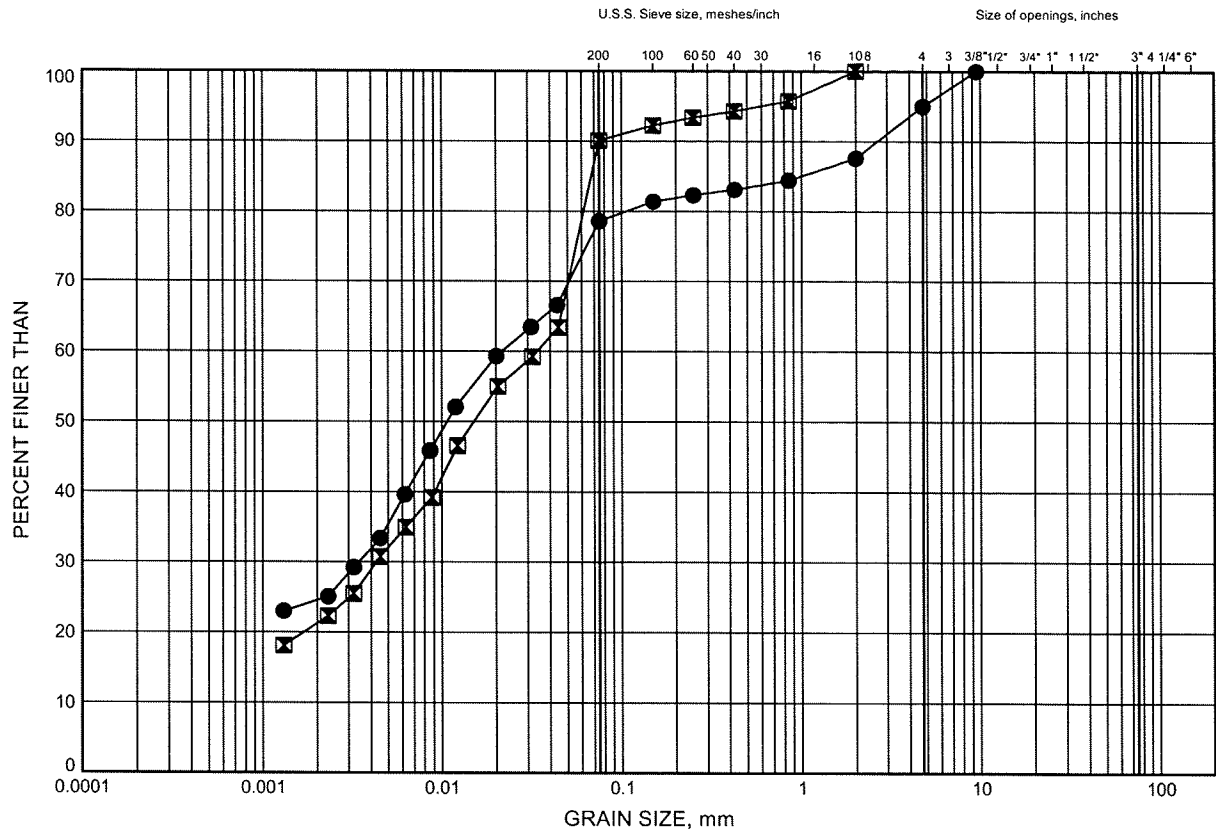


W.P.# 19-1351-160
Prepared By MFA
Checked By MRA

Mississauga BRT East GRAIN SIZE DISTRIBUTION

FIGURE B3

Silty CLAY TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-006	3.35	157.36
◻	09-008	3.31	157.67

