FOUNDATION INVESTIGATION AND DESIGN REPORT
EXTENSION OF EAST ABUTMENT
CENTRAL PARKWAY OVERPASS AT HIGHWAY 403
MISSISSAUGA BUS RAPID TRANSIT (BRT) PROJECT
MISSISSAUGA, ONTARIO

Geocres Number: 30M12-307

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

July 2, 2010
File: 19-1351-160

H:\19\1351\160 Mississauga BRT Detailed Design\Reports & Memos\Central Parkway\Central Parkway-Hwy 403 FIDR Final.doc

TABLE OF CONTENTS

Appendices

FOUNDATION INVESTIGATION AND DESIGN REPORT
EXTENSION OF EAST ABUTMENT
CENTRAL PARKWAY OVERPASS AT HIGHWAY 403
MISSISSAUGA BUS RAPID TRANSIT (BRT) PROJECT
MISSISSAUGA, ONTARIO

Geocres Number: 30M12-307

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted for the proposed extension of the east abutment at the existing Highway 403 - Central Parkway overpass structure in Mississauga, Ontario. Extension of the abutment has been requested by MTO as part of the proposed Bus Rapid Transit (BRT) project to be constructed along the south side of the highway.

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 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 embankment 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 Central Parkway will include a bus station and a grade separation structure to carry the proposed BRT over Central Parkway. The space between the completed station and the existing Highway 403 overpass structure to the north will be limited, and therefore extension of the east abutment will be carried out as part of busway construction, in anticipation of future widening of Highway 403.

Currently the BRT site is a vacant strip of land bordered on the north by Highway 403 and on the south by residential development. Vegetation consists mainly of tall grass and occasional shrubs.

A photograph of the site is included in Appendix F, showing 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 on April 9, 2010 and consisted of one borehole drilled and cored to a total depth of 5.4 m adjacent to the south end of the existing abutment. The borehole was numbered 10-04 after several boreholes drilled at the same time for other components of the BRT project.

The approximate borehole location is shown on the Borehole Locations and Soil Strata Drawing in Appendix G. The coordinates and elevation of the borehole are given on this drawing and on the Record of Borehole Sheet in Appendix A. Also provided in Appendix G is the borehole plan and soil strata drawing for this section of the BRT previously prepared for the overall corridor report.

Prior to commencement of drilling, utility clearances were obtained for the borehole location.

Solid stem augers were used to advance the borehole 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 a 3.0 m length of core from the underlying shale bedrock.

A member of Thurber's engineering staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the borehole, 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 borehole were observed throughout the drilling operations. A standpipe piezometer consisting of 25 mm PVC pipe with screen was installed in the borehole to permit monitoring of groundwater levels. Details of the piezometer installation are as shown in Table 3.1.

Table 3.1 – Borehole Completion Details

Borehole	Piezometer Tip Depth/ Elevation (m)	Completion Details
10-04	5.4/137.4	Piezometer with 1.5 m slotted screen installed with sand filter to 3.3 m, bentonite from 3.3 m to 0.2 m, then concrete to 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 sheet 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 were carried out on selected samples of intact limestone interbeds upon arrival at the laboratory to assist in evaluation of the uniaxial compressive strength (UCS) of the bedrock. The results of the point load tests are shown in the borehole logs.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheet 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 Sheet governs any interpretation of the site conditions.

The soil stratigraphy encountered at this site comprises a surficial pavement structure overlying native silty clay till, which is in turn underlain by weathered shale bedrock. More detailed descriptions of the individual strata are presented below.

The conditions encountered in Borehole 10-04 drilled for the current investigation are consistent with those documented during earlier investigation for the overall BRT project, as illustrated on the Borehole Locations and Soil Strata drawing from the BRT report, Drawing No. 19-1351-160-2 included in Appendix G

5.1 Pavement Structure

Borehole 10-04 was drilled on Central Parkway. The pavement structure encountered in the borehole consisted of 180 mm of asphalt over approximately 1.0 m of compact to dense sand and gravel fill. The moisture content of the granular fill ranges from 3 to 5%.

5.2 Silty Clay Till

Native brown silty clay till was contacted below the pavement structure. The clay till layer was 0.9 m thick with a lower boundary at 2.1 m depth (Elevation 140.7 m).

An SPT ‘N’ value of 13 blows/0.3 m was obtained in the clay till, indicating a stiff consistency. The natural moisture content of a single sample was 19%.

The results of a grain size distribution analysis and Atterberg Limits testing are presented on the Record of Borehole sheets and on Figures B1 and B2 of Appendix B. The results are summarized as follows:

Soil Particles	**(%)**
Gravel	1
Sand	15
Silt	64
Clay	20

Liquid Limit	35
Plastic Limit	22

The above results show that the silty clay till is of low to medium plasticity with group symbols of CL-CI.

Glacial tills inherently contain cobbles and boulders and the lower part of the till may contain pieces and slabs of bedrock.

5.3 Bedrock

The glacial till was underlain by shale bedrock of the Georgian Bay Formation, encountered at 2.1 m depth (Elevation 140.7 m). The shale recovered in the rock cores was described as thinly bedded, highly weathered to a depth of about 0.6 m below the bedrock surface, and slightly weathered to fresh below this level. Occasional hard limestone interbeds were observed and a 100 mm thick layer of highly broken rock was noted at 2.4 m depth.

Total core recovery (TCR) in the bedrock was 100%, and RQD values of 89 and 95% were recorded, indicating a good to excellent rock quality. The fracture Index (FI), expressed as fractures per 0.3 m of core, ranged from 0 to 3, locally greater than 5 in the uppermost 0.3 m of the initial run.

The unconfined compressive strength of the interbedded limestone assessed from Point Load testing on recovered core Run 2 was 111 MPa, indicating a very strong rock. Point load tests were possible only on the limestone interbeds as the more typical weathered shale cores tended to split along bedding planes and were not suitable for testing. Based on point load and unconfined compression testing carried out on shale cores from other areas of the BRT project, the shale strength ranges from about 3 to 30 MPa, indicating a very weak to medium strong rock.

5.4 Groundwater Levels

Rock coring operations introduced water into the borehole. The unstabilized water level observed in the borehole upon completion of coring was at 0.7 m depth.

