FOUNDATION INVESTIGATION AND DESIGN REPORT
CAWTHRA ROAD GRADE SEPARATION STRUCTURE
MISSISSAUGA BUS RAPID TRANSIT (BRT) PROJECT
MISSISSAUGA, ONTARIO

Geocres Number: 30M12-286

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

December 14, 2009
File: 19-1351-160

H:\19\1351\160 Mississauga BRT Detailed Design\Reports & Memos\Cawthra\191351-160_Cawthra_FIDR_Final.doc

TABLE OF CONTENTS

Appendices

FOUNDATION INVESTIGATION AND DESIGN REPORT
CAWTHRA ROAD GRADE SEPARATION STRUCTURE
MISSISSAUGA BUS RAPID TRANSIT (BRT) PROJECT
MISSISSAUGA, ONTARIO

Geocres Number: 30M12-286

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 Cawthra Road in Mississauga, Ontario. The proposed structure will be located adjacent to the north side of the Cawthra Road and Eastgate Parkway intersection, east of Highway 403.

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 and approach embankments 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 Cawthra Road will include the construction of a bus station and a grade separation structure to carry the proposed BRT under Cawthra Road.

The site is located on the north side of the Cawthra Road and Eastgate Parkway intersection; approximately 250 m east of Highway 403.

Currently, the lands adjacent to the Cawthra Road and Eastgate Parkway intersection comprise utility corridors with overhead transmission lines and buried pipelines. Vegetation consists mainly of tall grass and shrubs.

Lands located on the west and east sides of Cawthra Road, approximately 150 m south of the intersection, are generally residential and commercial.

Photographs of the site included in Appendix F, 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 till (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 from March 11 to 13, 2009. The field program consisted of drilling and sampling four boreholes (numbered 09-52 to 09-55) for the proposed grade separation structure.

Boreholes 09-052 to 09-054 were terminated at depths ranging from 12.6 m to 15.4 m (Elevations 135.9 to 138.9). Borehole 09-055 encountered auger refusal in silty clay till at 15.3 m depth on a possible boulder or slab of hard limestone and was advanced below this depth by coring to 19.8 m depth (Elevation 131.5).

The approximate borehole locations are shown on the Borehole Locations and Soil Strata Drawing in Appendix G. The coordinates and elevations of the boreholes are given on these drawings and on the individual Record of Borehole Sheets in Appendix A. For this report, Cawthra Road was considered to be running in a north-south direction.

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. 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 advance Borehole 09-055 when auger refusal was encountered on a rock slab within the silty clay till.

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.

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 (Boreholes 09-052 and 09-055) 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
North Abutment	09-052	12.2/139.3	Piezometer with 1.5 m slotted screen installed with sand filter to 7.6 m, bentonite from 7.6 m to ground surface.
	09-053	None installed	Bentonite to surface.
South Abutment	09-054	None installed	Bentonite to 0.1 m, then asphalt to surface.
	09-055	19.8/131.5	Piezometer with 1.5 m slotted screen installed with sand filter to 16.8 m, then bentonite from 16.8 m to ground surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and moisture content determinations. At least 25% of the recovered samples of soil were also subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg Limit testing where appropriate. 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.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil 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 pavement structure overlying fill which is underlain by native silty clay till and sand and silt till. More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

Pavement structure consisting of approximately 50 mm to 100 mm of asphalt overlying granular (sand and gravel) road base was encountered surficially in Boreholes 09-052 to 09-054 drilled on Cawthra Road. The thickness of granular fill measured in the boreholes ranged from 0.5 m to 0.75 m and the underside lay at elevations ranging from 150.6 to 150.7.

5.2 Fill

A 600-mm thick layer of sand and gravel fill was contacted surficially in Borehole 09-055.

Fill consisting of silty clay containing some sand, trace gravel and occasional rootlets was encountered below the sand and gravel fill in Borehole 09-055 and below the granular road base in Boreholes 09-052 to 09-054.

The thickness of the silty clay fill ranged from 1.4 m to 2.4 m.

The depth to the base of the silty clay fill ranges from 2.2 m to 3.0 m (Elevations 148.3 to 149.3).

Based on SPT 'N' values ranging from 6 to 17 blows for 0.3 m of penetration, the silty clay fill is described as firm to very stiff in consistency. The natural moisture contents of the samples recovered from the silty clay fill layer ranged from 8% to 19%. Moisture contents measured in the sand and gravel fill samples ranged from 2% to 9%.

5.3 Silty Clay Till

Native brown to grey silty clay till was contacted below silty clay fill in all the boreholes. The silty clay till is sandy and contains trace gravel, occasional rock pieces and cobbles. The thickness of the silty clay till ranged from 7.7 m to 8.5 m.

In Borehole 09-055, a second silty clay till unit was contacted at 15.3 m depth (Elevation 136.0).

The depth to the base of the silty clay till was 10.7 m (Elevations 140.6 to 140.8).

Based on SPT 'N' values ranging from 7 to 15 blows per 0.3 m of penetration, the upper 1.5 m of silty clay till is described as firm to very stiff in consistency. Below the upper 1.5 m, the silty clay till is described as very stiff to hard in consistency, based on SPT 'N' values ranging from 18 to 71 blows for 0.3 m of penetration. An SPT 'N' value of 50 blows per 0.075 m of penetration was measured near elevation 142.3 in Borehole 09-052.

The natural moisture contents of the samples recovered from the silty clay till layer ranged from 8% to 22%.

Grain size distribution curves for the samples tested are presented on the Record of Borehole sheets and on Figures B1 and B2 of Appendix B. Atterberg Limit test results are presented on Figures B4 and B5 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	**(%)**
Gravel	0 to 5
Sand	24 to 39
Silt	42 to 54
Clay	12 to 30

Liquid Limit	20 to 37
Plastic Limit	13 to 18

THURBER

The above results show that the silty clay till is typically of low plasticity with group symbols of CL and CL-ML. One tested sample is of medium plasticity with a group symbol of CI.

Visual assessment and a high SPT 'N' value measured in the sample recovered at 15.3 m depth (Elevation 136.0) in Borehole 09-055, indicated the possible presence of bedrock. Therefore, coring was conducted below 15.3 m depth (Elevation 136.0). The cores recovered consisted of silty clay till with rock fragments and occasional cobbles and boulders. Coring was terminated at 19.8 m depth (Elevation 131.5).

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

5.4 Sand and Silt Till

Native grey sand and silt till containing trace to some clay, trace gravel and occasional cobbles and sand seams was contacted below the silty clay till at 10.7 m depth (Elevations 140.6 to 140.8) in all the boreholes. Boreholes 09-052 to 09-054 were terminated within the sand and silt till at depths ranging from 12.6 m to 15.4 m (Elevations 135.9 to 138.9). The thickness of the sand and silt till was 4.6 m in Borehole 09-055.

The depth to the base of the sand and silt till in Borehole 09-055 was 15.3 m (Elevation 136.0).

Based on SPT 'N' values of 50 to100 blows per 0.025 m to 0.25 m of penetration, the sand and silt till is described as very dense in relative density. An SPT 'N' value of 21 blows per 0.3 m of penetration, indicating compact relative density was measured near elevation 140.3 in Borehole 09-055.

The natural moisture contents of the samples recovered from the sand and silt till layer ranged from 5% to 15%.

Grain size distribution curves for the sand and silt till samples tested are presented on the Record of Borehole sheets and on Figure B3 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	4 to 13
Sand	37 to 49
Silt	31 to 49
Clay	5 to 10

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

5.5 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 5.1, along with the measurements in the boreholes upon completion of drilling.

Table 5.1 – Measured Groundwater Levels

Foundation Element	Borehole	Date (2009)	Water Level (m)		Comment
			Depth (m)	Elevation (m)	
North Abutment	09-052	May 21	3.5	148.0	In piezometer
	09-053	March 12	15.4	135.9	Open borehole
South Abutment	09-055	May 5 May 21	3.4 3.6	147.9 147.7	In piezometer

The groundwater levels measured in the piezometers are typically near elevation 148.0.

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 Eastern Ontario Diamond Drilling Ltd. of Hawkesbury, Ontario. The field work was supervised on a full time basis by Mr. Luke Gilarski 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.

Rocío Palomeque Reyna, P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
R. Palomeque Reyna
100083209
Dec/14/09
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
Dec 14/09
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Dec 14/09
PROVINCE OF ONTARIO

FOUNDATION INVESTIGATION AND DESIGN REPORT
CAWTHRA ROAD GRADE SEPARATION STRUCTURE
MISSISSAUGA BUS RAPID TRANSIT (BRT) PROJECT
MISSISSAUGA, ONTARIO

Geocres Number: 30M12-286

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 Cawthra Road grade separation structure.

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

Based on the preliminary General Arrangement (GA) drawing provided by McCormick Rankin Corporation, a single-span, concrete structure is planned. The structure span will be approximately 13.8 m (parallel to existing Cawthra Road alignment) and the width will be approximately 32.1 m.

The existing ground surface at Cawthra Road is near elevation 151.4. The proposed BRT alignment will be in a cut with the base near elevation 145.5. A cut of approximately 6.0 m depth will therefore be required to pass BRT under Cawthra Road.

Four retaining walls, one on each corner of the grade separation structure, are also included in the design. The walls will extend parallel to the existing Cawthra Road alignment.

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 pavement structure, sand and gravel fill and silty clay fill overlying native firm to hard silty clay till and very dense sand and silt till. Piezometers installed in Boreholes 09-052 and 09-055 revealed that groundwater level is near 3.5 m depth (Elevation 148.0) 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 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.