GRAIN SIZE DISTRIBUTION - THURBER 1160(MTO).GPJ 6/12/09

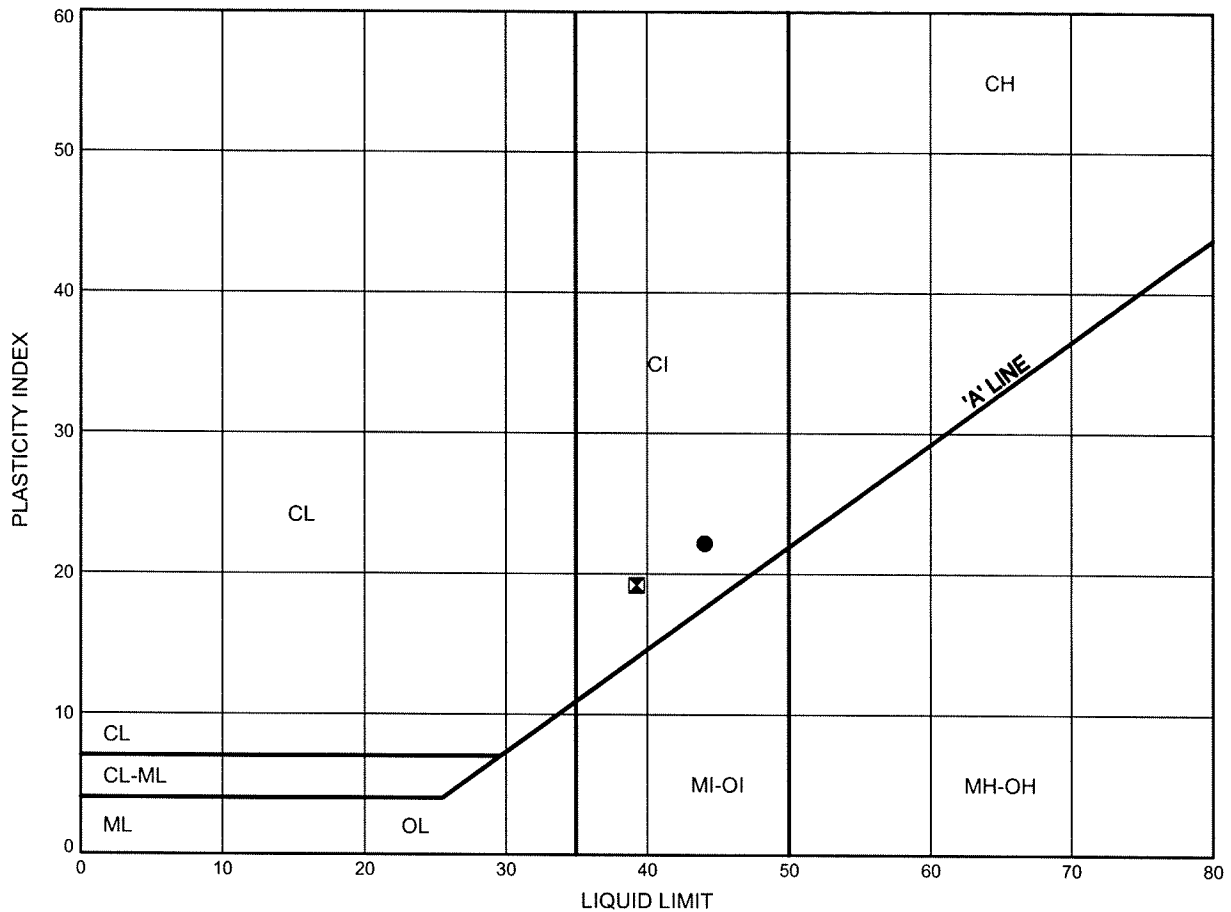
W.P.# 19-1351-160.....
Prepared By MFA.....
Checked By MRA.....