A standpipe piezometer was installed in the borehole to monitor water levels after completion of drilling. The water level measured in the piezometer approximately three weeks after drilling (April 30, 2010) was at 1.3 m depth (Elevation 141.6 m).

The measured water level is a short-term reading and seasonal fluctuations 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.

6 MISCELLANEOUS

The location and ground surface elevation at the borehole were established by Thurber Engineering using a Trimble Pathfinder ProXRT GPS unit with a precision of 0.1 m.

The drilling and sampling equipment was supplied and operated by DBW Drilling of Ajax, Ontario. The fieldwork was supervised on a full time basis by Mr. George Azzopardi of Thurber Engineering Ltd. Overall supervision of the field program was conducted by Mr. Mark Farrant, P. Eng.

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

Interpretation of the data and preparation of the report were carried out by Mr. Murray R. Anderson, P.Eng. Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects, reviewed the report.

THURBER ENGINEERING LTD.

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

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
July 2/10
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
July 2/10
PROVINCE OF ONTARIO

FOUNDATION INVESTIGATION AND DESIGN REPORT
EXTENSION OF EAST ABUTMENT
CENTRAL PARKWAY OVERPASS AT HIGHWAY 403
MISSISSAUGA BUS RAPID TRANSIT (BRT) PROJECT
MISSISSAUGA, ONTARIO

Geocres Number: 30M12-307

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 selection and design of a suitable foundation system for the proposed abutment extension at the Highway 403 - Central Parkway overpass.

Based on the preliminary General Arrangement (GA) drawing provided by McCormick Rankin Corporation, the east abutment will be extended to the south by approximately 4.1 m. The new abutment will include an adjoining 10.3 m long retaining wall constructed parallel to Highway 403.

The General Plan drawing for the existing overpass structure (Contract No. 80-37) indicates that the existing abutment foundation design consists of spread footings with a top-of-footing at Elevation 143.1 m (typ.). The existing retaining wall parallel to the highway is supported on a granular pad, with the top-of-footing stepping up to Elevation 146.2 m a distance of 7.0 m behind the abutment.

The discussion and recommendations presented in this report are based on the information provided and on the factual data obtained in the course of the investigation.

8 FOUNDATION DESIGN

In general terms, the soil stratigraphy encountered at this site consists of a surficial pavement structure overlying stiff silty clay till, underlain by shale bedrock at 2.1 m depth. The groundwater level was measured at 1.3 m depth.

Consideration was given to various alternate foundation systems, taking account of the site stratigraphy and the foundations supporting the existing structure. Initial consideration was given to the following foundation types:

- Spread footings on native silty clay till
- Spread footings on weathered 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.

As noted above, the existing structure is supported on spread footings. To provide consistent performance with the existing foundation system, and in view of the shallow depth to shale bedrock, it is recommended that the abutment extension be supported on spread footings as well. The use of deep foundations (caissons and piles) is not justified and these alternatives are not considered further.

8.1 Spread Footings on Native Silty Clay Till

The geotechnical resistance available in the stiff native clay till is considered inadequate for design of spread footings of practical dimensions to support abutment loads. Further, the till layer is thin (0.9 m) and underlain by competent shale bedrock at shallow depth. Construction of spread footings on native clay till is therefore not recommended, and this option has not been developed.

8.2 Spread Footings on Weathered Shale Bedrock

Spread footings may be extended down to the underlying shale bedrock to achieve a higher bearing resistance than is available in the overlying clay till. The footings should be founded on or below the shale surface encountered at 2.1 m depth (Elevation 140.7 m) in Borehole 10-04.

Spread footings bearing on undisturbed weathered shale bedrock may be designed for the following geotechnical resistance:

Factored geotechnical resistance at Ultimate Limit States = 750 kPa

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

The possibility exists that the existing abutment footing is constructed on clay till, as the founding level and bearing stratum of the structure have not been confirmed by exposing the existing footing. If so, excavation to shale for the footing extension may extend below the level of the existing footing. The geometry of the excavation adjacent to and below the level of the existing footing must be carefully controlled to prevent undermining and disturbance of the existing foundation. This can be achieved by stepping the excavation below the founding level at an inclination of 1H:1V downwards and outwards from the edge of the existing footing base, as illustrated in Figure 1, Appendix D.

These resistance values are 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

Consideration may be given to placing the retaining wall footings on engineered fill similar to the design of the existing retaining wall.

If an engineered fill pad is used at this site, all topsoil, fill, 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 native, undisturbed, stiff silty clay till or shale. The highest permitted elevation at which engineered fill should be placed is Elevation 141.3 m.

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 2 in Appendix D.

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

> Factored geotechnical resistance at Ultimate Limit States = 900 kPa
>
> Geotechnical resistance at Serviceability Limit States = 350 kPa

To achieve the above resistances the engineered fill pad should be at least 2.0 m thick where founded on stiff clay till. A minimum pad thickness is not required when the engineered fill is placed directly on shale.

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 along the length of the wall.

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

8.4 Frost Protection

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

Although the shale is geologically defined as bedrock, it is susceptible to frost action. Therefore, footings on shale must also be provided with a minimum of 1.2 m of earth cover.

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 shale at the same level as the existing abutment foundation are recommended for the abutment extension. The retaining wall footings should be placed on shale or on granular engineered fill.

9 TEMPORARY EXCAVATION

Earth excavations required at this site will penetrate primarily through pavement and native silty clay till. The till may contain cobbles, boulders and slabs of 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.

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 impact of excavation in close proximity to existing facilities must be assessed and measures to control impacts should be included in the design. Care must be taken during footing excavation to avoid disturbing and undermining adjacent roadways, structures, underground utilities, and in particular the existing abutment and retaining wall footings.

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.
- 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, and possible dislodging and removal of cobbles, boulders and rock slabs.

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

The requirements for unwatering during excavation are discussed in Section 11.

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.

10 ROADWAY PROTECTION

It is anticipated that roadway protection will be required at Central Parkway East during construction. An item titled "Protection System" as per OPSS 539 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.01.01 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 bedrock to develop the required toe resistance.

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

11 UNWATERING

Temporary excavation for construction of footings founded on bedrock may extend below the groundwater level measured in the piezometer. Considering the consistency and low permeability of the clay till soils, groundwater control measures such as perimeter ditches and pumping from filtered sumps should be suitable 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, or if concentrated seepage is experienced from seams or fractures near the shale bedrock surface. Footings founded on shale bedrock 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.