8.1 Spread Footings on Native Silty Clay Till

Spread footings can be founded on the native undisturbed very stiff to hard silty clay till. Provided a minimum footing width of 2 m is maintained footings founded on the above recommended strata at the founding elevation recommended below may be designed for the following values:

- Factored geotechnical resistance of 525 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

The highest permitted founding level for spread footings is elevation 146.5, generally 4.8 m to 5.0 m below existing ground surface. It should be noted that this elevation is above the base of the cut (elevation 145.5); the actual footing base levels will be lower than elevation 145.5, to provide adequate embedment depth below finished grade for frost protection.

The geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be substantially complete by the end of construction. Differential settlement is not expected to exceed 20 mm in a 6 m span.

The sliding resistance of mass concrete poured on the native till soil may be computed on the basis of an ultimate friction factor of 0.45. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Where subexcavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using engineered fill or concrete of the same class as the footing. The engineered fill must consist of OPSS Granular "A" placed in 150 mm lifts, compacted to 100% of its SPMDD at ±2% of optimum moisture content.

All footings should be provided with a minimum of 1.2 m of earth cover over the footing base (founding elevation) as protection against frost action.

8.2 Spread Footings on Engineered Fill

Consideration was also given to placing spread footings on an engineered fill pad if higher founding levels are required.

If an engineered fill pad is used at this site, all topsoil, soft/loose soils or other deleterious materials must be stripped from the footprint of the foundation to expose competent native subgrade material. The engineered fill should bear on stiff to hard silty clay till and the highest permitted founding/base elevations at which engineered fill should be placed, are given in Table 8.1.

Table 8.1 – Highest Permitted Elevations for Base of Engineered Fill

Foundation Element	Borehole	Engineered Fill Base	
		Depth below existing ground surface (m)	Founding Elevation
North Abutment	09-052	3.5	148.0
	09-053	3.3	148.0
South Abutment	09-054	3.3	148.0
	09-055	3.3	148.0

The engineered fill must consist of OPSS Granular "A" placed in 150 mm lifts and compacted to 100% of its SPMDD at ±2% of optimum moisture content and generally conforming to the geometry illustrated in Figure 1 in Appendix D.

Provided a minimum footing width of 2 m is maintained, footings bearing on an engineered fill pad at least 2.0 m thick may be designed for the following values:

- Factored geotechnical resistance of 900 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

These resistance values are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is expected to not exceed 25 mm. Differential settlements are not expected to exceed 20 mm across the width of the structure.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction factor of 0.6. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

8.3 Augered Caissons (Drilled Shafts)

Drilled shaft foundations founded on very dense sand and silt till may be considered for the support of structural loads at this site. The caissons will develop resistance through a combination of sidewall shear and end bearing in the till deposits.

To achieve the high resistances required to make caisson foundations practical, the caissons should be extended at least 4.0 m into the very dense (100 blow) sand and silt till encountered below the silty clay till.

The recommended caisson base elevations and vertical geotechnical resistance values for 1.2 m and 1.5 m diameter caissons founded at the levels, calculated based on contributions from both skin friction and end bearing, are presented in Table 8.2 and 8.3, respectively.

Table 8.2 – Highest Permitted Founding Elevations for Caissons

Foundation Element	Borehole	Caisson	
		Depth below existing ground surface (m)	Founding Elevation
North Abutment	09-052	13.5	138.0
	09-053	14.3	137.0
South Abutment	09-054	14.3	137.0
	09-055	16.3	135.0

Table 8.3 – Recommended Resistance Values for Caisson Design

Caisson Diameter (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Resistance at SLS (kN)
1.2	3,900	3,100
1.5	5,800	4,600

8.3.1 Caisson Installation

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

The caisson installation equipment should be able to dislodge and remove or penetrate any obstructions or cobbles, boulders or rock slabs/fragments in the till. The glacial till is hard or very dense at this site below elevations 147.0 to 144.0 and augering might be laboured.

Soil sloughing and groundwater seepage may be experienced where the caissons extend into cohesionless till below the groundwater level noted at Elevation 148.0. A temporary steel liner will be required in such conditions to support the sidewalls and provide seepage cut-off. It may also be necessary to extend the caisson and liner below the permeable layer to provide cut-off, or place the concrete by tremie if dewatering of the caisson is not practical. A head of at least 600 mm of concrete must be maintained in the liner at all times during liner withdrawal to minimize necking of the concrete by saturated soils.

The contract documents must contain a statement to alert bidders that caisson installation may extend through potential obstruction such as cobbles and boulders, and that dewatering of the caissons may be required prior to placing concrete. The suggested wording for an NSSP addressing these issues is included in Appendix E.

The resistance values provided above are based on the assumption that the walls and base of each caisson are cleaned of loose material prior to placement of concrete. The caisson excavation should be dewatered (if necessary) to allow cleaning of the base and walls and prior to pouring concrete. Concrete should be poured with a minimum delay after the caisson is drilled and cleaned. A delay of 24 hours is considered to be the maximum permissible and the caisson must be maintained in a dewatered condition throughout any delay before concrete placement.

8.4 Driven Piles

The subsurface conditions at the site are considered suitable for foundations consisting of steel H-piles driven to refusal in the very dense sand and silt till.

The elevations at which the piles are expected to encounter refusal are given in Table 8.4.

Table 8.4 – Estimated Pile Tip Elevation

Foundation Element	Borehole	Depth below existing ground surface (m)	Founding Elevation
North Abutment	09-052	12.5	139.0
	09-053	12.8	138.5
South Abutment	09-054	12.8	138.5
	09-055	14.3	137.0

The pile tip elevations shown in Table 8.4 should be used for estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.4.4 Pile Driving.

8.4.1 Axial Resistance

The factored, axial, geotechnical resistances for a typical pile section when driven into very dense till are presented in Table 8.5.

Table 8.5 – Axial Resistance of a Pile Section

Pile Section	Piles Driven Into Very Dense Sand and Silt Till	
	Factored ULS (kN)	SLS (kN)
HP 310 X 110	1,600	1,400

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

8.4.2 Pile Tips

Due to the presence of cobbles and boulders in the till soils, the tips of all piles should be fitted with driving shoes as per OPSD 3000.100.

8.4.3 Pile Installation

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

The Contract Documents should contain a NSSP alerting the Bidders to:

- The presence of cobbles and boulders in the expected bearing stratum.
- The possibility of piles within a group achieving the specified resistance at different elevations.
- The possibility of some piles meeting refusal on a large boulder.

Suggested texts for NSSP's are included in Appendix E.

8.4.4 Pile Driving

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance should be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile". "R" must have a value of two times the design load at ULS calculated by the structural engineer.

The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving and contact the design team for further instructions.

To facilitate pile installation, embankment fill through which piles may be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

8.4.5 Downdrag

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

8.4.6 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design. The use of H-piles at the abutments allows for the design of an integral abutment structure.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. The near surface, native soils at this site are firm to hard and the lateral resistance of a pile in this soil may not provide sufficient flexibility. In addition, the upper 3 m of the pile may lie partially within the compacted fill of the approach embankment. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is driven, the space between the pile and the CSP should be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 8.6.

Table 8.6 – Integral Abutment Sand Backfill Grading

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

8.4.7 Lateral Resistance

The upper soils encountered at this site may be treated as essentially cohesive. Accordingly, the lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

k_s = $67*S_u/D$ (kN/m^3)

p_{ult} = $9*S_u$ (kPa) at and below a depth of 3*D (m) reduced to zero at the ground surface

where D = pile width in metres

S_u = undrained shear strength (kPa)
= 175 kPa

For the cohesionless till below elevation 140.7, the lateral resistance of the pile may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

k_s = $n_h \cdot z / D$ (kN/m^3)

p_{ult} = $3 \cdot \gamma \cdot z \cdot K_p$ (kPa)

where z = depth of embedment of pile in metres

D = pile width in metres

n_h	=	coefficient of horizontal subgrade reaction
	=	8,000 kN/m^3
γ	=	unit weight (bouyant unit weight below water table)
	=	11.0 kN/m^3
K_p	=	passive earth pressure coefficient
	=	3.5

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis 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 pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult}*L*D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in one pile be limited to no more than 200 kN at ULS and 110 kN at SLS.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing.

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

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

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

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

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

Intermediate values may be obtained by interpolation.

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

8.5 Frost Protection

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

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.

8.6 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, spread footings founded on native undisturbed very stiff to hard silty clay till is considered the most cost effective foundation option for supporting the grade separation structure at this site.

9 PERMANENT CUT

Permanent earth cut is required to construct the BRT alignment at this site. In the lands adjacent to Cawthra Road, the cut will generally extend to depths of about 4.0 m and be carried out within native firm to hard silty clay till. At Cawthra Road, the cut will also extend through the existing pavement materials and silty clay embankment fill overlying the native clay till, increasing the total cut depth to about 6.0 m at the structure.

Based on the stratigraphy encountered in the boreholes, permanent cuts through the cohesive and cohesionless fills and native soils are expected to be stable at inclinations not steeper than 2H : 1V. The results of limit equilibrium analyses carried out for the proposed cut depth indicate that minimum factors of safety considered appropriate to achieve stability (F.S. $\geq$ 1.3 for undrained condition, during construction; and F.S. $\geq$ 1.5 for drained condition, post-construction) are available for both short-term and long-term conditions in the cohesive foundation soils.