Mississauga BRT East
ATTERBERG LIMITS TEST RESULTS

FIGURE B4

Silty CLAY FILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	09-005	3.99	156.84
⊠	09-007	4.19	156.96

Date June 2009
 Project 19-1351-160

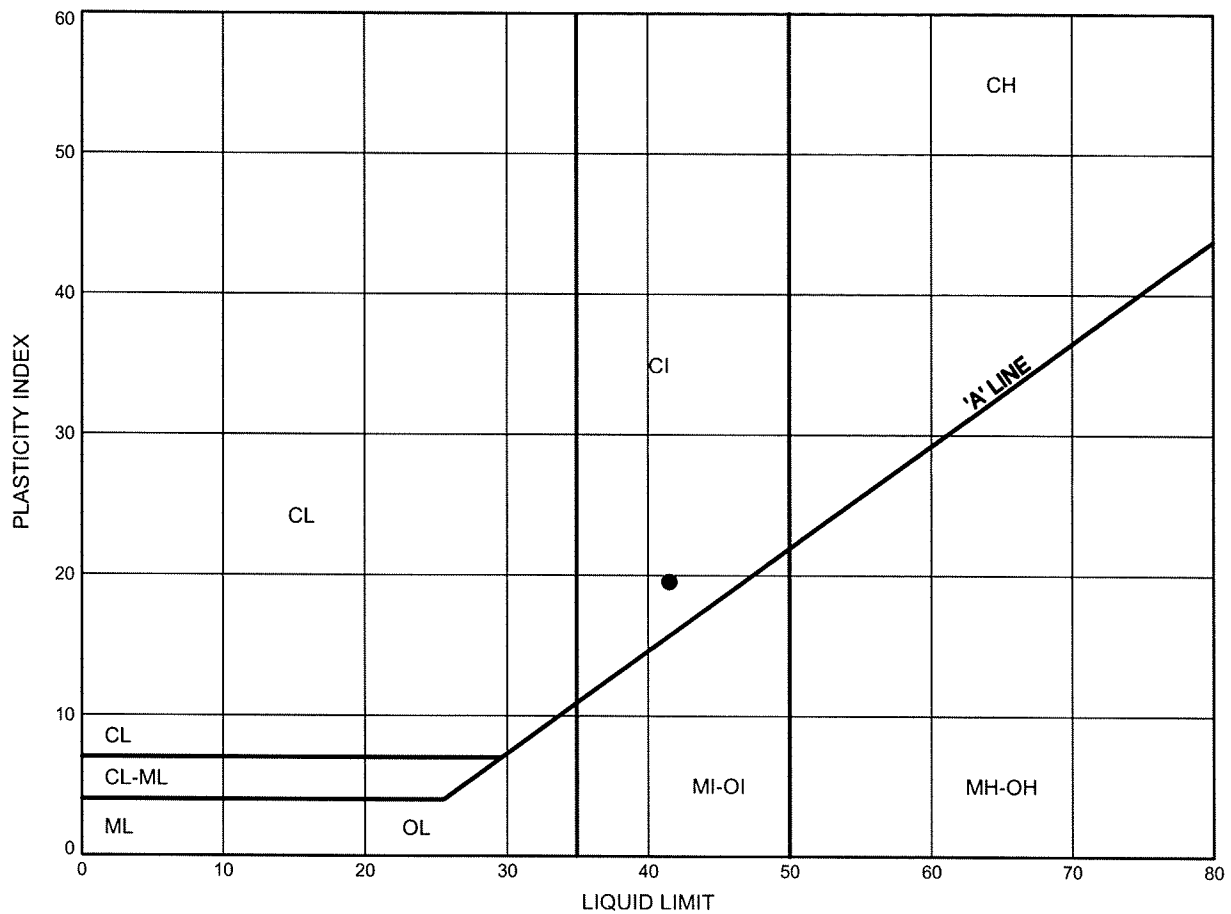


Prep'd MFA
 Chkd. MRA

Mississauga BRT East
ATTERBERG LIMITS TEST RESULTS

FIGURE B5

Silty CLAY TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	09-006	3.35	157.36

Date June 2009
 Project 19-1351-160



Prep'd MFA
 Chkd. MRA

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-04

SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1116-0011	RUN NUMBER	6
BOREHOLE NUMBER	09-8	SAMPLE DEPTH, m	9.73-9.91

TEST CONDITIONS

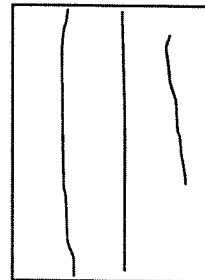
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.53