12 APPROACH EMBANKMENTS

The foundation soils governing stability of the approach embankments consist of existing native stiff silty clay till underlain by shale bedrock. It is anticipated that the maximum embankment height will be about 6.5 m.

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002. It is recommended that earth fill should consist of SSM, granular materials or local inorganic till in compliance with Special Provision 110F13, "Amendment to OPSS 1010 March 1993". Excavated shale bedrock must not be used for approach embankment construction.

The embankment foundation soils are considered to provide adequate stability to new earth fills inclined at 2H:1V or flatter.

Considering the embankment height and consistency of the foundation soils, post construction settlement induced by embankment loading will be less than 25 mm.

All topsoil and organic soils should be stripped from the footprint of the approach fills. Particular attention should be paid to removing all organics and softened material from existing ditches that fall within the footprint of the new embankment.

Prior to placement of new fill against the existing embankment slope, the existing earth slope should be benched in accordance with OPSD 208.010.

Earth fill embankment slopes must be provided with erosion protection in accordance with SP572S01.

13 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.

14 STATIC EARTH PRESSURE

Earth pressures acting on the abutment and retaining wall may be assumed to be triangular and to be governed by the characteristics of the wall 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 14.1)

γ = unit weight of retained soil (see Table 14.1)

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 14.1.

Table 14.1 – Earth Pressure Coefficients (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS 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 Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.46*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

* For retaining 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 14.1 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.

15 SEISMIC CONSIDERATIONS

15.1 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.

15.2 Liquefaction Potential

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

15.3 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 15.1 may be used:

Table 15.1 – Earth Pressure Coefficient for Earthquake Loading

	Earth Pressure Coefficient (K) for Earthquake Loading			
	OPSS 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	
Wall Condition	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
At Rest (K_{OE})**	0.54	-	0.58	-
Passive (K_{PE})	3.6	-	3.1	-

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

16 CONSTRUCTION CONCERNS

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

1. Excavations

   Care must be exercised during excavation to avoid disturbing the existing abutment foundations and new footing subgrade. The exposed subgrade soils should be expeditiously inspected, approved and protected from disturbance.

   Cobbles, boulders and rock slabs should be expected within the till layer.

2. Shale bedrock protection

   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.

3. Groundwater control

   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.

4. Undulations in surface of the shale bedrock.

   The elevation of the top of bedrock may vary between and beyond the exploratory boreholes drilled for this structure. The QVE must ensure that footings specified to be founded on bedrock are indeed founded on bedrock. Where it is found that the top of the bedrock lies below the design underside of the footing, the overburden must be stripped to expose the bedrock and the resulting difference in level made up using concrete of the same class as the footing.

5. Existing underground utilities and roadways

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

17 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Murray R. Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

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

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
July 2/10
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
July 2/10
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

THURBER

RECORD OF BOREHOLE No 10-04

METRIC

G.W.P. 19-1351-160 LOCATION N 4 829 221.8 E 610 040.4 ORIGINATED BY GA

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

DATUM DATE 2010.04.09 - 2010.04.09 CHECKED BY MA

ELEV / DEPTH (142.9)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**ASPHALT:** (180mm)					
141.6 / 0.2	**SAND** and **GRAVEL** Compact to Dense Brown Dry (FILL)	1	SS	50/ 0.150		
		2	SS	20		
140.7 / 1.2	Silty **CLAY**, some sand, trace gravel Stiff Brown (TILL)	3	SS	13		1 15 64 20
137.4 / 2.1	**SHALE**, highly weathered, thinly bedded, very weak to weak, grey, occasional very strong limestone interbeds slightly weathered to fresh 100mm highly broken zone at 2.4m	4	SS	50/ 0.150	FI	
		1	RUN		>5, 1, 1, 1, 0	RUN 1# TCR=100%, SCR=93%, RQD=89%
		2	RUN		0, 0, 1, 1, 3	RUN 2# TCR=100%, SCR=100%, RQD=95% UCS=111MPa (Limestone)
5.4	END OF BOREHOLE AT 5.4m. BOREHOLE OPEN TO 5.4m AND WATER LEVEL AT 0.7m UPON COMPLETION. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2010.04.30 1.3 141.6					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