Based on water levels measured in the piezometers, the groundwater level is about 2.5 m above the base of the cut. The cohesive fills and native silty clay till soils that will be exposed within the cut are considered to be of generally low permeability and consequently seepage from the sides 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 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, silty clay fill and native silty clay till. The soils, especially the tills, may contain cobbles, boulders and slabs of shale and hard limestone rock.

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 for fill and Type 2 for native stiff to hard silty clay till.

Flatter 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 must be based on his equipment, experience and interpretation of the site conditions. It is anticipated that a hydraulic excavator will be suitable. Provision must be made for the handling of pavement materials, potential obstructions in the fill, and possible cobbles, boulders and rock slabs. Arduous excavation may be experienced in the hard and very dense till materials.

A NSSP should be included in the contract alerting the Contractor to the possible presence of cobbles and boulders in the till deposits. Suggested wording 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 at Cawthra Road 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 socketted into the very stiff to hard silty clay till or the underlying silty and sand till 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 footing construction founded on very stiff to hard silty clay till are anticipated to extend up to 3.7 m below the groundwater level. Considering the consistency and low permeability of the clayey soils, 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. All footings must be constructed in the dry.

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.

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 $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2$ kN/m^3	
	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. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B, Type I) might be preferred as it results in lower forces acting on the ballast wall as the soil moves towards the soil mass.

The factors in Table 15.1 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 RETAINED SOIL SYSTEMS

It is understood that Retained Soil System (RSS) walls are proposed at each corner of the new grade separation structure. The proposed north and south RSS walls are approximately 13.8 m and 12.9 m, in length respectively with an approximate maximum height of 6.0 m.

RSS walls used in conjunction with the new grade separation structure 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 soil conditions at the wall locations comprise existing clay fill overlying firm to hard silty clay till. The soil conditions encountered on site are generally suitable for the support of RSS walls.

To provide an acceptable foundation performance, the RSS mass must be founded on the native undisturbed very stiff to hard silty clay till. The highest base levels for the underside of the wall are indicated in Table 16.1.

Table 16.1 – Maximum Elevation at Underside of Wall Base or Granular A Fill

RSS location	Borehole	Depth to Native Till (m)	Native Till Elevation (m)
Northwest	09-052	4.0	147.5
Northeast	09-053	3.0	148.3
Southwest	09-054	3.1	148.2
Southeast	09-055	3.8	147.5

A wall founded on native very stiff to hard silty clay till at or below elevations shown in Table 16.1 should be designed for a factored bearing resistance of 300 kPa at ULS and a bearing resistance of 200 kPa at SLS.

Alternatively, the RSS may be founded on engineered fill founded on the native very stiff silty clay till contacted at the above elevations. Engineered fill placed under the RSS mass to achieve the design founding level should consists of OPSS Granular "A" compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.

The geotechnical resistances provided above are 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 engineered granular fill may be estimated using an ultimate friction coefficient of 0.55. For an RSS block founded on very stiff native till, an ultimate coefficient of sliding friction of 0.45 may be used.

Topsoil, fill, soft/loose soil, and any soft/wet native material should be stripped from the footprint of the RSS. The native soil under the RSS foundation should be proofrolled to detect and replace any soft areas.

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 foundation soils, the geometry of the embankment and location of the RSS within the embankment. Typically, global stability should not be a concern for a RSS wall founded on the very stiff to hard till at this site.

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 (K_{AE}) and passive (K_{PE}) 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 may be used:

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 $\phi = 35°$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\gamma = 21.2$ kN/m^3	
	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
Passive (K_{PE})	3.6	-	3.1	-
At Rest (K_{OE})**	0.54	-	0.58	-

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

18 BURIED UTILITIES

The BRT alignment follows transportation and utility corridors containing roadways, 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, 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 utilities should also be obtained by the designer from the owner/designer of these 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. Pile refusal at higher elevation.

   Glacial till deposits inherently contain boulders. It is possible that a pile will achieve refusal at a higher elevation than anticipated due to encountering a boulder. If it is suspected that this is happening, the QVE must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

2. Groundwater control

   Proper groundwater and surface water control measures must be in place prior to commencing excavation. All footing concrete must be placed in the dry.

3. Excavations

   Care must be exercised during excavation to avoid disturbing the founding subgrade. The exposed subgrade soils should be expeditiously inspected, approved and protected from disturbance. Cobbles and boulders should be expected within the silty clay till layer.

4. Existing underground utilities and roadways

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

5. Caisson installation

   The cohesionless till soils would be susceptible to disturbance under conditions of unbalanced hydrostatic head. Temporary liners should be installed to support the caisson sidewalls and provide seepage cut-off where required.

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.

LICENSED PROFESSIONAL ENGINEER
R. Palomeque Reyna
100083209
Dec/14/09
PROVINCE OF ONTARIO

Rocío Palomeque Reyna, P.Eng.
Geotechnical Engineer

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

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
Dec 14/09
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Dec 14/09
PROVINCE OF ONTARIO

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

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
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.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength (MPa)**	**(psi)**	**Field Estimation of Hardness***
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 09-052

METRIC

G.W.P. 19-1351-160 LOCATION N 4 830 140.1 E 610 596.5 ORIGINATED BY LG

HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM DATE 2009.03.13 - 2009.03.13 CHECKED BY LT

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
151.5						
0.0	**ASPHALT** (100mm)					
0.1	**SAND** and **GRAVEL**, some silt Brown (FILL)	1	AS		151	
150.7 0.8	Silty **CLAY**, some sand, trace gravel Very Stiff Brown Moist (FILL)	1	SS	15	150	
		2	SS	17		
149.3 2.2	Silty **CLAY**, sandy, trace gravel, occasional cobbles Stiff to Firm Brown (TILL)	3	SS	8	149	1 26 47 26
		4	SS	7	148	
	Becoming hard	5	SS	39	147	3 24 54 19
		6	SS	28	146, 145	
		7	SS	37	144	5 26 45 24
		8	SS	50/ .075	143, 142	

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 40 80 120 160 200
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m3

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-052

METRIC

G.W.P.	19-1351-160	LOCATION	N 4 830 140.1 E 610 596.5	ORIGINATED BY	LG
HWY	403 / BRT	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM		DATE	2009.03.13 - 2009.03.13	CHECKED BY	LT

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Continued From Previous Page										
	Silty **CLAY**, sandy, trace gravel, occasional cobbles Stiff to Firm Brown (TILL)				141						
140.8 10.7	**SAND** and **SILT**, some clay, trace gravel, occasional cobbles Very Dense Grey Moist (TILL)	9	SS	50/ .100	140	o		4	37	49	10
	Wet	10	SS	50/ .075	139	o					
138.9 12.6	END OF BOREHOLE AT 12.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2009.05.21 3.5 148.0										

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 40 80 120 160 200; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 09-053

 METRIC

G.W.P. 19-1351-160	LOCATION N 4 830 164.4 E 610 612.2		ORIGINATED BY LG
HWY 403 / BRT	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY AN
DATUM	DATE 2009.03.12 - 2009.03.12		CHECKED BY LT

Legend headers: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40, 80, 120, 160, 200); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20, 40, 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
151.3									
0.0	**ASPHALT** (50mm)	1	AS						
	SAND and **GRAVEL** Brown Moist (FILL)				151				
150.6 / 0.8	Silty **CLAY**, some sand, trace gravel Stiff to Firm Mottled Brown Grey (FILL)	1	SS	10	150				
		2	SS	14					
		3	SS	6	149				
148.4 / 3.0	Silty **CLAY**, sandy, trace gravel Very Stiff to Hard Brown (TILL)	4	SS	15	148	0	25	45	30
					147				
		5	SS	35	146				
		6	SS	42	145				
					144				
		7	SS	28	143	0	25	46	29
	Occasional rock pieces	8	SS	53	142				

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

15 (20, 5, 10) (%) STRAIN AT FAILURE

ONTMT4S 1160.GPJ 5/26/09

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 09-053

METRIC

G.W.P. 19-1351-160 LOCATION N 4 830 164.4 E 610 612.2 ORIGINATED BY LG

HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM DATE 2009.03.12 - 2009.03.12 CHECKED BY LT

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, sandy, trace gravel Very Stiff to Hard Grey (TILL)(CI to CL)					141		
140.7 10.7	**SAND** and **SILT**, trace gravel, trace clay Very Dense Grey Moist (TILL)	9	SS	50/ .125		140	o	
		10	SS	50/ .075		139	o	
	Wet Occasional coarse sand seams	11	SS	50/ .075		138 / 137	o	9 38 48 5
135.9 15.4	END OF BOREHOLE AT 15.4m. BOREHOLE OPEN AND WATER LEVEL AT 15.4m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.	12	SS	50/ .025	▽	136	o	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 40 80 120 160 200

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity

15 -○- 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 09-054

METRIC

G.W.P.	19-1351-160	LOCATION	N 4 830 125.6 E 610 610.8	ORIGINATED BY	LG
HWY	403 / BRT	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM		DATE	2009.03.12 - 2009.03.12	CHECKED BY	LT

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
151.3	
0.0	**ASPHALT** (100mm)
0.1	**SAND** and **GRAVEL** Brown Moist (FILL)
150.7 / 0.6	Silty **CLAY**, some sand, occasional rootlets Stiff Mottled Brown (FILL)
149.1 / 2.2	Silty **CLAY**, sandy, trace gravel Stiff to Very Stiff Mottled Brown (TILL)(CI to CL) Becoming grey Becoming hard

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	AS					
1	SS	12				
2	SS	13				
3	SS	8				
4	SS	15	0	39	45	16
5	SS	27				
6	SS	18				
7	SS	52	5	37	42	16
8	SS	71				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (151, 150, 149, 148, 147, 146, 145, 144, 143, 142); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (40, 80, 120, 160, 200); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³.