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	12.26	WATER CONTENT, (specimen) %	1.20
SAMPLE DIAMETER, cm	4.85	UNIT WEIGHT, kN/m ³	24.91
SAMPLE AREA, cm ²	18.47	DRY UNIT WT., kN/m ³	24.61
SAMPLE VOLUME, cm ³	226.50	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	575.54	VOID RATIO	0.08
DRY WEIGHT, g	568.72		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	74.6
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REMARKS:

DATE: 4/8/2009

Checked By: *ML*

Golder Associates

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-04

SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1116-0011	RUN NUMBER	2
BOREHOLE NUMBER	09-5	SAMPLE DEPTH, m	7.90-8.05

TEST CONDITIONS

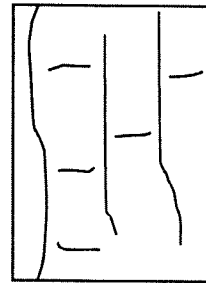
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.27

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.93	WATER CONTENT, (specimen) %	2.30
SAMPLE DIAMETER, cm	4.82	UNIT WEIGHT, kN/m ³	24.45
SAMPLE AREA, cm ²	18.25	DRY UNIT WT., kN/m ³	23.90
SAMPLE VOLUME, cm ³	199.44	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	497.33	VOID RATIO	0.11
DRY WEIGHT, g	486.15		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	51.2
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REMARKS:

DATE: 4/8/2009

Checked By: *YH*

Golder Associates

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-04

SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1116-0011	RUN NUMBER	4
BOREHOLE NUMBER	09-5	SAMPLE DEPTH, m	10.79-10.90

TEST CONDITIONS

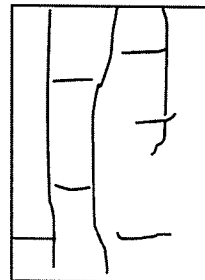
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.05

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.89	WATER CONTENT, (specimen) %	3.70
SAMPLE DIAMETER, cm	4.82	UNIT WEIGHT, kN/m ³	24.49
SAMPLE AREA, cm ²	18.21	DRY UNIT WT., kN/m ³	23.62
SAMPLE VOLUME, cm ³	180.09	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	449.94	VOID RATIO	0.12
DRY WEIGHT, g	433.89		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	9.0
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REMARKS: Sample broken upon arrival.

DATE: 4/8/2009

Checked By: *hli*

Golder Associates

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Engineered Fill	Footings on Shale Bedrock	Driven Piles	Caissons
<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower geotechnical resistance in soil than in bedrock ii. Native soil layer is thin, discontinuous and provides variable resistance. iii. Proposed busway cut extend below the level of till. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. ii. Founding level can be adjusted. iii. Higher bearing resistance than native soils. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Cost of engineered fill placement. ii. No advantage to excavating shale to place engineered fill. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High values of geotechnical resistance are available on the bedrock. ii. Relatively simple construction method. iii. Base of subway cut will lie within shale. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. More likely to encounter groundwater than footing on native soils. ii. Cannot accommodate integral abutments. <p>RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance in shale. ii. Installation of piles could continue in freezing weather. iii. Accommodates integral abutment design. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Not practical to drive piles into shale bedrock. iii. Potential difficulties penetrating hard limestone layers in shale iv. Pre-drilling will be required in order to install the piles to adequate length. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded in bedrock. ii. Construction of caissons could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost than spread footings which already obtain resistance on shale. ii. Potential difficulties penetrating hard limestone layers in shale. iii. More likely to encounter groundwater. iv. Potential difficulty in cleaning and inspecting bases. <p>NOT RECOMMENDED</p>

Appendix D

List of SPs and OPSS

Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- SP 105S10
- SP 105S19
- SP 110F13
- SP 572S01
- SP 902S01
- OPSS 902
- OPSS 1010
- OPSD 3101.150
- OPSD 3102.100

2. Suggested Text for NSSP on “Native Hard Till” and “Rock Excavation” and Dewatering

Cobbles, boulders and slabs of shale and hard limestone rock should be expected within the silty clay till layer. The silty clay till grades into weathered bedrock, accordingly, excavation of the lower zones of the till may be arduous.

