

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HURONTARIO STREET UNDERPASS
HWY 401 WIDENING, HWY 410 TO CREDIT RIVER
MISSISSAUGA, ONTARIO
G.W.P. 2149-01-00 & 2150-01-00, SITE 24-132**

Geocres Number: 30M12-268

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at Highway 401 and Hurontario Street interchange in Mississauga, Ontario. The project involves widening of Highway 401 and replacement of the structure carrying Hurontario Street over Highway 401.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to 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 MMM Group Limited (MMM), under the Ministry of Transportation Ontario (MTO) Agreement Number 2005-A-000347.

In the preparation of this report and in addition to the boreholes drilled under the current assignment, reference has been made to information on subsurface conditions contained in an earlier foundation report under G.W.P. 54-82-02. The title of this report is listed as follows:

- Engineering Materials Office, report titled "Foundation Design Section, Highway 10 Underpass, Highway 401, District 6, Toronto", G.W.P. 54-82-02, Site 24-132, GEOCRETS No. 30M12-183, dated August 1994 (Reference 1).

2 SITE DESCRIPTION

The site is located on Hurontario Street at the Highway 401 and Hurontario Street interchange in Mississauga, Ontario.

The lands to the northwest quadrant of Highway 401 and Hurontario Street are generally vacant, undeveloped and/or agricultural. Vegetation consists mainly of tall grass and shrubs. To the east of Hurontario Street and south of Highway 401, lands have been developed for commercial and industrial uses. The topography is generally flat.

The general site area is located within the physiographic region known as Peel Plain, characterized by a level to undulating cohesive glacial till typically less than 3 m thick underlain by reddish brown shale or limestone of the Queenston Formation.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out from October 30 to December 11, 2006 and consisted of drilling and sampling eight boreholes (numbered H1 to H8) at the site. Boreholes were drilled at the projected locations of foundations elements (piers and abutments) and approaches along the alignment of the Hurontario Street and Highway 401 interchange structure.

Borehole H2, H3 and H8 were terminated in shale bedrock at depths of 4.6 m to 15.3 m with SPT 'N' values of at least 100 blows for 0.3 m of penetration. Borehole H5 was terminated in the fill layer upon refusal to auger due to the presence of an obstruction at 6.1 m depth (elevation 194.0 m). Borehole H7 drilled at the north approach was terminated within the native silty clay till layer at 8.1 m depth (elevation 191.1 m). Boreholes H1, H4 and H6 drilled at or near the foundation elements were advanced into shale bedrock by coring to depths of 14.8 m to 19.8 m (elevations 180.2 m to 182.8 m), with a minimum 3.0 m rock core recovered in each borehole.

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. Records of boreholes drilled during the previous investigation (1983) are enclosed in Appendix C.

The survey for this earlier investigation was apparently carried out using the NAD 27 datum. The coordinates have been converted to NAD 83 and the boreholes (numbered 1 to 8) have been plotted on the Borehole Locations and Soil Strata drawing on the basis of the converted coordinates.

Prior to commencement of drilling, utility clearances were obtained for all borehole locations. Road occupancy and lane closure permits were also obtained.

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

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

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations and standpipe piezometers consisting of 19 mm PVC pipe with a slotted screen were installed in five of the boreholes to permit longer term groundwater level monitoring. The boreholes in which no piezometers were installed were grouted with bentonite. Details of the piezometer installations and other borehole completion details are as shown in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Element	Borehole	Piezometer Tip (Sand Filter) Details			Backfill
		Depth (m)	Elevation (m)	Stratum	
North Approach	H7	None installed			Bentonite grout for full depth
North abutment	H1	None installed			Bentonite grout for full depth
	H2	13.5 – 15.3	186.3 – 184.5	Shale Bedrock	Bentonite to 0.3 m and concrete to surface.
Pier 1	H3	2.8 – 4.6	191.7 – 189.8	Shale Bedrock/Till	Bentonite to surface.
	H4	16.7 – 19.8	183.3 – 180.2	Shale Bedrock	Bentonite to surface.
Pier 2	H5	None installed			Bentonite grout for full depth
South Abutment	H6	15.0 – 16.8	184.6 – 182.9	Shale Bedrock	Bentonite to 14.0 m, grout to 2.4 m, bentonite to 0.3 m and concrete to surface.
South Approach	H8	7.6 – 9.4	190.4 – 188.6	Shale Bedrock/Till	Bentonite to surface.

4 LABORATORY TESTING

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

Core samples of the shale bedrock were carefully protected to prevent drying during transport to the laboratory. Point load tests were carried out on selected samples of intact shale upon arrival at the Thurber laboratory to assist evaluation of the compressive strength of the bedrock. Two shale samples (Borehole H4, Run 1 and Borehole H6, Run 2) were subjected to unconfined compression

tests. The results of point load and unconfined compression tests on rock cores retrieved from the boreholes are shown on the Record of Borehole sheets in Appendix A and also in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendices A and C. Details of the encountered soil and rock stratigraphy are presented in these appendices and on the “Borehole Locations and Soil Strata” drawing in Appendix 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 consists of topsoil or pavement structure overlying fill which is underlain by native silty clay till and occasional silty sand. Weathered shale bedrock was contacted below the till deposits. More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

Pavement structure consisting of approximately 100 to 150 mm of asphalt overlying granular (sand and gravel fill) road base was encountered in Boreholes H2, H5 to H7 drilled on Hurontario Street lanes. The thickness of granular fill measured in the boreholes ranged from 0.6 to 1.4 m and the underside lay at elevations 198.2 m to 198.9 m.