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 09-054

METRIC

G.W.P. 19-1351-160 LOCATION N 4 830 125.6 E 610 610.8 ORIGINATED BY LG

HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM DATE 2009.03.12 - 2009.03.12 CHECKED BY LT

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page						
140.7	Silty **CLAY**, sandy, trace gravel Hard Grey (TILL)(CI to CL)				141		
10.7	**SAND** and **SILT**, some gravel, trace clay Very Dense Grey Moist (TILL)	9	SS	100/ .250	140	o	
		10	SS	50/ .125	139	o	13 48 31 8
137.4		11	SS	50/ .075	138	o	
13.9	END OF BOREHOLE AT 13.9m. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE TO 0.1m THEN ASPHALT TO SURFACE.						

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (40 80 120 160 200) — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m3; GROUND WATER CONDITIONS; STRAT PLOT.

+3, X3: Numbers refer to Sensitivity   20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 09-055

METRIC

G.W.P. 19-1351-160 LOCATION N 4 830 149.3 E 610 626.1 ORIGINATED BY LG

HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN

DATUM DATE 2009.03.11 - 2009.03.11 CHECKED BY LT

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
151.3						
0.0	**SAND** and **GRAVEL** Brown Wet	1	AS		151	
150.7	(FILL)					
0.6	Silty **CLAY**, some sand, trace gravel Firm to Stiff Mottled Brown (FILL)	1	SS	8	150	
		2	SS	10		
	Some gravel	3	SS	14	149	
148.3						
3.0	Silty **CLAY**, sandy, trace gravel Stiff to Hard Mottled Grey Brown (TILL)(CL)	4	SS	11	148	
					147	
		5	SS	38	146	2 25 53 20
		6	SS	35	145	
					144	
		7	SS	34	143	
	(CL-ML)	8	SS	46	142	3 39 46 12

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 40 80 120 160 200

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 1160.GPJ 5/26/09

RECORD OF BOREHOLE No 09-055

METRIC

G.W.P.	19-1351-160	LOCATION	N 4 830 149.3 E 610 626.1	ORIGINATED BY	LG
HWY	403 / BRT	BOREHOLE TYPE	Hollow Stem Augers / NQ Coring	COMPILED BY	AN
DATUM		DATE	2009.03.11 - 2009.03.11	CHECKED BY	LT

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40, 80, 120, 160, 200); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, sandy, trace gravel Hard Grey (TILL)				
140.6 / 10.7	**SAND** and **SILT**, trace clay, trace gravel Compact to Very Dense Grey Wet (TILL)	9	SS	21	
		10	SS	100/ .125	8 49 34 9
		11	SS	50/ .025	
136.0 / 15.3	Silty **CLAY**, some sand, trace gravel, occasional cobbles and boulders, with fragments and slab of shale and limestone Grey (TILL)	~~12~~	~~SS~~	~~50/~~ .025	
					Refusal to auger on shale slab. Set up to core.
		1	RUN		
		2	RUN		
	Occasional rock fragments	3	RUN		
131.5 / 19.8	END OF BOREHOLE AT 19.8m.				

Elevation scale: 141, 140, 139, 138, 137, 136, 135, 134, 133, 132

Continued Next Page

+ ³, × ³ : Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 09-055

METRIC

G.W.P. 19-1351-160 LOCATION N 4 830 149.3 E 610 626.1 ORIGINATED BY LG

HWY 403 / BRT BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN

DATUM DATE 2009.03.11 - 2009.03.11 CHECKED BY LT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40 80 120 160 200	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
	Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2009.05.09 3.4 147.9 2009.05.21 3.6 147.7										

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Appendix B

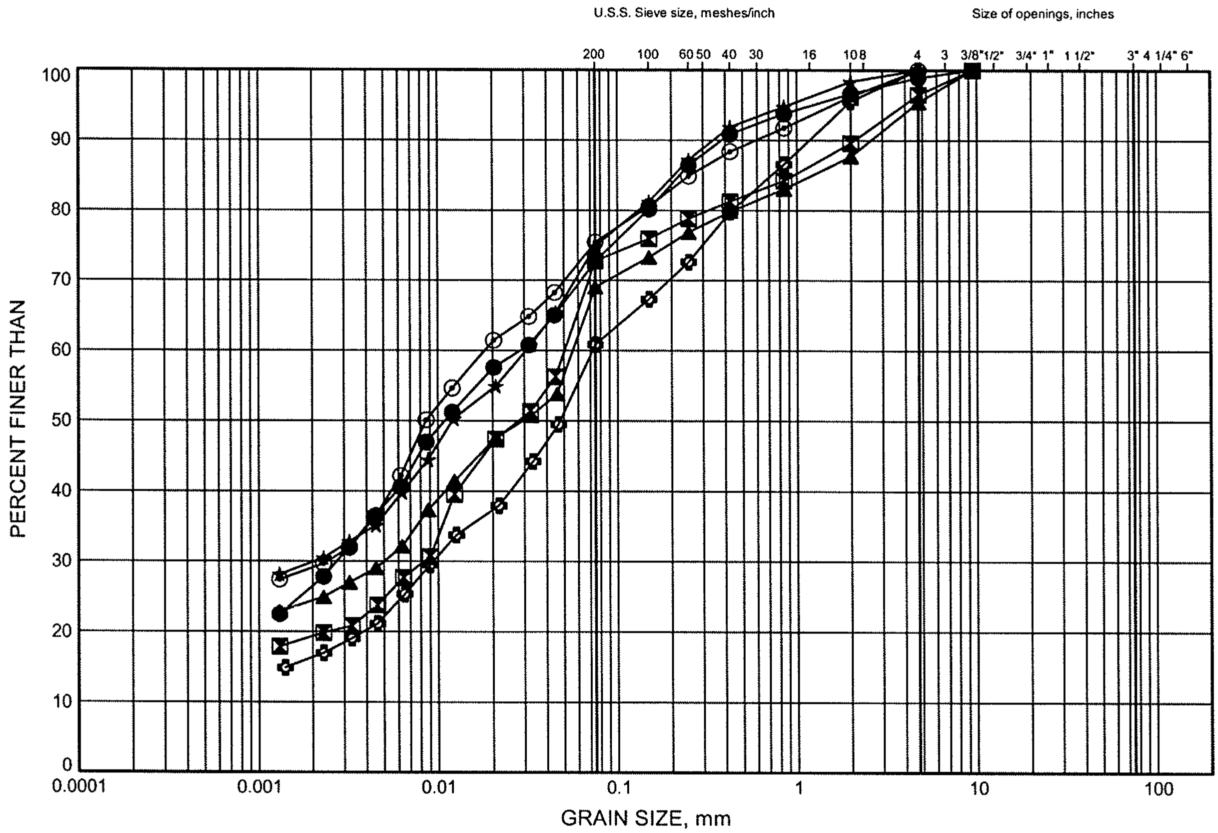
Laboratory Test Results

Mississauga BRT East

GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-052	2.59	148.89
⊠	09-052	4.88	146.61
▲	09-052	7.92	143.56
★	09-053	3.35	147.99
⊙	09-053	7.92	143.41
✪	09-054	3.35	147.99

W.P.# 19-1351-160

Prepared By AN

Checked By RPR

THURBER

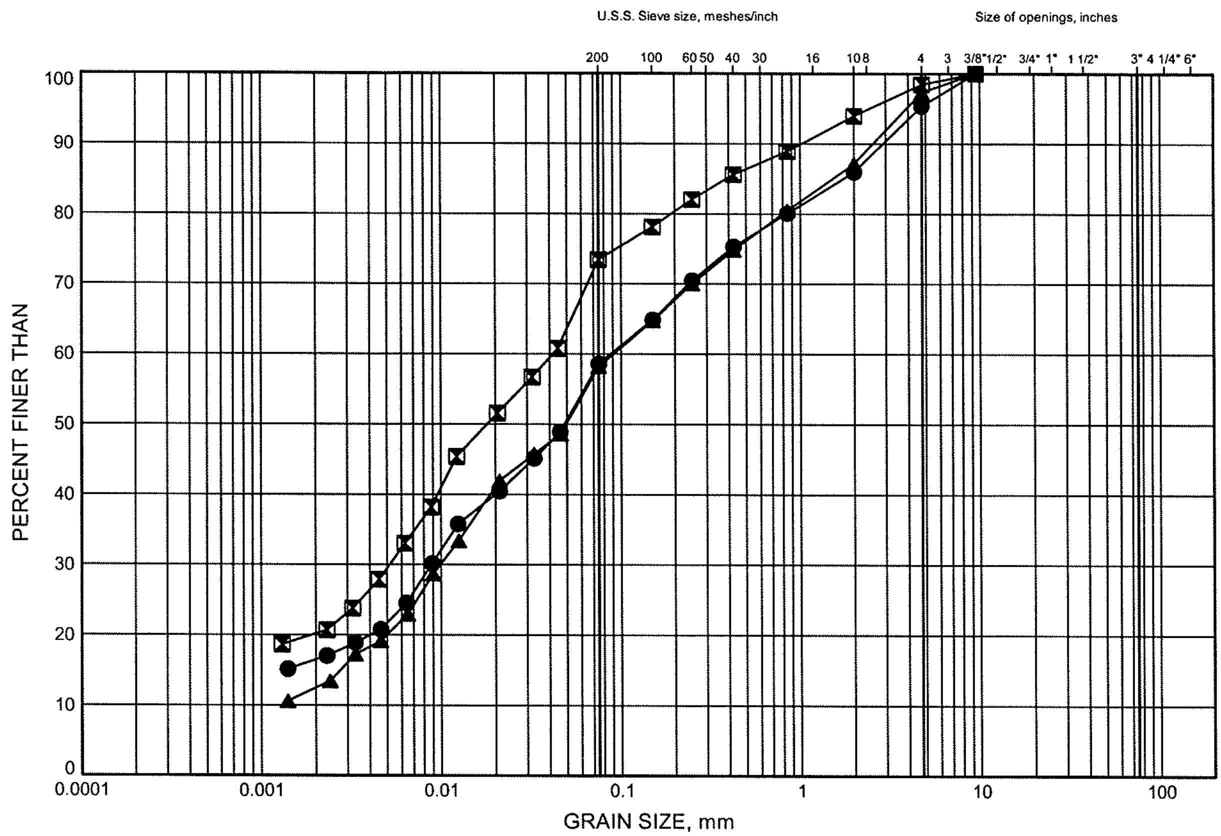
GRAIN SIZE DISTRIBUTION - THURBER 1160.GPJ 5/21/09