The strength of the shale bedrock increases with depth and there is presence of very hard limestone and/or siltstone interbeds within the shale bedrock. Bulk excavation through the sound shale and the hard interbeds will be difficult. Rock coring equipment, pneumatic rock splitting/breaking equipment and ripping machinery should be available on site to assist in excavation and drilling.

Seepage may be experienced from localized seams or fractures in the rock. Means to handle this seepage, such as additional pumps, should be made available.

3. Suggested Text for NSSP on Contiguous Caisson Wall Installation

The glacial till above the shale bedrock is very stiff to hard at this site and augering for caisson installation may be laboured. The caisson installation equipment should be able to dislodge and remove any obstructions or cobbles and boulders in the fill or clay till.

The shale becomes stronger with depth and contains hard interbeds of limestone, siltstone or calcareous shale that may slow drill production and/or require the use of specialized equipment to penetrate. The use of coring or breaking equipment may be required to penetrate hard interbedded limestone layers while advancing the socket in the shale bedrock.

To achieve proper interlock and minimize displacements behind the caisson wall during installation, the following procedures are recommended for construction of the contiguous (secant) caisson wall:

- i. The infill (intermediate) caissons should be installed first. These caissons should be extended a nominal distance of approximately 0.6 m below the rock surface at the base of the excavation.
- ii. The primary caissons (“king piles”) should then be installed within one to three days of the adjoining infill caissons, timed such that the concrete in the infill caissons has attained adequate strength to remain intact yet be easily drilled.
- iii. The caissons should be filled with concrete the same day as drilling to minimize displacements in the adjacent soils and avoid deterioration of the shale socket.
- iv. To minimize the potential for instability or lateral movement of the soils adjacent to the wall, caisson installation should be staged so that all caissons in a section are not left open simultaneously, effectively forming a trench.