5.2 Topsoil

Topsoil was identified surficially at the proposed locations of North Abutment, Pier 1 and South Approach (Boreholes H1, H3, H4 and H8). The topsoil thickness generally ranged from 50 mm to 225 mm. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.3 Fill

A layer of soil was encountered that was identified as fill on the basis of its composition and disturbed appearance. The fill was encountered in all the boreholes drilled along the entire project area: North and South Approaches, North and South Abutments and Piers 1 and 2.

Fill was contacted below the topsoil or below the pavement structure in Boreholes H1 to H8 from the current investigation and surficially in Boreholes 1 to 8 from the previous investigation (1983). The fill generally consists of cohesionless layers of brown sand and gravel with some silt, trace of clay and occasional shale fragments and cohesive layers of brown to reddish brown silty clay fill with some sand and trace of gravel. Cobbles were also observed within the fill.

Fill extended to depths ranging from 4.4 m to 9.4 m (elevations 192.2 m to 189.7 m) at the locations of proposed approaches, east side of piers and both abutments. Locally in

Boreholes 2, 7, 8 and H3 drilled from the level of Highway 401, the fill extended to depths ranging from 0.6 m to 1.4 m (elevations 193.8 m to 191.1 m).

After several attempts, borehole H5, located on the east side of Pier 2 was terminated within the fill layer at 6.1 m (elevation 194.0 m) upon consistent auger refusal.

The results of gradation and Atterberg Limit Tests conducted on samples of fill are summarized below:

Soil	(%)
Gravel	0 to 73
Sand	27 to 69
Silt	9 to 53
Clay	9 to 27

Liquid Limit	22 to 39
Plastic Limit	13 to 21

The grain size distribution of the cohesionless fill is represented by the data plotted in Figures B3 and B4 in Appendix B. The grain size distribution results of the cohesive fill are presented in Figures B1 and B2 in Appendix B. The results of Atterberg limit testing, as plotted in Figure B8 of Appendix B, indicate that the clay is low to medium plastic.

SPT N-values collected in the silty clay fill ranged from 5 to 56 blows per 0.3 m penetration indicating a firm to hard consistency. SPT N-values of the sand and gravel fill ranged from 5 to 85 blows per 0.3 m penetration indicating a loose to very dense relative density. Locally in Borehole H4, very loose silty sand fill was apparently contacted at 6.1 m depth (elevation 193.9 m) possibly due to some disturbance by groundwater. SPT N-values greater than 50 blows per 0.15 m penetration were observed in Borehole H2, possibly due to the presence of limestone fragments. The moisture content of samples collected ranged from 2% to 21%.

Although not directly encountered in the boreholes, the existing fill may contain cobbles and boulders. Borehole H5 was terminated on an obstruction and other obstructions such as limestone slabs or concrete slabs or building rubble may be encountered in the fill during construction.

5.4 Silty Clay Till

A deposit of native, brown to grey silty clay till with trace to some sand, trace to some gravel and occasional shale fragments was contacted below the fill in all the boreholes, except in Boreholes H1, H5 and 1, drilled on the west side of the North Abutment, Pier 2 and South Abutment, respectively.

The till deposit extended to depths of at least 8.1 m and 8.8 m (elevations 191.1 m and 189.2 m) at the North and South Approaches, respectively. At the North and South Abutments, the till extended to depths ranging from 13.7 m to 10.9 m (elevations 186.1 m to 188.9 m). Boreholes drilled in close proximity to Piers 1 and 2 revealed till extending to depths ranging from 4.0 m to 10.7 m (elevations 188.3 m to 190.5 m). Locally the till extended to 15.2 m depth (elevation 184.8 m) in Borehole H4 drilled at the proposed Pier 1. The silty clay till extended to depths ranging from 3.5 m to 2.5 m (elevations 189.0 m to 190.1 m) in boreholes drilled on Highway 401 centreline (Boreholes 7 and 8).

The results of gradation and Atterberg Limit Tests carried out on samples of the till are summarized below:

Soil	(%)
Gravel	0 to 7
Sand	21 to 44
Silt	36 to 46
Clay	9 to 37

Liquid Limit	23 to 35
Plastic Limit	14 to 19

Grain size distribution results from the silty clay till are presented on the Record of Borehole sheets and Figures B5 and B6 of Appendix B. Atterberg Limit testing result is presented in Figure B9 of Appendix B.

Based on SPT N-values generally ranging from 14 blows for 0.3 m of penetration to higher than 50 blows per 0.1 m, the deposit is classified as stiff to hard in consistency. The measured natural moisture contents range from 3% to 18% and locally 22% in Borehole BH7.