Mississauga BRT East

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-054	7.92	143.42
⊠	09-055	4.88	146.41
▲	09-055	9.45	141.84

W.P.# 19-1351-160

Prepared By AN

Checked By RPR

THURBER

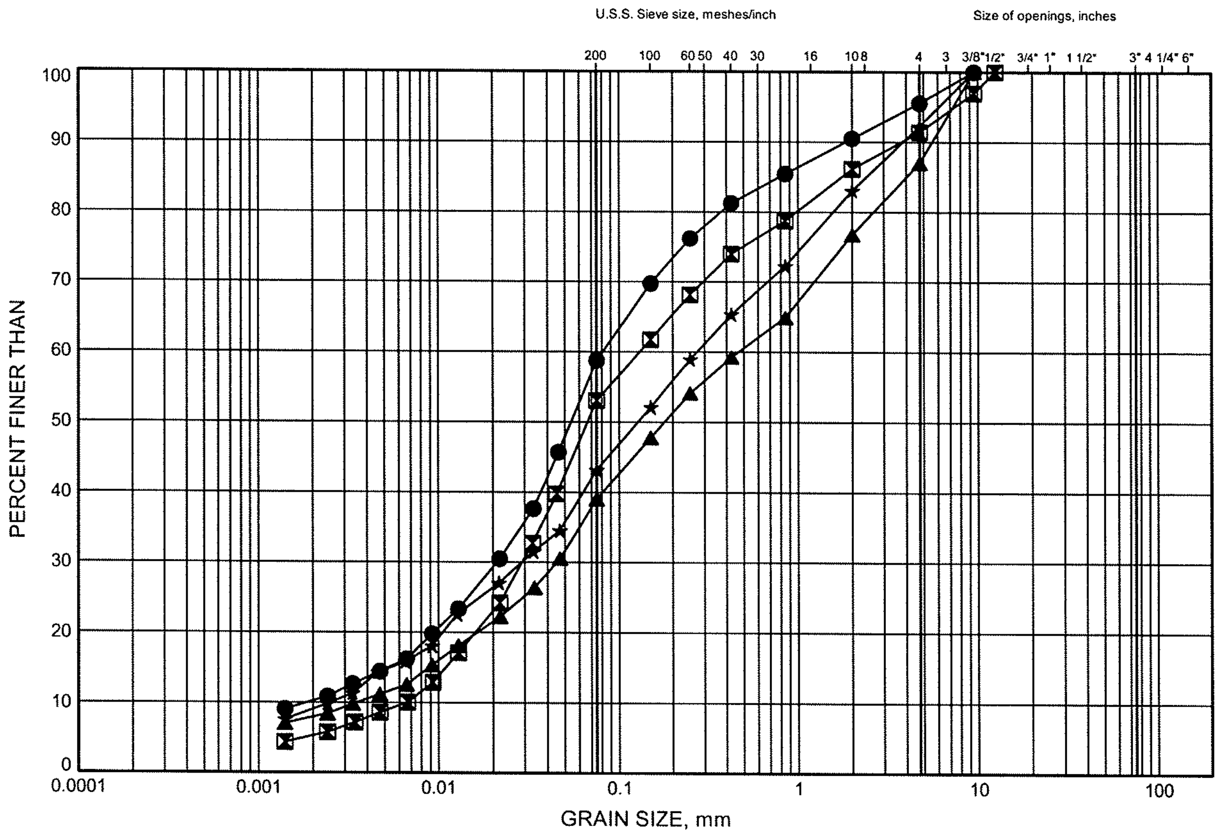
GRAIN SIZE DISTRIBUTION - THURBER 1160.GPJ 5/21/09

Mississauga BRT East

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND & SILT TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-052	10.80	140.69
⊠	09-053	13.83	137.51
▲	09-054	12.26	139.09
★	09-055	12.33	138.96

W.P.# 19-1351-160

Prepared By AN

Checked By RPR

THURBER

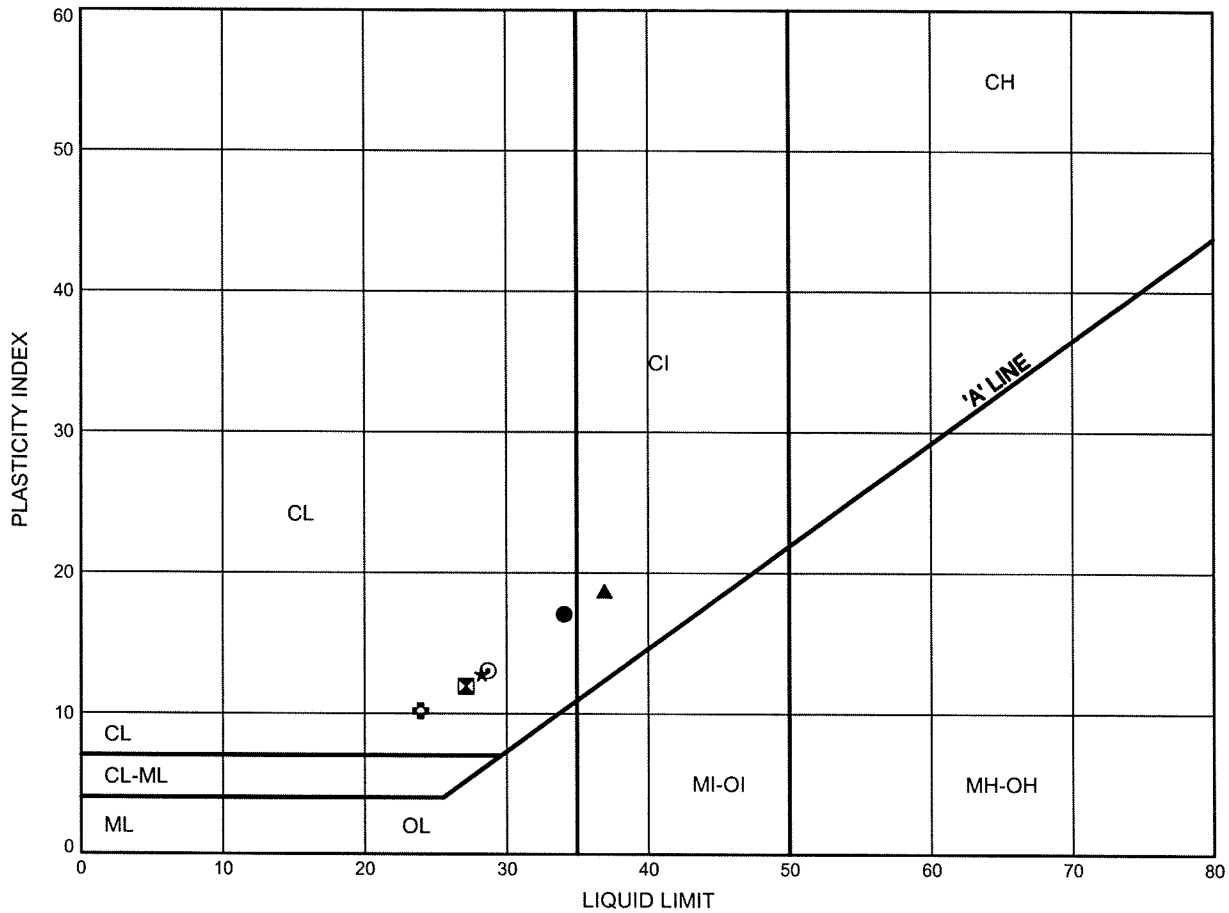
GRAIN SIZE DISTRIBUTION - THURBER 1160.GPJ 5/21/09

Mississauga BRT East

ATTERBERG LIMITS TEST RESULTS

FIGURE B4

SILTY CLAY TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	09-052	2.59	148.89
⊠	09-052	7.92	143.56
▲	09-053	3.35	147.99
★	09-053	7.92	143.41
⊙	09-054	3.35	147.99
✪	09-054	7.92	143.42