ELEVATION SCALE 142 141 140 139 138

+³, ×³: Numbers refer to Sensitivity

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

Appendix B

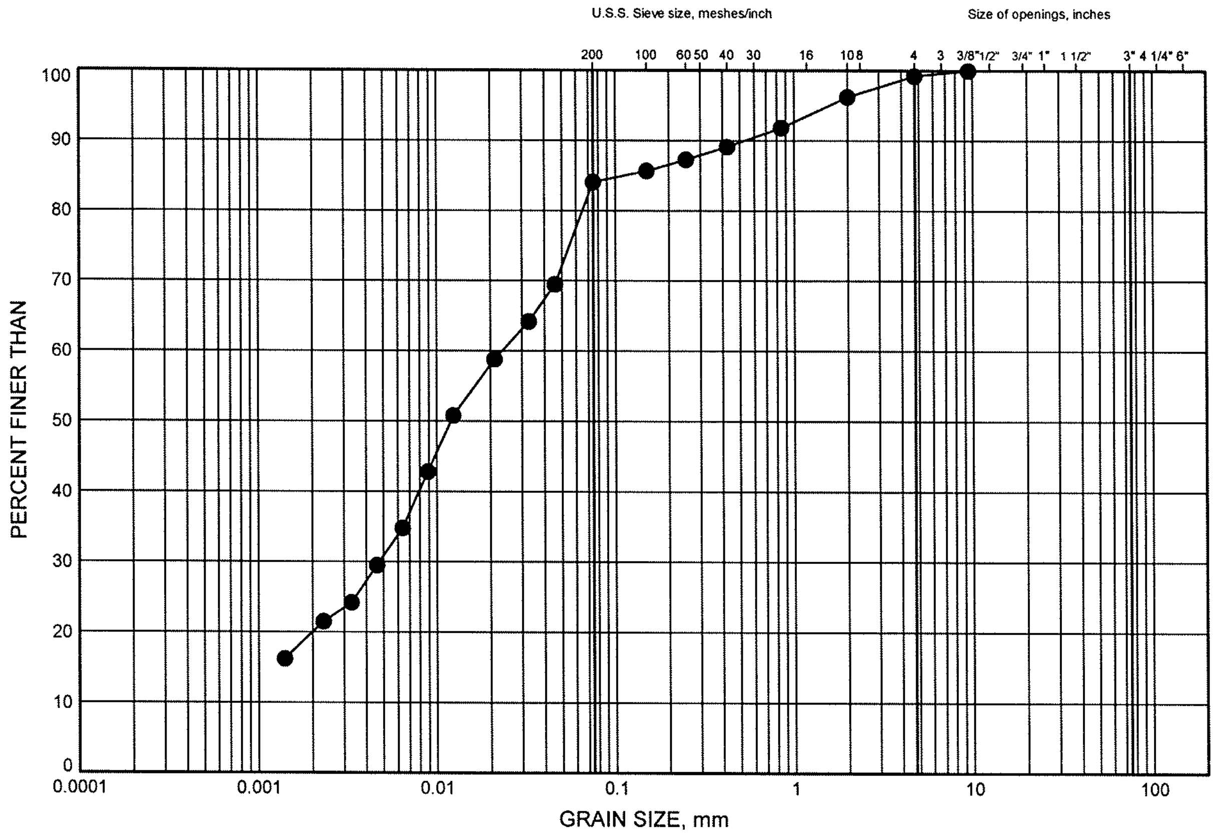
Laboratory Test Results

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)
●	10-04	1.83	141.03

THURBER

W.P.# 19-1351-160

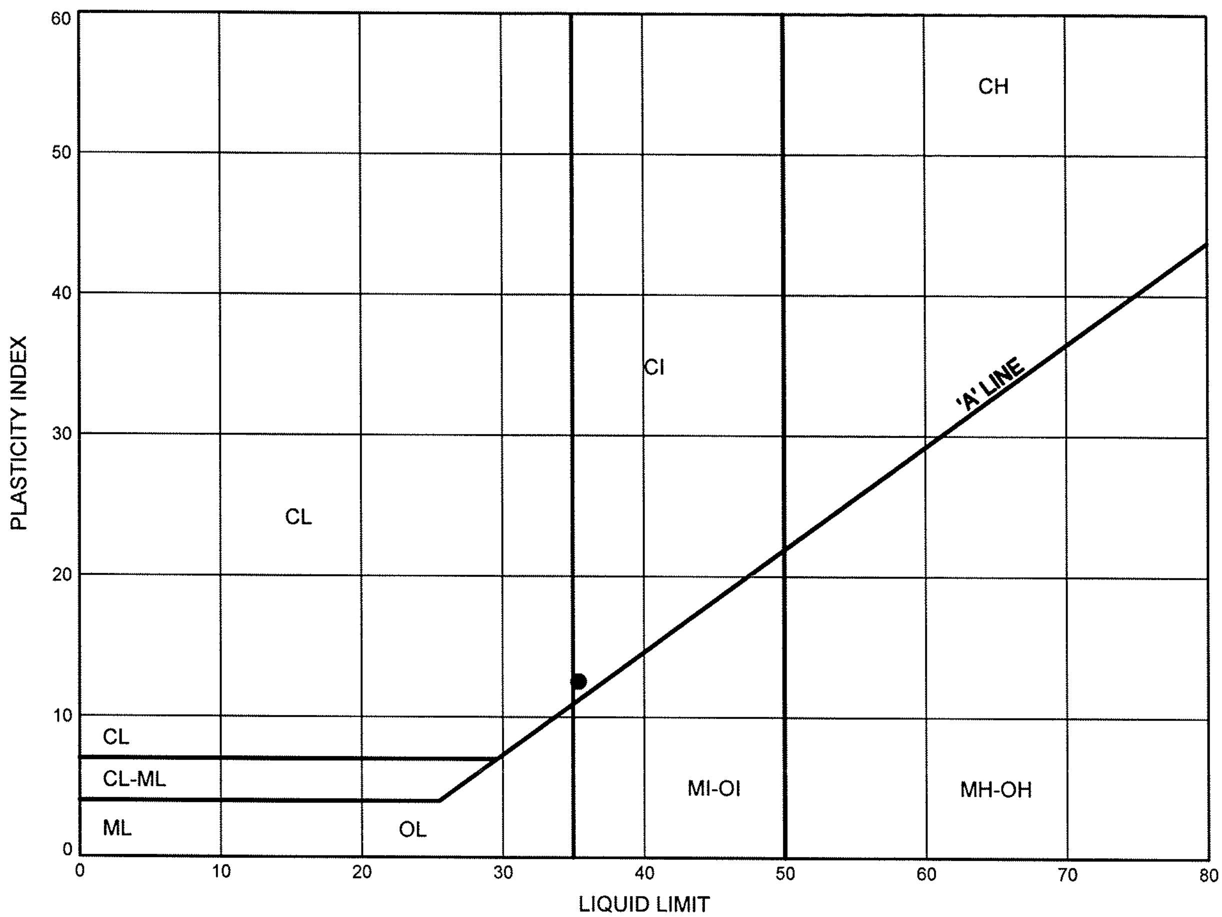
Prepared By MFA

Checked By MRA

ATTERBERG LIMITS TEST RESULTS

FIGURE B2

SILTY CLAY TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	10-04	1.83	141.03