Although not encountered in the boreholes, glacial till may contain cobbles and boulders.

5.5 Silty Sand

A 1.5 m thick layer of native silty sand with trace to some clay was contacted below the silty clay fill and extending to 9.1 m depth (elevation 188.1 m) at the North Abutment (Borehole H1).

The results of gradation tests carried out the silty sand are summarized below:

Soil	(%)
Gravel	0
Sand	50
Silt & Clay	50

Laboratory results are presented in Figure B7 of Appendix B.

SPT N-value was 50 blows per 0.15 m penetration, indicating a very dense relative density. Moisture content was 18%.

5.6 Bedrock

The soils described above were found to be underlain by reddish brown weathered shale bedrock. The shale encountered in the boreholes is described as thinly bedded and contains numerous very strong interbedded siltstone and limestone layers. Rubble zones were contacted within the bedrock at depth ranging from 14.1 m to 14.6 m (elevations 185.6 m to 185.1 m) in Borehole H6 and at 11.9 m and 13.3 m depth (elevations 185.4 and 184.0) in Borehole H1.

SPT N-values obtained in the shale bedrock were 85 blows per 0.3 m penetration to greater than 50 blows per 0.10 m penetration. Moisture contents ranged from 5% to 14%.

At the locations of Boreholes H2 and H4 (east side of North Abutment and Pier 1), bedrock was contacted 4 to 5 m deeper than other current and previous boreholes drilled in the vicinity of Boreholes H2 and H4.

Elevations of the top of bedrock are shown in Table 5.1.

Table 5.1 – Elevation of Top of Bedrock

Foundation Element	Borehole	Depth (m)	Bedrock Elevation (m)
North Approach	H7	-	-
North abutment	H1	9.1	188.1
	H2	13.7	186.1
Pier 1	H3	4.0	190.5
	H4	15.2	184.8
	3	7.0	188.7
	4	9.8	190.1
Highway 401 Centreline	7	3.5	189.0
	8	2.5	190.1
Pier 2	H5	-	-
	2	4.4	188.3
	5	10.7	189.3
South Abutment	H6	11.7	187.9
	1	4.4	190.3
	6	10.9	188.9
South Approach	H8	8.8	189.2

Bedrock core was collected using NQ sized coring equipment in three boreholes. Total core recovery (TCR) in the bedrock was 100% in most core runs. TCR value of 70% was observed in Borehole H1, Run 1.

The RQD values recorded for seven of the eight core runs ranged from 20 to 58% indicating poor to fair rock quality. Lower RQD value of 7% was obtained in Borehole H1 Run 1, cored near elevation 186.6 m, indicating a very poor quality rock. Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 2 to greater than 10. In Run 1 and Run 2 of Borehole H6 values were generally low, except in the upper section of each run, where FI was greater than 5.

The results of Point Load tests conducted on bedrock samples indicated unconfined compressive strength (UCS) values of 44 to 105 MPa (estimated from point load tests on intact samples) indicating that the samples tested were medium strong to strong. Low UCS values of 3 MPa were observed in Borehole H1, Runs 2 and 3, which represents weak rock.

Two bedrock samples were subjected to unconfined compression tests. Results indicated that the UCS values were 9.5 MPa and 29 MPa, in samples from Borehole H4, Run 1 and Borehole H6, Run 2, respectively. UCS results for Boreholes H4 and H6 are attached in Appendix B. The samples tested will be classified as weak to medium strong rock.

It must be noted however that point load tests were possible only on less weathered shale or higher strength limestone samples as the more typical weathered shale cores tended to disintegrate under point loading.

The shale bedrock typically contains layers of limestone that can be significantly harder than the shale itself. The distribution, thickness and strength of these layers vary from location to location, and these layers typically exhibit less pronounced weathering than the shale. Thickness of hard layers generally ranges from 30 to 200 mm. Sampling and interpretation from small diameter boreholes may underestimate the frequency and strength of the strong layers and therefore geological expertise and past experience must be applied in any decision making process regarding the bedrock.

5.7 Water Levels

Upon completion of drilling, water was measured at depths of 9.1 m and 13.1 m (elevations 188.2 m and 186.7 m) in open Boreholes H1 and H2. Standpipe piezometers were installed in five boreholes to monitor water levels after completion of drilling. The water levels measured in the piezometers are summarized in Table 5.2, along with the measurements in the boreholes upon completion of drilling.

Table 5.2 – Measured Groundwater Levels

Borehole	Date	Water Level (m)		Comment
		Depth	Elevation	
H1	15-Nov-2006	9.1	188.2	In open borehole
H2	3-Nov-2006	13.1	186.7	In open borehole
	13-Nov-2006	6.0	193.8	In piezometer
H3	29-Jan-2007	1.6	192.8	In piezometer
H4	14-Nov-2006	8.1	191.9	In piezometer
H6	14-Nov-2006	7.1	192.6	In piezometer
H8	13-Nov-2006	7.7	190.3	In piezometer
	12-Dec-2006	7.1	190.9	
	29-Jan-2007	7.4	190.6	

Based on these observations, local groundwater levels exist at Elevations 190.6 m to 193.8 m.