Date May 2009

Project 19-1351-160

THURBER

Prep'd AN

Chkd. RPR

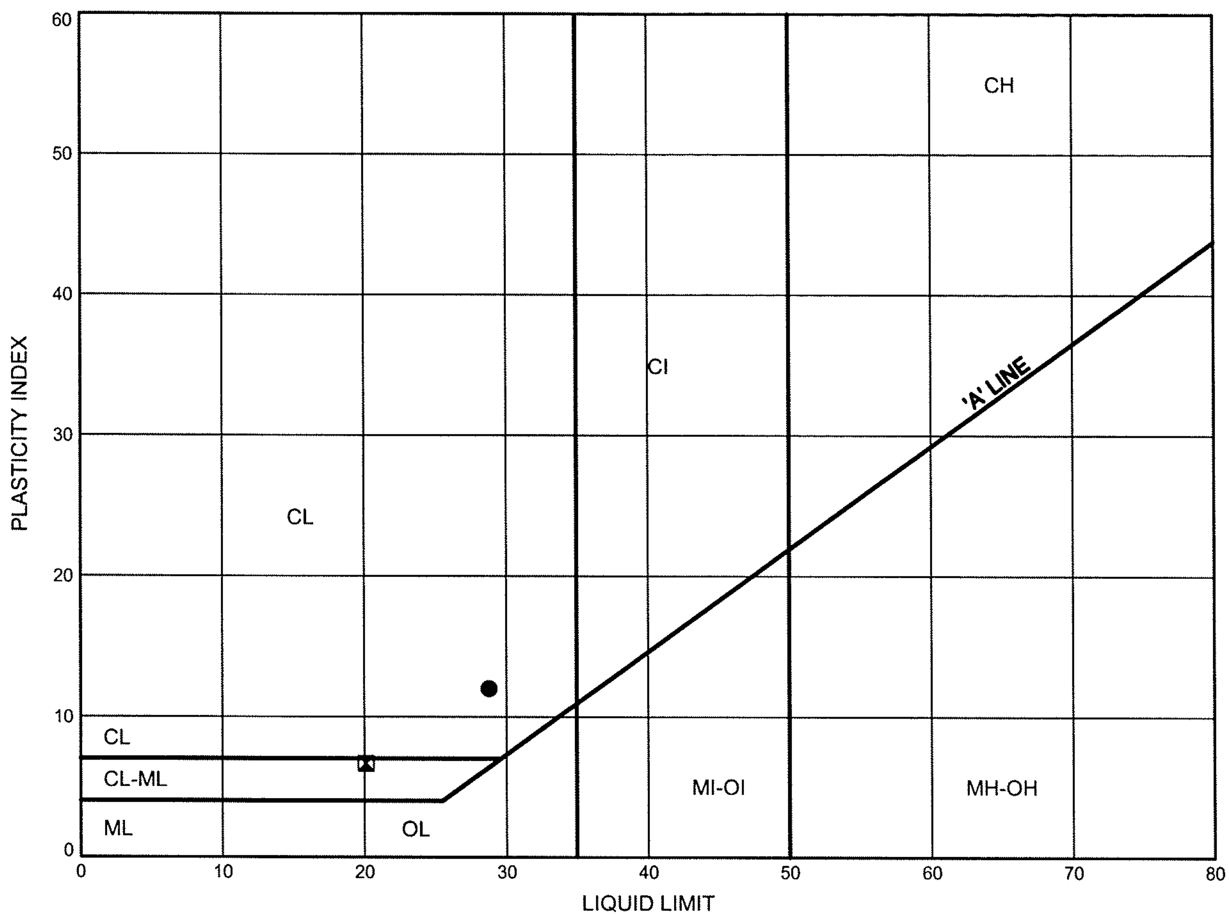
THURBALT 1160.GPJ 5/21/09

Mississauga BRT East

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

SILTY CLAY TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	09-055	4.88	146.41
⊠	09-055	9.45	141.84

Date May 2009

Project 19-1351-160

THURBER

Prep'd AN

Chkd. RPR

THURBALT 1160.GPJ 5/21/09

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

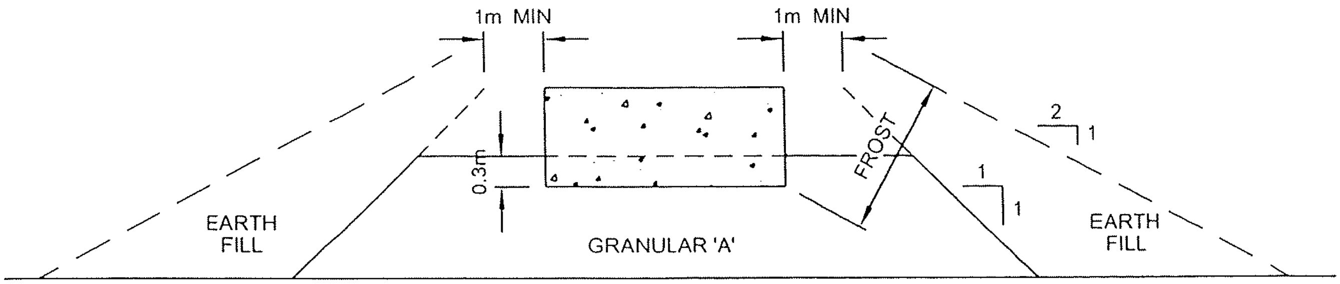
Footings on Native Soil	Footings on Engineered Fill	Driven Piles	Caissons
Advantages: i. Ease of construction. ii. Good geotechnical resistance is available on the till deposits. iii. Lower cost than deep foundations.	***Advantages:*** i. Generally less costly construction than deep foundation elements.	***Advantages:*** i. Piles will develop high geotechnical resistance in dense soils. ii. Installation of piles could continue in freezing weather. iii. Readily installed. iv. Foundation construction requires less volume of excavation than footings	***Advantages:*** i. High resistance is available for caissons founded in dense till. ii. Construction of caissons could continue in freezing weather. iii. Subexcavation of fill and variable material not required.
Disadvantages: i. Unwatering may be required, depending on depth of excavation	***Disadvantages:*** i. Dewatering may be required, depending on depth of excavation for fill placement. ii. Cost of engineered fill placement.	***Disadvantages:*** i. Higher unit costs than footings. ii. Possibility that cobbles and boulders may be encountered in till.	***Disadvantages:*** i. Higher cost than spread footings ii. Possibility of boulders or rock slabs being encountered during augering. iii. More likely to encounter groundwater. Temporary lines may be required to install caissons in cohesionless till under the water table. iv. Potential difficulty in cleaning and inspecting bases.
RECOMMENDED	**FEASIBLE**	**FEASIBLE**	**FEASIBLE**

THURBER

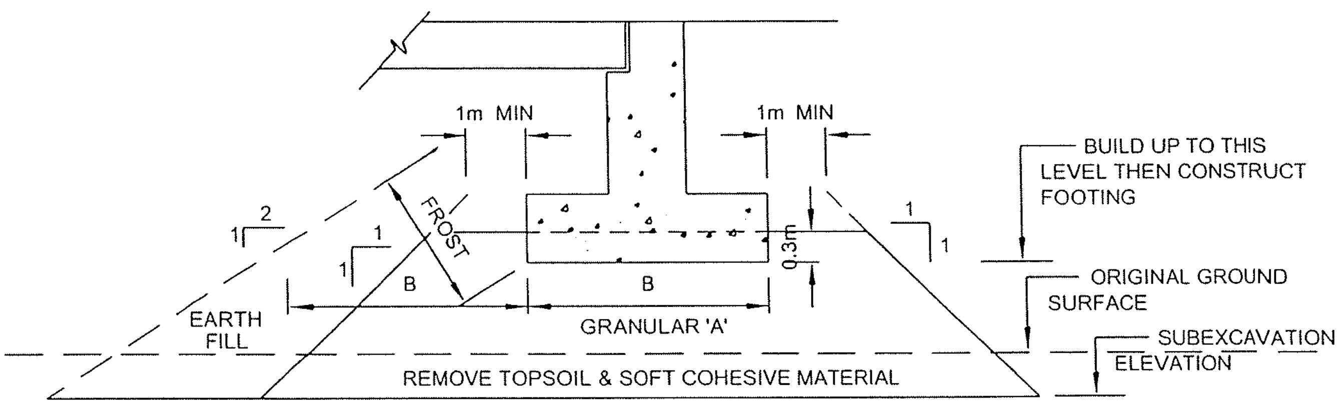
Appendix D

Figure

March , 2004



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND SOFT SILTY CLAY SUBSOIL UNDER FOOTPRINT OF COMPACTED GRANULAR 'A'.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ENGINEER AEG		
DRAWN SS		
DATE April , 2004	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE	
APPROVED PKC		
SCALE NTS		DWG. NO. FIGURE 1

TED35146 .DWG

Appendix E

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
- SP 903S01
- OPSS 902
- OPSS 1010
- OPSD 208.010
- OPSD 3000.100
- OPSD 3101.150
- OPSD 3102.100

2. Suggested Text for NSSP on "Excavation of Till Soil"

Cobbles, boulders and slabs of shale or hard limestone rock should be expected within the silty clay till and sand and silt till soils on site. Accordingly, equipment suitable for handling and removal of cobbles, boulders or rock slabs should be provided for excavation of the till.

Arduous excavation may be experienced in the hard and very dense till materials.