THURBALT 1160(MTO).GPJ 5/18/10

Date May 2010

Project 19-1351-160

THURBER

Prep'd MFA

Chkd. MRA

Appendix C

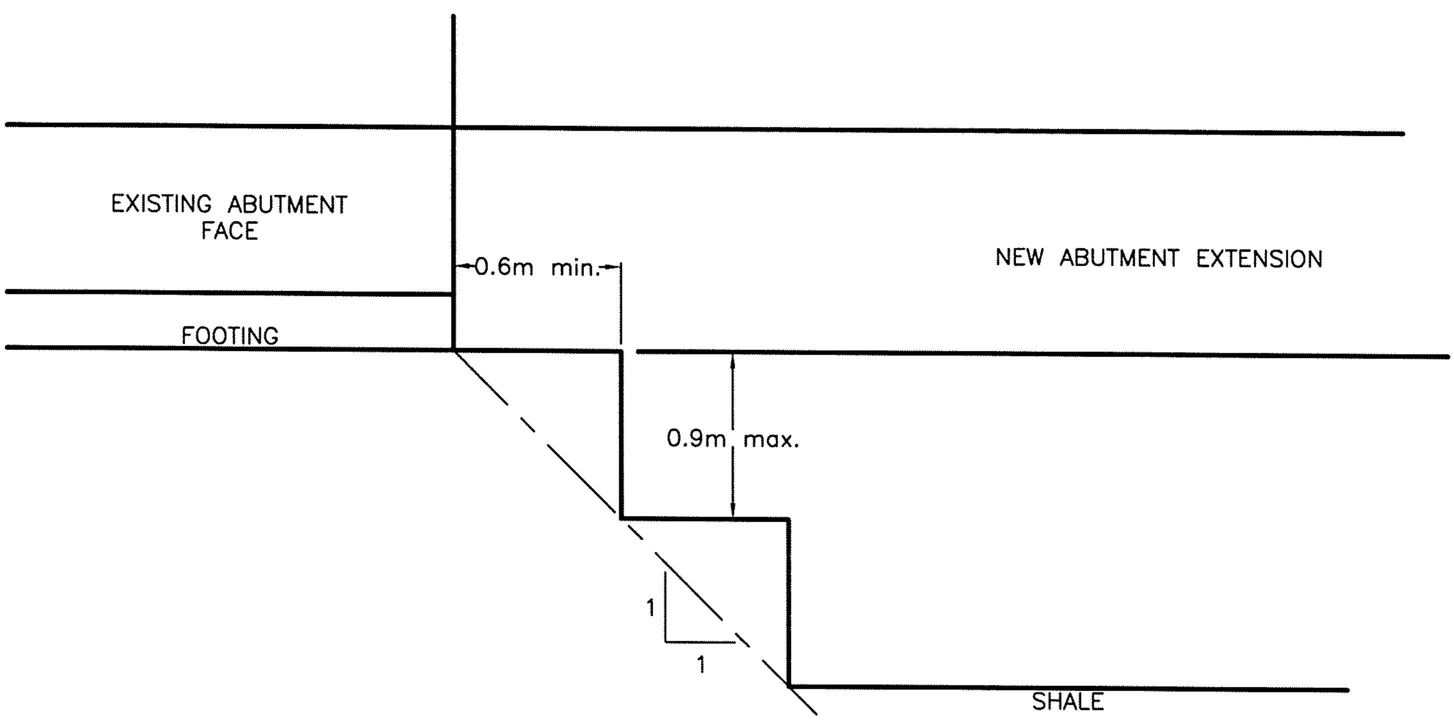
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Engineered Fill	Footings on Shale	Driven Piles	Caissons
Advantages: i. Ease of construction. ii. Lower cost than deep foundations. iii. Reduced depth of excavation compared to footings on engineered fill or shale. ***Disadvantages:*** i. Low geotechnical resistance in soils on site. ii. Dewatering may be required, depending on the depth of excavation. iii. Not suitable for integral abutment design. **NOT RECOMMENDED**	***Advantages:*** i. Generally less costly construction than deep foundation elements. ii. Founding level can be adjusted. iii. Higher bearing resistance than native soils. ***Disadvantages:*** i. Dewatering may be required, depending on depth of excavation for fill placement. ii. Cost of engineered fill placement **FEASIBLE**	***Advantages:*** i. High values of geotechnical resistance are available on the bedrock. ii. Relatively simple construction method iii. Similar to existing abutment foundation. ***Disadvantages:*** i. Subexcavation required to penetrate the native soils. ii. Higher cost of deeper excavation to bedrock. iii. More likely to encounter groundwater than footing on native soils. **RECOMMENDED**	***Advantages:*** i. Piles will develop high geotechnical resistance on bedrock. ii. Installation of piles could continue in freezing weather. iii. Accommodates integral abutment design. iv. Foundation construction requires less excavation than footings ***Disadvantages:*** i. Higher unit cost compared to footings. ii. Pre-augering will be required in order to install the piles to adequate length. iii. Potential difficulties penetrating hard limestone layers in shale. **NOT RECOMMENDED**	***Advantages:*** i. High resistance is available for caissons founded on bedrock. ii. Construction of caissons could continue in freezing weather. ***Disadvantages:*** i. Higher cost than spread footings ii. Possibility of boulders being encountered during augering. iii. Potential difficulties penetrating hard limestone layers in shale. iv. More likely to encounter groundwater. v. Potential difficulty in cleaning and inspecting bases. **NOT RECOMMENDED**