Water levels were observed on the site during previous investigation (Reference 1) indicated that groundwater level varying from elevations 187.2 m to 190.6 m.

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

The borehole locations were staked on site and the ground surface elevations were supplied to Thurber by MMM Group Limited.

The drilling and sampling equipment was supplied and operated by DBW Drilling of Ajax Ontario. The field work was supervised on a full time basis by Mr. George Azzopardi and Mr. Stephane Loranger, C.E.T. of Thurber Engineering Ltd.

Laboratory testing was carried out at Thurber's Laboratory in Oakville and Golder's Laboratory in Mississauga.

Supervision of the field program, interpretation of the field data and preparation of the Foundation Investigation Report were conducted by Mr. A. E. Gorman, 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.

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PART 2: ENGINEERING DISCUSSION 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 and approach fills for the proposed structure.

Based on the preliminary General Arrangement (GA) drawing provided by MMM Group Limited, a three-span, steel box girder structure supported on integral abutments is proposed. The centre span will be 51 m long and the north and south spans will be 37 m and 37.5 m long, respectively. The proposed finished grade at the structure will be near Elevation 200.6 m and the finished grade of Highway 401 will be at approximate elevation 192.5 m.

The resulting Hurontario Street approach fills will be approximately 7 m and 8 m high at the north south abutments, respectively.

Fill will be excavated from the forward slopes of the existing approach fills in order to accommodate the proposed widening of Highway 401. The existing approach fills will be widened to the west to accommodate the widening of Hurontario Street.

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 STRUCTURE FOUNDATIONS

Consideration was given to various possible foundations systems, taking account of the site stratigraphy and the structure General Arrangement. The stratigraphy encountered at the site consists of topsoil or pavement structure overlying the fill of the Hurontario Street approaches underlain by native silty clay till and occasional silty sand layers. Fill thickness varies from 6.1 m to 9.4 m at the approaches, east side of piers and abutments. Locally in the west side of the piers fill thickness

ranges from 0.6 to 1.4 m. Weathered shale bedrock was contacted below the till deposits at depth varying from 2.5 m to 15.2 m (elevations 190.5 m to 184.8 m).

The following alternatives were considered for foundation types at this site:

- Spread footings on native soil
- Spread footings on shale bedrock
- Caissons (drilled shaft piles)
- Driven steel H-piles

8.1 Spread Footings on Native Soil

Spread footings can be founded on the very dense native undisturbed sand or hard silty clay till. Provided a minimum footing width of 2 m is maintained footings founded on the above recommended strata may be designed for the following values:

- Factored geotechnical resistance of 600 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 400 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 elevations for spread footings are given in Table 8.1.

Table 8.1 – Highest Permitted Founding Elevations

Foundation Element	Borehole	Footings on Soil		Footings/Caissons on Bedrock	
		Depth below existing ground surface (m)	Founding Elevation	Depth below existing ground surface (m)	Founding Elevation
North abutment	H1	7.6	189.7	9.1	188.1
	H2	9.4	190.3	13.7	186.1
Pier 1	H3	2.2	192.2	4.0	190.5
	H4	9.0	191.0	15.2	184.8
	3	5.2	190.5	7.0	188.7
	4	8.4	191.5	9.8	190.1
Pier 2	H5	-	-	-	-
	2	1.4	191.3	4.4	188.3
	5	7.9	192.1	10.7	189.3
South Abutment	H6	9.1	190.6	11.7	187.9
	1	-	-	4.4	190.3
	6	9.3	190.5	10.9	188.9

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 coefficient of friction of 0.55. 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. The engineered fill must consist of OPSS Granular “A” placed in 150 mm lifts, compacted to 100% of its SPMDD at $\pm 2\%$ 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 Shale Bedrock

Spread footings bearing on undisturbed weathered shale bedrock may be designed on the basis of a factored geotechnical resistance at ULS of 750 kPa. The SLS condition will not govern for footings on shale bedrock. The highest permissible bearing elevations for spread footings on bedrock are given in Table 8.1.

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.

Calculations of the lateral resistance of a footing poured on bedrock may be carried out using a value of 0.55 for the ultimate friction factor between the concrete and bedrock.

The bearing surface should be prepared by removing all loose material, shattered rock and rock slabs and protecting the shale from exposure to air by placing concrete or mud slab within 24 hours of completing excavation. The mud slab concrete should be the same class as the footing concrete and should take a minimum thickness of 100 mm.

The bedrock elevations shown in Table 8.1 indicate that there is a difference in elevation of the top of bedrock of approximately 1 m to 5 m across abutments, Pier 1 and Pier 2 of the proposed Hurontario Street SBL and NBL overpass. Any areas where the bedrock is below founding elevation, or where subexcavation is required to reach bedrock, the founding elevation should be re-established using concrete of the same class as the footing to compensate for difference in elevations of the top of bedrock. In this case, the geotechnical resistance of 750 kPa may still be used for footing design.

Where bedrock slopes within the foundation footprint, the foundation should be prepared by excavation of a horizontal surface in bedrock, though stepping of the footing base is permissible.