3. Suggested text for a NSSP on Pile Installation

The till soils may contain cobbles, boulders or rock slabs. The presence of cobbles and boulders will potentially have an impact on the installation of piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- The need to provide protection to the pile tips in the form of driving shoes.
- The cobbles and boulders may impede the driving of the piles.
- Some piles may meet refusal on boulders that are large enough not to be dislodged or broken by the pile driving.
- As a result of the presence of boulders, piles may meet refusal at varying depths.
- Pile driving must be controlled according to the criteria specified for the site.
- If a pile meets refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving.

4. Suggested Text for NSSP on "Caisson Construction"

Caisson installation through the till may encounter cobbles, boulders or hard rock slabs and the installation equipment should be capable of penetrating such obstructions.

Water seepage and/or soil sloughing into the caisson hole will occur from existing fill or the sand till at some locations. The cohesionless soils would be susceptible to disturbance under conditions of unbalanced hydrostatic head. Temporary liners shall be available on site, or be made available on very short notice, to support the caisson sidewalls and provide seepage cut-off where required.

Appendix F

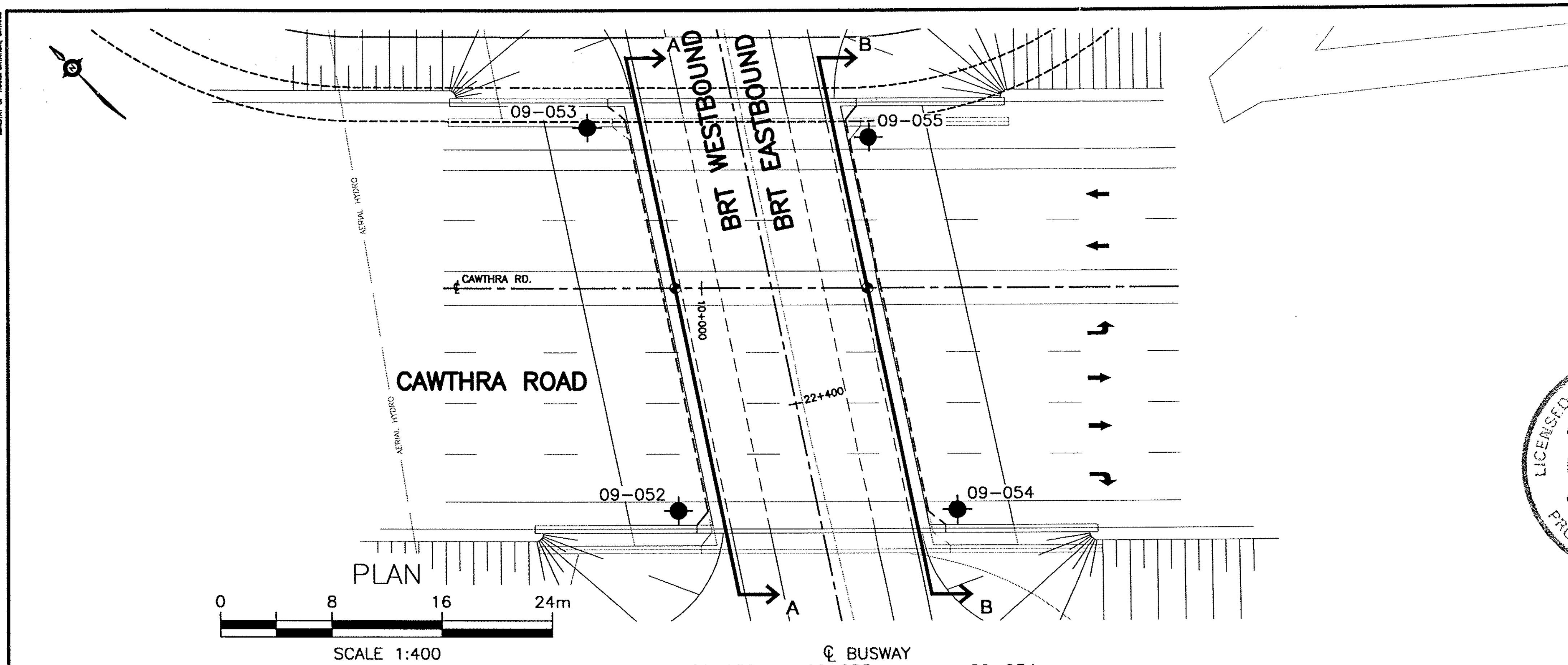
Site Photographs

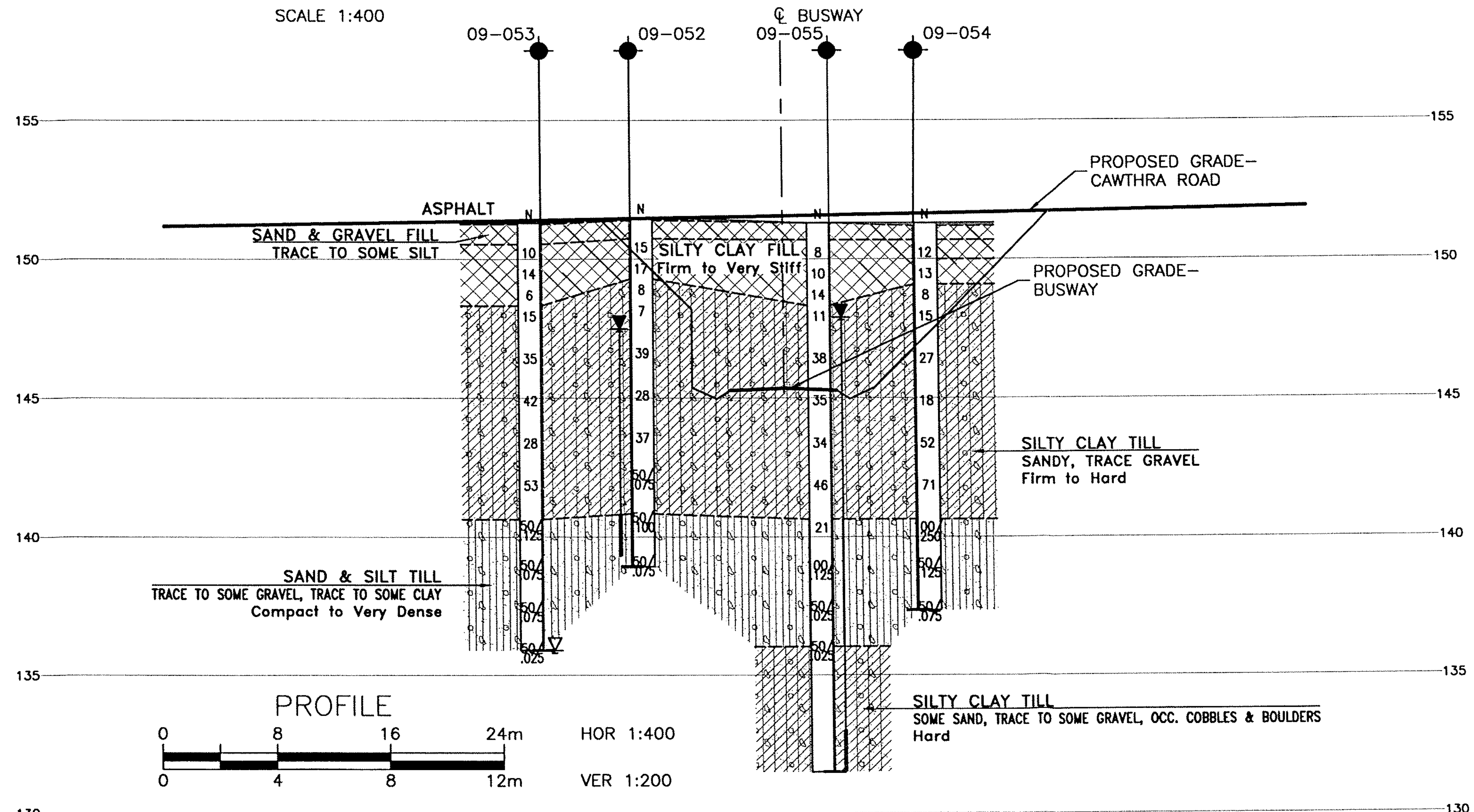
Photo 1. Cawthra Road just north of Eastgate Parkway, facing north

Photo 2. Cawthra Road and Eastgate Parkway intersection, facing south

Appendix G

Borehole Locations and Soil Strata Drawing





METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No

GWP No

SHEET

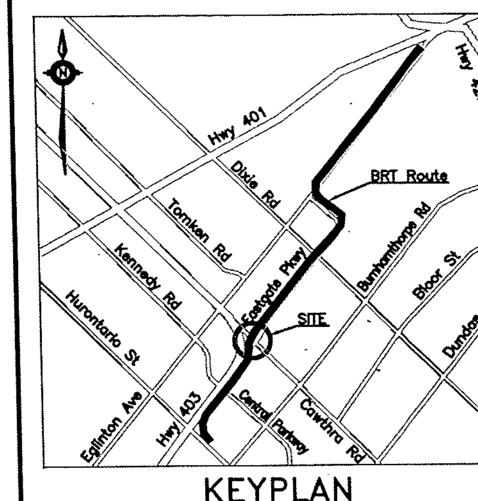
MISSISSAUGA BRT EAST
DETAILED DESIGN
GRADE SEPARATION STRUCTURE AT CAWTHRA RD
BOREHOLE LOCATIONS AND SOIL STRATA

McCORMICK RANKIN CORPORATION

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
Dec 14/09
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Dec 14/09
PROVINCE OF ONTARIO



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
09-052	151.5	4 830 140.1	610 596.5
09-053	151.3	4 830 164.4	610 612.2
09-054	151.3	4 830 125.6	610 610.8
09-055	151.3	4 830 149.3	610 626.1

-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.