Appendix D

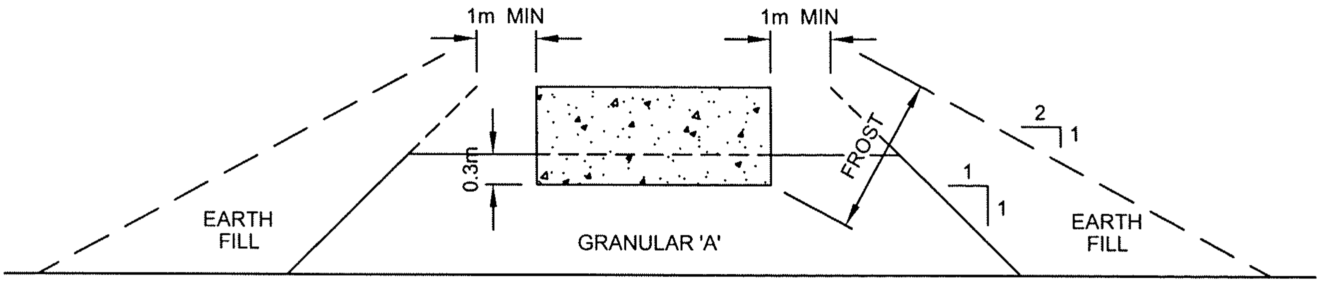
Figures



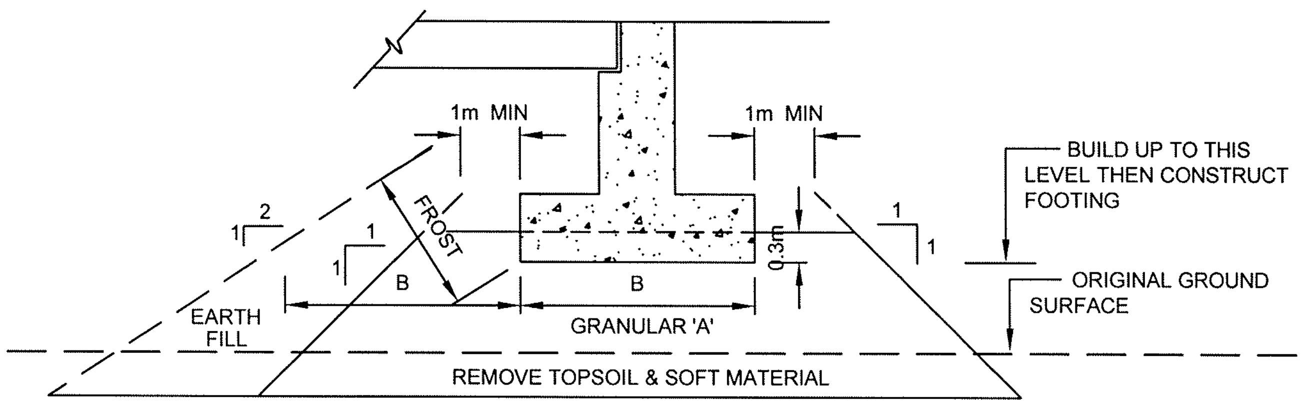
STEPPED EXCAVATION ADJACENT TO
EXISTING ABUTMENT FOOTING

THURBER ENGINEERING LTD.
GEOTECHNICAL ▪ ENVIRONMENTAL ▪ MATERIALS

ENGINEER:	DRAWN:	APPROVED:
MRA	MFA	PKC
DATE:	SCALE:	DRAWING No.
MAY 2010	N.T.S.	FIGURE 1



CROSS-SECTION



LONGITUDINAL SECTION

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO 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.

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE

THURBER ENGINEERING LTD.
GEOTECHNICAL ▪ ENVIRONMENTAL ▪ MATERIALS

ENGINEER:	DRAWN:	APPROVED:
MRA	MFA	PKC
DATE:	SCALE:	DRAWING No.
MAY 2010	N.T.S.	FIGURE 2

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 110F13
- SP 572S01
- SP 902S01
- OPSS 206
- OPSS 539
- OPSS 902
- OPSS 1010
- OPSD 3101.150
- OPSD 3102.100

2. Suggested Text for NSSP on "Native Silty Clay Till"

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

Arduous excavation may be experienced near the transition between the till and underlying shale bedrock.