The mass concrete fill required at the edge of a footing must extend beyond the footing perimeter by sufficient distance to distribute the shear stresses from the footing and prevent stress concentrations under the edge of the footing. This condition must be checked structurally but extension of the mass concrete to 200 mm beyond the edge of the footing is typically sufficient.

8.3 Augered Caissons (Drilled Shafts)

Drilled shaft foundations are also suitable for the support of structural loads at this site. To avoid the requirement of hand cleaning and inspection of the base, the drilled shaft should be designed to achieve geotechnical resistance through shaft adhesion in the bedrock. No contribution from end bearing is assumed in this case. A value of 350 kPa at factored ULS is recommended for the shaft adhesion, reflecting the weathered condition of the upper bedrock. The SLS condition will not govern for caissons in bedrock.

The bedrock elevations below which the lengths of the caisson sockets must be measured are given in Table 8.1. The caisson should be founded on intact or tightly jointed bedrock, at a minimum depth of 1.5 m below the bedrock surface.

While caissons may be more expensive than spread footings, they have an advantage at the piers in that they can often be installed with less disruption to traffic than would be caused by a footing excavation.

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 any obstructions in the fill or cobbles and boulders in the till. The glacial till is hard at this site and augering in the fill will be laboured. It should also be noted that hard layers were encountered in the shale. Hard interbedded layers of limestone, calcareous shale or siltstone in the shale will require the use of coring or breaking equipment in addition to the auger equipment. The contract documents must contain a statement to alert bidders of these facts. The suggested wording for an NSSP addressing this issue is included in Appendix F.

Caissons at this site may encounter a sloping bedrock surface. The contract documents must contain a warning to alert bidders to the possibility of installation into a sloping bedrock surface. The first observed contact with bedrock will typically be on the “high” side of the rock surface. To provide for adequate embedment of the caisson into bedrock, the socket depth must be measured from the “low” side contact where sloping bedrock surface is encountered.

The resistance value provided above is based on the assumption that the walls and base of each caisson are cleaned of loose material prior to placement of concrete using air jet and vacuum suction equipment. 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 rock socket 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 Steel Piles

The native very dense/hard till soils and the shale bedrock encountered at this site are considered suitable for the support of foundations on driven steel H-piles. However, piles are only considered feasible for perched abutments. In the case of closed abutments or pier foundations, it is anticipated that the underside of the pile cap would be at an elevation that would result in very short piles, unless the piles were pre-augered into the shale.

In some cases, driven piles are expected to achieve resistance in the very dense or hard silty clay till. In other cases, the piles are expected to fully penetrate the overburden and achieve resistance in the weathered shale bedrock. Since the upper levels of the shale are weathered and will behave essentially as a soil, piling recommendations are provided on the basis of resistance being achieved in very dense/hard soil.

The elevations at which the piles are expected to develop resistance are given in Table 8.2.

Table 8.2 – Estimated Pile Tip Elevation

Foundation Element	Borehole	Anticipated Pile Tip Elevation To Develop Resistance
North abutment	H1	188.7
	H2	189.3
South Abutment	H6	188.6
	1	190.3
	6	189.5

The recommended minimum length of pile below the underside of the abutment is 6 m. In the case of an integral abutment design, the minimum length will consist of 3 m in loose sand (600 mm diameter CSP filled with loose sand) and a minimum of 3 m driven into the resisting material below. In some situations it may not be possible to drive a 6 m pile due to the presence of bedrock at shallow depth. For these cases, please refer to Section 9 of this report.

8.4.1 Axial Resistance

The factored, vertical, axial, geotechnical resistances for two typical pile sections when driven into very dense/hard till or to top of weathered bedrock are presented in Table 8.3

Table 8.3 – Axial Resistance of Various Pile Sections

Pile Section	Piles Driven Into Clay Till, Sand or Shale
	ULS (Factored) (kN)
HP 310 X 110	1,800
HP 360 x 132	2,400

The SLS condition will not govern for piles founded on hard or very dense soils or shale bedrock.

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

The pile tip elevations shown in Table 8.2 should be used for cost estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.4.4 Pile Driving.

8.4.2 Pile Tips

The tips of all piles must be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

The use of rock points is recommended for the following reasons:

- The piles will be driven into glacial till soil that may contain cobbles and boulders, which requires a higher level of protection than driving into soils containing only smaller particle sizes.
- Some piles may achieve resistance in the bedrock.

In the case of partial bearing on bedrock, the cast steel point will provide better stress redistribution without failure than would be achieved in a pile tip reinforced with a driving shoe.

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 or rubble within the fill.
- The possibility of piles within a group achieving the specified resistance at different elevations.

Suggested wording for the NSSP is included in Appendix F.

8.4.4 Pile Driving

Pile driving must be controlled by the Hiley Formula with an ultimate pile resistance as specified below and in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile.

“R” must have the minimum values shown in Table 8.5.

Table 8.5 – Ultimate Geotechnical Resistance of Piles

Pile	Ultimate Resistance (R) (kN)
HP 310 X 110	3,600
HP 360 x 132	4,800

The NSSP referenced in 8.4.3 should also require the QVE to terminate driving before the pile is damaged by overdriving.