Appendix E

Site Photographs

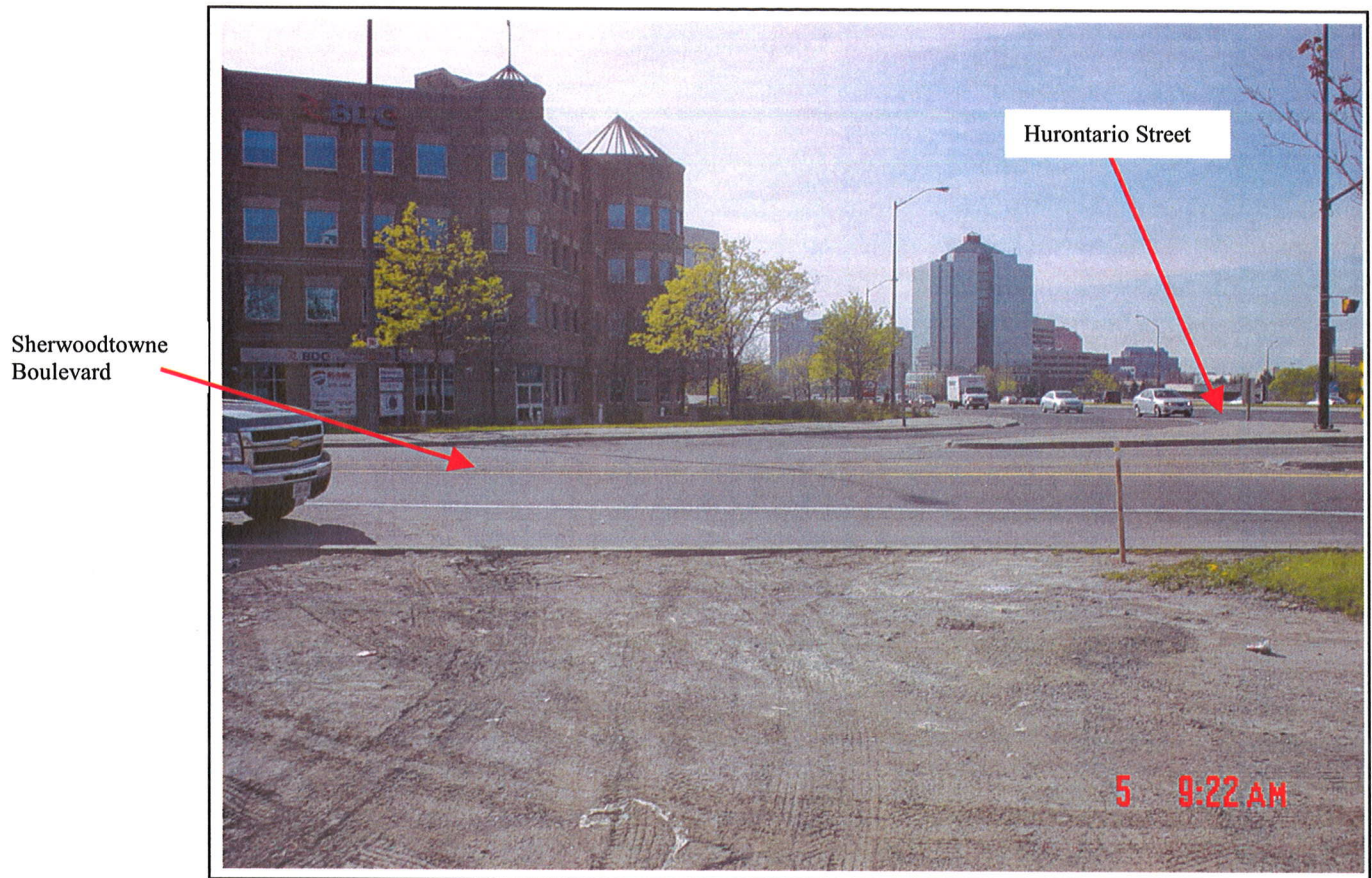


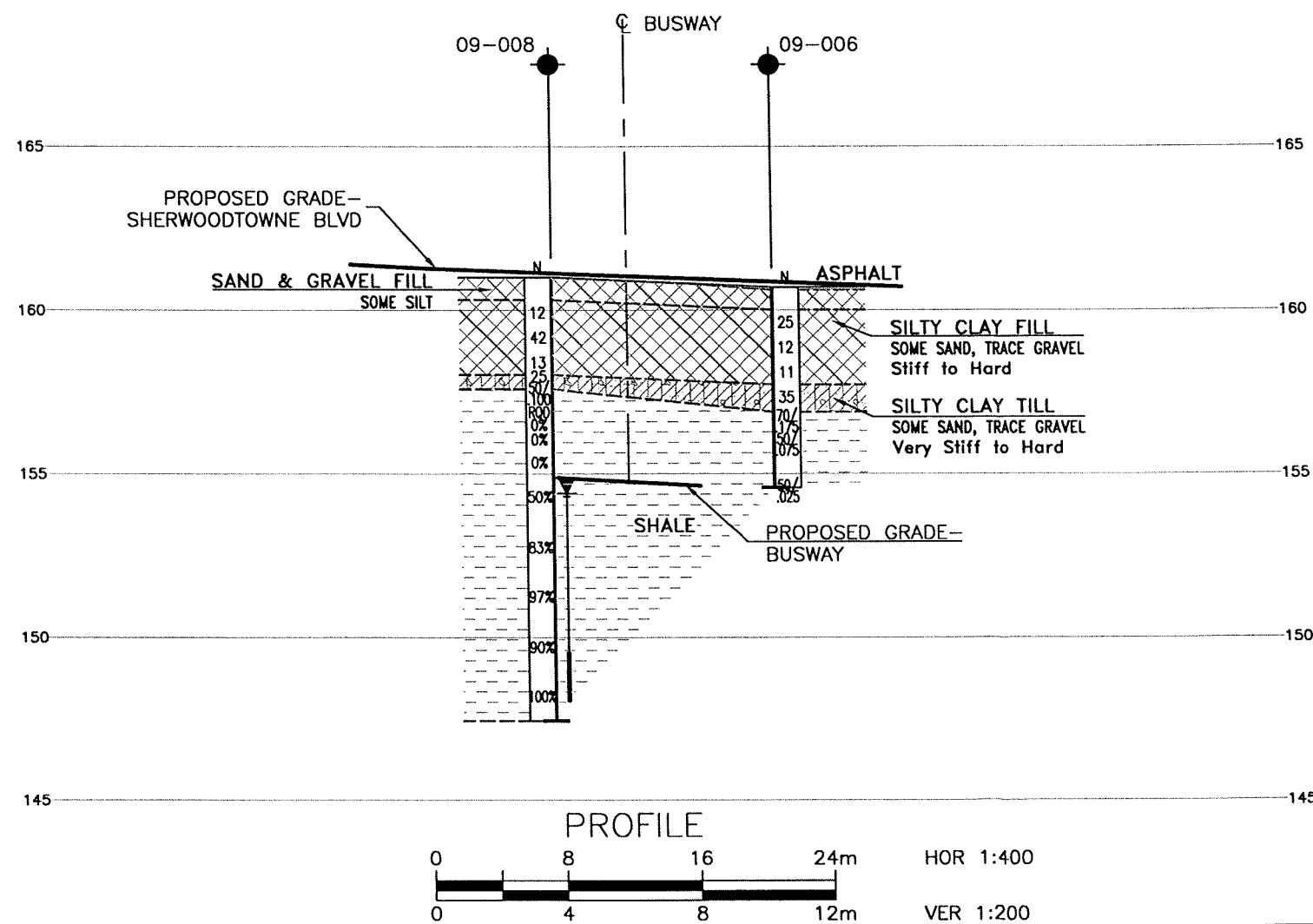
Photo 1. Looking south from the north side of Sherwoodtowne Boulevard, just east of Hurontario Street