Appendix F

Site Photograph

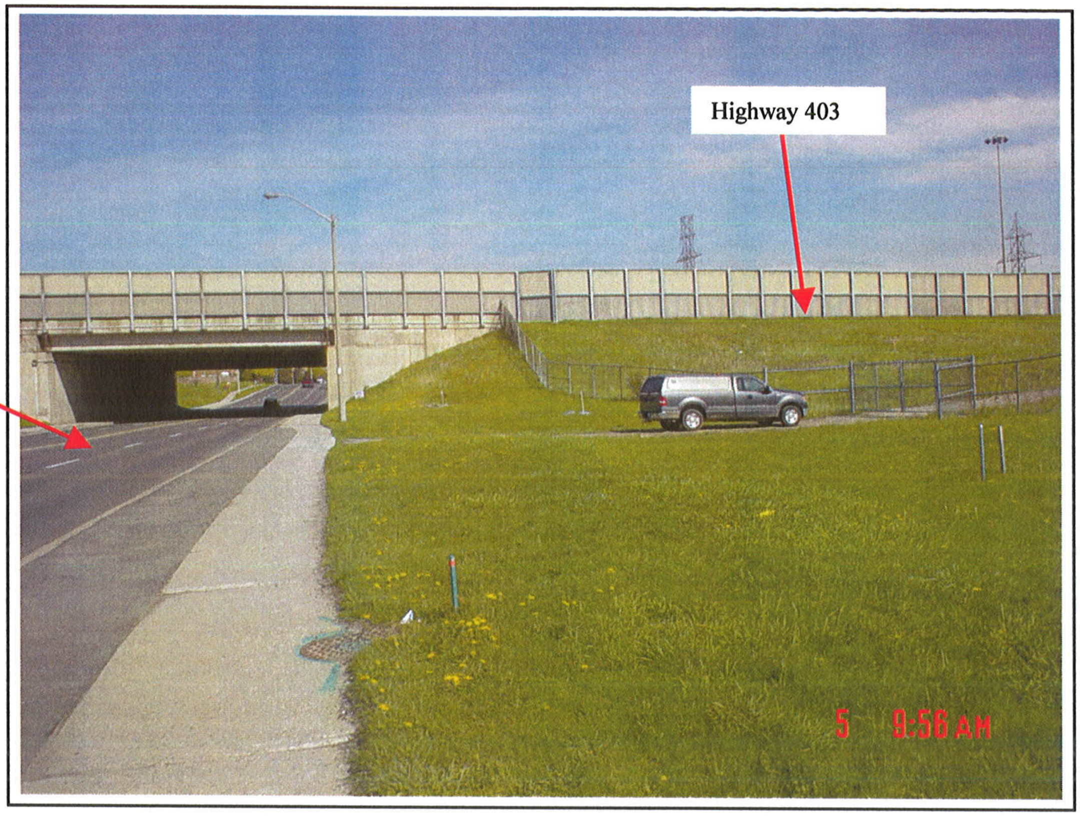
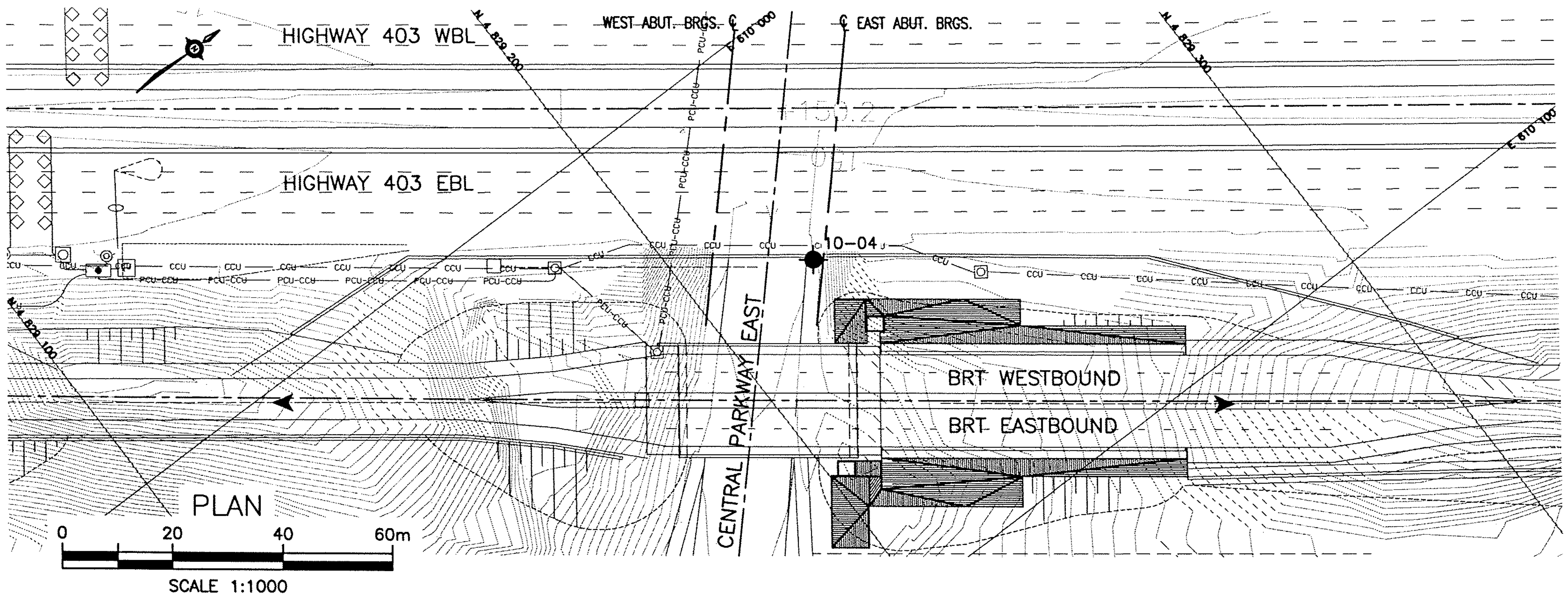
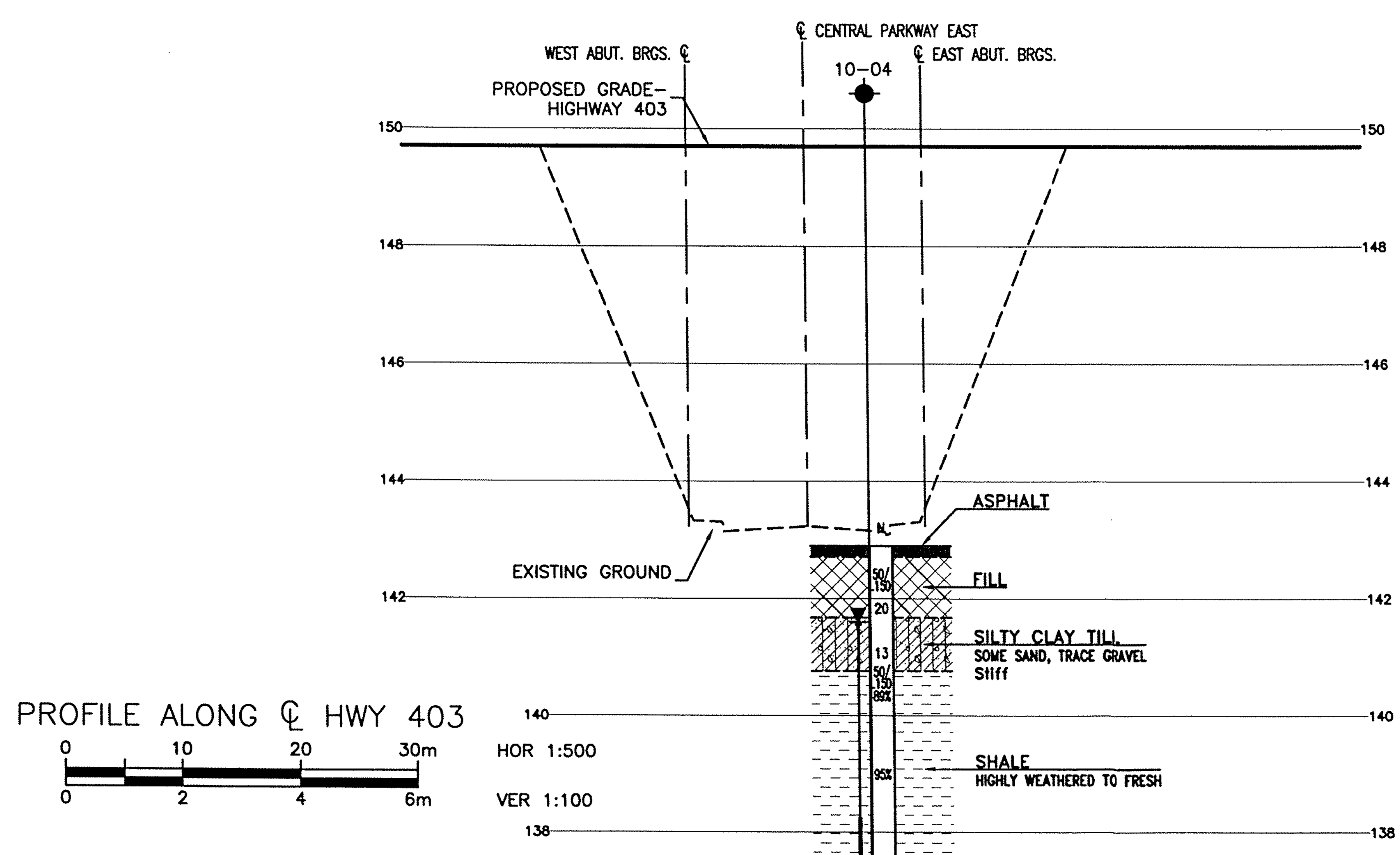


Photo 1. Looking at the south side of Highway 403 and Central Parkway intersection