To facilitate pile installation, new embankment fill through which piles will 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, subject to recommendations provided in Section 9.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. The upper 3 m of the pile will lie within the stiff to very stiff approach embankment fill or the underlying hard till. 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 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 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:

$$k_s = 67 \cdot S_u / D \quad (\text{kN/m}^3)$$

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

where

$$D = \text{pile width in metres}$$

$$S_u = \text{undrained shear strength (kPa)}$$

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/m³), 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 150 kN at ULS and 50 kN at SLS.

Table 8.7 – Parameters for Lateral Pile Resistance

Location	Elevation	K_p	S_u kPa	Unit Weight* (kN/m ³)	Soil Conditions
South Abutment	OGL to 192.5	3.0	100	20	Silty clay fill (some sand and gravel at surface)
	192.5 to 191.1	3.0	100	10	Silty clay fill
	191.1 to 187.9 (BDR)	3.3	200	10	Hard clay till
North Abutment	OGL to 194.0	3.0	100	20	Silty clay fill (some sand and gravel at surface)
	194.0 to 190.0	3.0	100	10	Silty clay fill
	190.0 to 186.0 (BDR)	3.3	200	10	Hard clay till (Some sand seams)

*Buoyant unit weight below the water table.

The modulus of subgrade reaction may have to be reduced, based on the 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.

In the case of conventional abutments, i.e. non-integral, horizontal loads may be resisted by means of battered piles.

8.5 Recommended Foundation

From a foundations technical, constructability and cost-effectiveness perspective, the recommended foundations at this site are:

- Abutments - steel H-piles driven to achieve resistance in the very dense or hard native soil or on the shale bedrock
- Piers - Augered caissons (drilled shaft piles) socketed into the shale bedrock.

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

8.6 Frost Cover

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

9 CONSTRUCTION OF INTEGRAL ABUTMENTS IN SHALLOW BEDROCK

The stratigraphy encountered at some locations near the Piers and Abutments consists of fill overlying native silty clay till underlain by shallow bedrock. In general, the proposed Highway 401 grade lies 2.1 m to 7.7 m above the top of bedrock. The proposed Highway 401 EBL grade lies 2.2 m to 4.5 m above the top of bedrock at the south abutment. Based on these subsurface conditions, the south abutment, in particular, is not considered suitable for driven piles and consequently not suitable for integral abutment. However, it is recognized that integral abutment bridges, which require pile foundations, offer significant long term advantages. Therefore the installation procedures outlined below are recommended for the use of integral abutments at the south abutment.

The recommended minimum pile length below the abutment is 6 m, consisting of 3 m in loose sand and a minimum of 3 m driven into resisting material below. An integral abutment structure may be designed at this site if the foundation area is prepared as follows:

- Excavate to a depth of 5.0 m below the underside of the abutment stem. The base of the excavation must be 2.6m \pm wide and be level and the sides of the excavation must be sloped at 45° to the horizontal.
- At each pile location, a suitable diameter socket must be drilled to a depth of 1.2 m (a 600 mm diameter hole is appropriate for a HP 310X110 pile). The bottom of the socket must be level and clean.
- The piles must be placed in their respective sockets, in full face contact with the rock and supported in a vertical position (or as required by the design).
- The socket must then be filled with concrete. The concrete may be of the same class as specified for foundations, but typically 30 MPa.
- Backfill the excavation and complete construction of engineered fill up to a level 3 m below the underside of the abutment using OPSS Granular “A” compacted in accordance with OPSS 501.
- Continue with normal procedures for integral abutment construction.

Relatively short piles are also anticipated at the north abutment, where the bedrock surface lies between 4.3 and 6.4 m below ground surface. If short piles cannot provide sufficient resistance to horizontal loading, it is recommended that the piles be socketed into rock for a distance of 1.2 m with the rock socket backfilled with concrete.

Construction of sockets will require mobilization of a separate rig and will add significantly to the cost of the foundation. Foundation details for construction are shown in Figure 1.

10 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the soils within the probable depth of excavation at this site may be classed as Type 3 for fill and Type 2 for native soils (silty clay till and silty sand).

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.

If seepage into an excavation is experienced, dewatering using sumps and pumps is considered feasible. The possibility exists that additional pumps may be required if localized zones of perched water are encountered in the fill, or if concentrated seepage is experienced from seams or fractures in the shale bedrock.

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

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

The design of roadway protection should be the responsibility of the Contractor. However, one option that is considered to be suitable for use as temporary shoring at this site is a soldier pile and lagging wall. Due to the presence of shallow bedrock under Highway 401, the soldier piles may have to be installed through pre-drilled holes and socketted into bedrock in order to develop the required toe resistance.

A temporary soldier pile and lagging wall may be designed using the parameters given below.

γ	=	20 kN/m ³
γ_w	=	10 kN/m ³
K_a	=	0.35 (road embankment fill)
	=	0.33 (sandy silt to silty sand)
	=	0.33 (silty clay till)
K_p	=	3.0 (road embankment fill)
	=	3.0 (silty clay till)

For rock sockets formed within the shale bedrock, the ultimate passive force that can be mobilized by the embedded portion of a pile is given by :

$$P_p = 6 \cdot c \cdot D \cdot L$$

where c = 300 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)

D = diameter of socket, m

L = depth of socket in rock, m

The designer of the roadway protection system should check whether the socket is sufficiently deep to provide base fixity.