Photo 2. Looking northwesterly from south side of Sherwoodtowne Boulevard just east of Hurontario Street.

Appendix F

Borehole Locations and Soil Strata Drawing



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

CONT No
GWP No

MISSISSAUGA BRT EAST
DETAILED DESIGN
SHERWOODTOWNE BLVD
SOIL STRATA

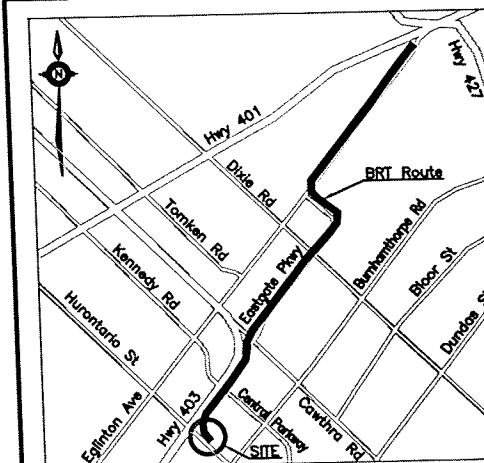
SHEET



**McCORMICK RANKIN
CORPORATION**








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GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



KEYPLAN

LEGEND

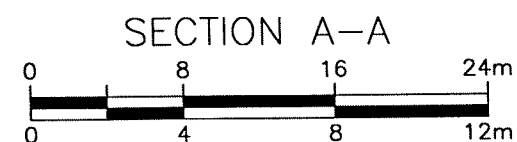
	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
09-005	160.8	4 828 307.9	609 577.0
09-006	160.7	4 828 327.0	609 564.9
09-007	161.1	4 828 304.2	609 558.7
09-008	161.0	4 828 323.4	609 551.5