Appendix G

Borehole Locations and Soil Strata Drawing





METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No

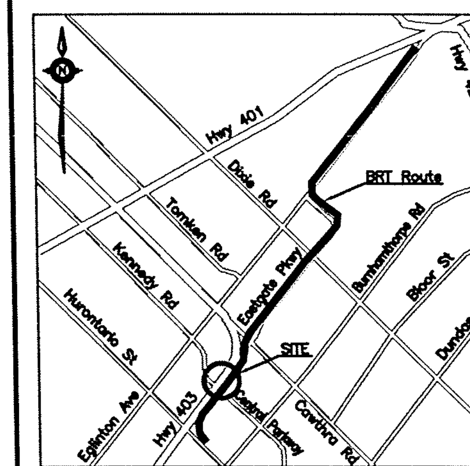
GWP No

SHEET

MISSISSAUGA BRT EAST
HIGHWAY 403 / CENTRAL PARKWAY EAST OVERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

MRC 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
10–04	142.9	4 829 221.8	610 040.4

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
July 2/10
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
July 2/10
PROVINCE OF ONTARIO

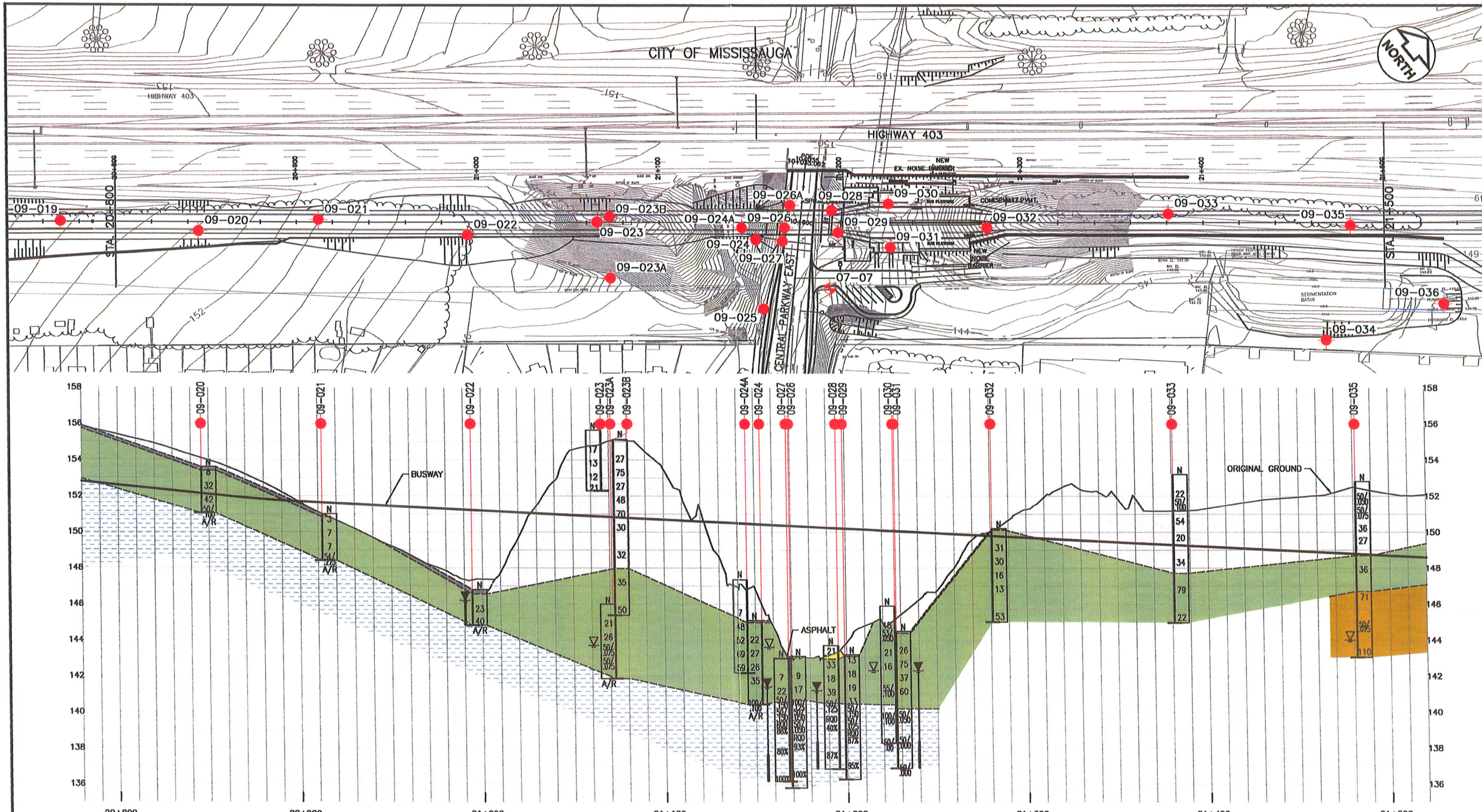
-NOTES-

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. 30M12-307

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN MRA	CHK PKC	CODE	LOAD	DATE JUN. 2010	
DRAWN MFA	CHK AEG	SITE	STRUCT	DWG 1	

PLOTDATE: Jun 30, 2010 – 4:37pm



LEGEND:

	PRESENT BOREHOLE LOCATION		SANDY SILT TILL / SILTY SAND TILL
	PREVIOUS BOREHOLE LOCATION		SANDY SILT / SILTY SAND
	TOPSOIL		SAND / SAND AND GRAVEL
	FILL		BEDROCK (SHALE)
	SILTY CLAY TILL / CLAYEY SILT TILL		GROUNDWATER LEVEL IN STANDPIPE PIEZOMETER
	SILTY CLAY / CLAYEY SILT		OPEN BOREHOLE GROUNDWATER LEVEL (UNSTABILIZED)

McCORMICK RANKIN CORPORATION

MISSISSAUGA BUS RAPID TRANSIT - EAST SECTION
CENTRAL PARKWAY
STATION 20+800 TO 21+500
BOREHOLE LOCATIONS AND SOIL STRATA

19-1351-160

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

ENGINEER:	DRAWN:	APPROVED:
MEF	MFA	PKC
DATE: DECEMBER 2009	SCALE: 1:2000 HOR 1:200 VER	DRAWING No 19-1351-160-2

FILENAME: H:\Drafting\19\1351\160\ted1160(02)-Plan&Profile(20+800-21+500).dwg