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system. All shoring systems should be designed by a Professional Engineer experienced in such designs.

12 UNWATERING

Piezometers installed in boreholes revealed that groundwater level is near 1.6 m to 8.1 m depth (elevations 193.8 m to 190.6 m) although perched water may be encountered at higher levels within the overlying fill.

The Contractor must make provision to control the groundwater seepage and use sump pumps to pump any accumulation of water from the footing base prior to placing concrete. This is critical for footings founded on shale, since shale will soften and deteriorate if water is allowed to accumulate on the proposed footing base. Unwatering must remain operational and effective until the footing is constructed and backfilled.

Based on the preliminary GA for the underpass structure indicating the use of a piled foundation, it is expected that work at the abutments will not require excavation below the groundwater level. If spread footings are the selected design for pier foundations, water may be encountered at pier locations during excavation. The Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation. Placement of concrete must be done in the dry.

The design of the unwatering system that may be required must remain the responsibility of the Contractor.

13 APPROACH EMBANKMENTS

The widened approach embankments for this structure will be constructed on either native hard clay till deposits or shale bedrock and will butt against the existing approach fill. These foundation soils will satisfactorily support the approach fills at this site, which are expected to be approximately 7 m and 8 m high at the north and south abutments, respectively. The foundation materials will provide adequate stability of earth fill embankments constructed with side slopes of 2H:1V.

Considering the embankment height and consistency/relative density of the foundation soils, settlement induced in the foundation soils due to embankment loading will be less than 20 mm. The new embankment will also experience settlement resulting from consolidation of the fill. This settlement in the fill is expected to range between 30 mm and 40 mm in a 8 m high embankment. The settlement will be relatively rapid and essentially complete within three months of when construction of the fill is completed. Accordingly, embankment construction three months in advance of road paving is recommended as a good construction practice to minimize any post construction time-dependent settlement due to consolidation in the embankment fill itself.

All topsoil and organic soils should be stripped from the footprint of the new approach fills. Particular attention should be paid to removing all softened material from existing ditches that fall within the footprint of the new embankment. The existing fill slopes must be benched in accordance with OPS 208.010 prior to placing new fill against the existing embankment. All topsoil, organics or

other deleterious materials should be stripped from the existing fill slopes prior to benching and placement of new embankment fill.

Embankments constructed using granular material and most inorganic earth materials will have stable side slopes at inclinations no steeper than 2H:1V. Approach embankments should be placed and compacted in accordance with OPSS 501. Excavated shale bedrock should not be used for new fill construction.

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

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

Where earth fill embankments are higher than 8 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 8 m
- be at least 2 m wide
- have 2% positive drainage to shed run-off water.

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

14 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material. In the case of a conventional abutment, Granular A or Granular B Type II backfill is recommended. 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.

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

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with SP 105S01.

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 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)

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.

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 $\phi = 35^\circ$		OPSS Granular B Type II $\phi = 35^\circ$		OPSS Granular B Type I $\phi = 32^\circ$	
	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	0.48*
At rest (Restrained Wall)	0.43	-	0.43	-	0.47	-
Passive	3.7	-	3.7	-	3.30	-

* For wing walls.

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

16 RETAINED SOIL SYSTEMS

It is understood that Retained Soil System (RSS) walls are proposed to support the north and south approaches. The borehole information indicates that the foundation conditions at the wall locations are comprised of approximately 6.1 to 8.5 m of firm to hard silty clay fill or loose to dense sand fill overlying very stiff to hard silty clay till or weathered shale bedrock.

Retained soil system (RSS) walls may be used subject to the requirements presented in this section. RSS walls should be specified to be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

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. 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.

To provide an acceptable foundation performance, the RSS mass must be founded on the native very stiff to hard silty clay till at or below elevations 192.0 m and 190.5 m in the north and south approach, respectively. A wall founded on this material should be designed for a factored bearing resistance of 600 kPa at ULS and a bearing resistance of 300 kPa at SLS.

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

The geotechnical resistance provided above is for concentric, vertical loading of a strip footing. The effects of footing shape, load inclination and eccentricity need to be taken into account according to the CHBDC 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 or native silty clay till may be estimated using an ultimate friction coefficient of 0.55.

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

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.

If a RSS wall system is selected, the global stability must be analyzed after the location of the wall is known. The global stability of the RSS wall is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment. Typically, however a RSS wall founded on the very stiff to hard till at this site will possess acceptable global stability.

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 0.04

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 potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method¹.

Using this method, 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.

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

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

Table 16.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading				
Wall Condition	Granular A or Granular B Type II $\phi = 35^\circ$; $\delta = 17.5^\circ$ $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$; $\delta = 16^\circ$ $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.3	0.45	0.33	0.54
Passive (K_{PE})	6.3	6.3	5.4	5.4
At Rest (K_{OE})**	0.59		0.63	

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

** After Woods

18 IMPACT ON ADJACENT STRUCTURES

The Contract must include constraints to limit the impact of the Contractor's work on the existing Hurontario Street and Highway 401 underpass and any other adjacent structures.