-NOTES-

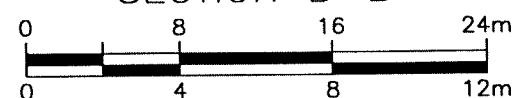
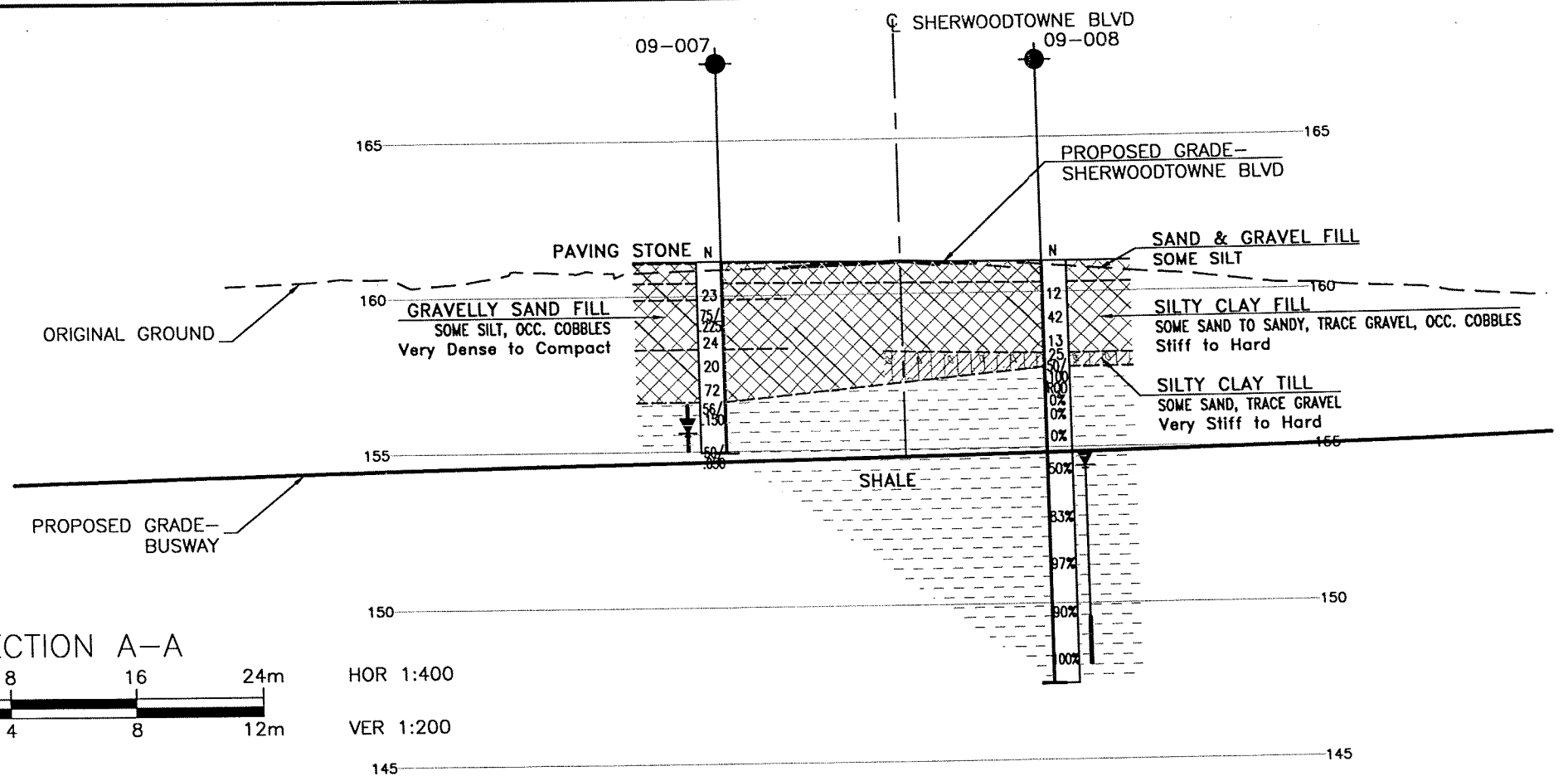
- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 30M12-287

[illegible]

HOR 1:400

VER 1:200



HOR 1:400

VER 1:200