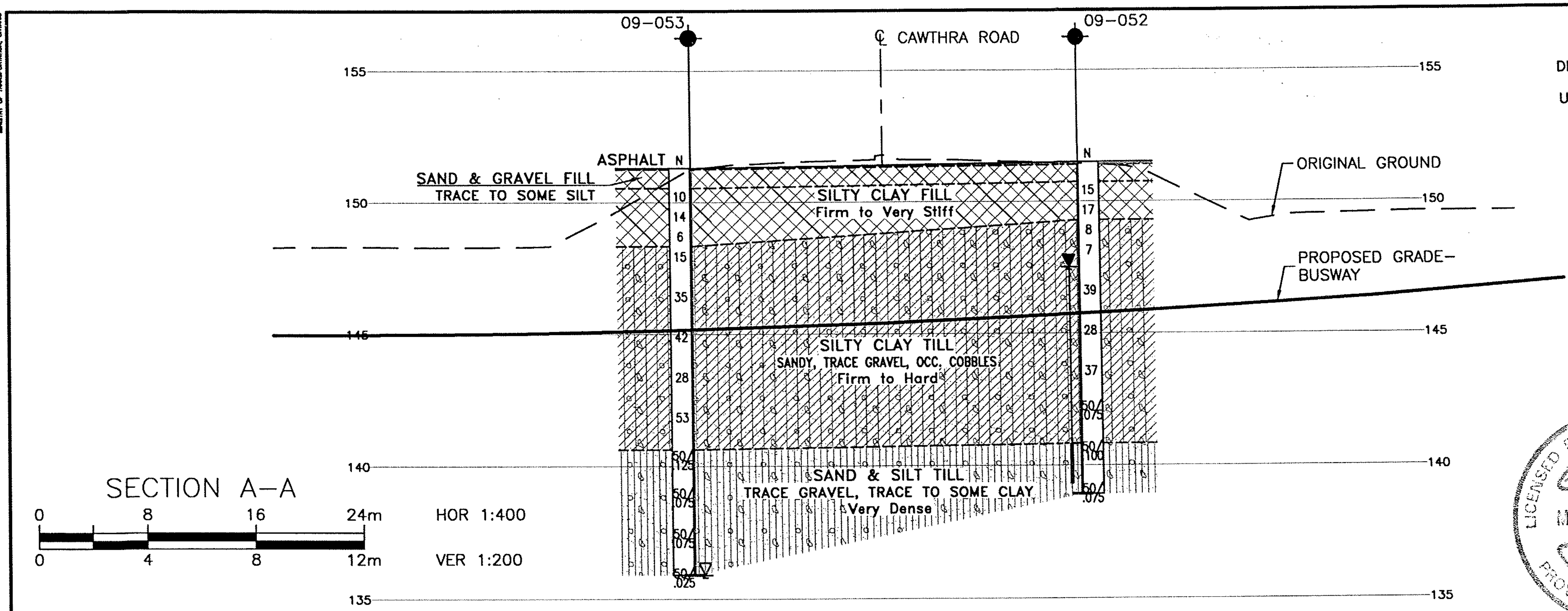
GEOCRES No. 30M12-286

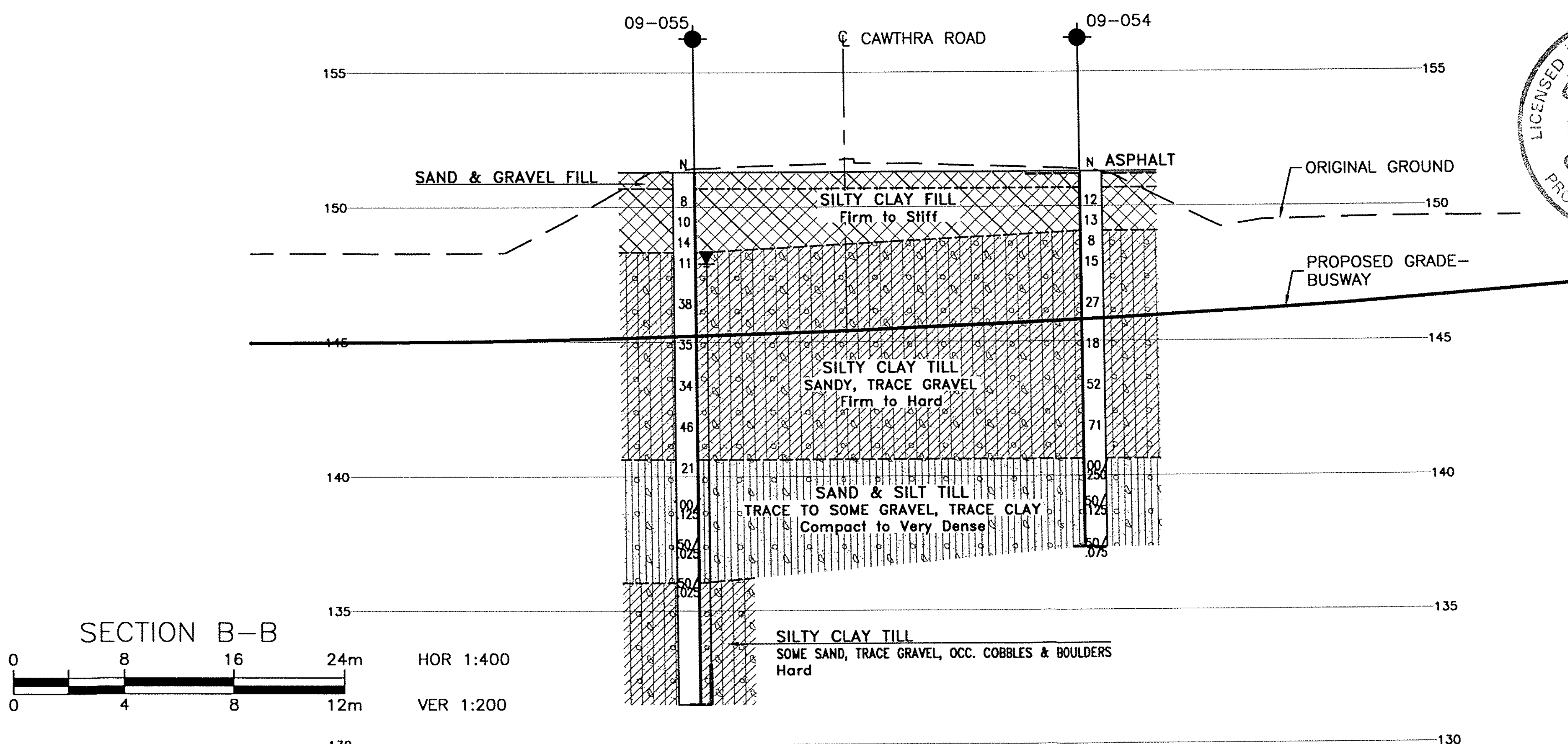
REVISIONS		
DATE	BY	DESCRIPTION

DESIGN MEF	CHK PKC	CODE	LOAD	DATE DEC. 2009
DRAWN MFA	CHK MEF	SITE	STRUCT	DWG 1

MINISTRY OF TRANSPORTATION, ONTARIO

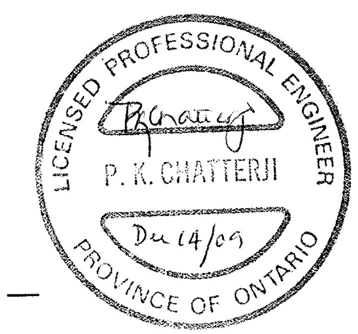
FILENAME: H:\Drafting\19\1351\180\MTO Drawings\bd1180-CawthraRoad.dwg





METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



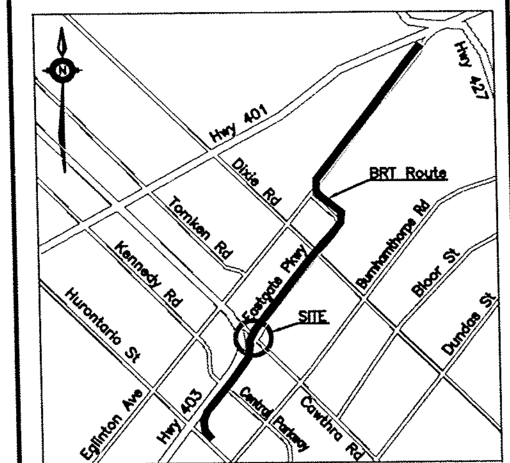
CONT No
GWP No

MISSISSAUGA BRT EAST
DETAILED DESIGN
GRADE SEPARATION STRUCTURE AT CAWTHRA RD
SOIL STRATA

SHEET

McCORMICK RANKIN CORPORATION

THURBER ENGINEERING LTD.
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
09-052	151.5	4 830 140.1	610 596.5
09-053	151.3	4 830 164.4	610 612.2
09-054	151.3	4 830 125.6	610 610.8
09-055	151.3	4 830 149.3	610 626.1

-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. 30M12-286

REVISIONS | DATE | BY | DESCRIPTION

DESIGN MEF | CHK PKC | CODE | LOAD | DATE DEC. 2009
DRAWN MFA | CHK MEF | SITE | STRUCT | DWG 2