These constraints should include, among others:

- Protection of the active Highway 401 and Hurontario Street during excavation at the north and south abutments.
- Continuous monitoring of the existing structures should be conducted during construction to detect any indication of movement. The structural designers must determine the acceptable limits of movement in the existing structure and incorporate the requirements in the contract.

Suggested wording for an NSSP addressing the above issues is included in Appendix F.

19 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not limited to:

- The possibility of piles reaching refusal on boulders or bedrock. In this case, the Hiley formula is not appropriate and site staff must make a decision regarding pile resistance and the appropriateness of continued driving.
- Care must be taken during footing excavation to avoid disturbing and undermining the nearby existing structure foundations and travelled lanes of the roadways.

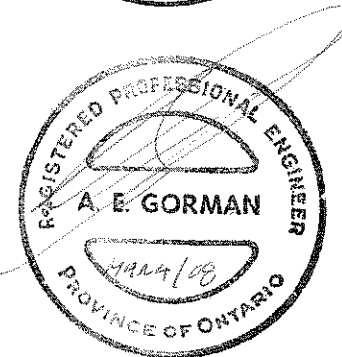
20 CLOSURE

Engineering analysis and preparation of the Foundation Design Report was conducted by Mr. A. E. Gorman, P. Eng. and Ms. R. Palomeque Reyna, P.Eng. The report was reviewed by P. K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Record of Borehole Sheets

(present investigation)

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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

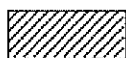

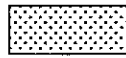


 Water Level
 C_{pen} Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	(psi)	Field Estimation of Hardness*
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
TERMS					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
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.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No H1

1 OF 2

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION

Proposed Hurontario St. Underpass N 4 832 263.1 E 289 657.2

ORIGINATED BY GA

HWY 401

BOREHOLE TYPE

Solid Stem Augers/NQ Coring

COMPILED BY JHL

DATUM Geodetic

DATE

2006-11-15 - 2006-11-15

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
197.3	TOPSOIL: (75 mm)													
0.0	Silty CLAY, some sand, trace gravel		1	SS	10		197							
0.1	Firm to Stiff													
	Reddish Brown		2	SS	15		196							
	(FILL)													
	occasional shale fragments		3	SS	13		195							
	Becoming Brown to Grey		4	SS	7		194							5 32 47 16
			5	SS	5		193							
			6	SS	10		192							
	occasional rootlets		7	SS	16		191							0 20 53 27
	Very Stiff						190							
189.7	Silty SAND, fine grained, trace to		8	SS	50/		189							0 50 50
7.6	some clay				150									(SI+CL)
	Very Dense													
	Brown													
	Moist to Wet													
188.1	SHALE, highly weathered, thinly		9	SS	85		188							
9.1	bedded, reddish brown, frequent rubble													
	zones													

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No H1

2 OF 2

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION

Proposed Hurontario St. Underpass N 4 832 263.1 E 289 857.2

ORIGINATED BY GA

HWY 401

BOREHOLE TYPE

Solid Stem Augers/NQ Coring

COMPILED BY JHL

DATUM Geodetic

DATE

2006-11-15 - 2006-11-15

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page													
	SHALE, highly weathered, thinly bedded, reddish brown, frequent rubble zones		10	SS	50/									
					.125									
	Rubble zones from 11.92m to 11.97m, 12.07m to 12.22m Limestone interbeds at 12.19m to 12.34m Moderately weathered Limestone interbeds at 12.47m to 12.58m, 13.16m to 13.21m, 13.21m to 13.24m, 13.31m to 13.36m, 13.41m to 13.49m, 13.53m to 13.59m		1	RUN										
	Rubble zones from 13.31m to 13.41m, 13.46m to 13.51m		2	RUN										
	Slightly weathered Limestone interbeds at 14.07m to 14.10m, 14.56m to 14.58m, 14.63m to 14.68m		3	RUN										
182.5														
14.8	END OF BOREHOLE AT 14.78 m. BOREHOLE OPEN AND WATER LEVEL AT 9.10 m UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.													

+³, X³: Numbers refer to Sensitivity

20
15
10
(%) STRAIN AT FAILURE

METRIC

ORIGINATED BY GA/JHL

COMPILED BY JHL

CHECKED BY RPR

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No H2

2 OF 2

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION

Proposed Hurontario St. Underpass N 4 832 276.4 E 289 882.0

ORIGINATED BY GA/JHL

HWY 401

BOREHOLE TYPE

Solid Stem Augers

COMPILED BY JHL

DATUM Geodetic

DATE

2006-10-30 - 2006-11-03

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE				WATER CONTENT (%) W _P W W _L				
	Continued From Previous Page															
	Silty CLAY, some sand, trace gravel, occasional siltstone and limestone fragments Hard Grey (TILL)		10	SS	50/ .125		189									7 43 36 14
			11	SS	50/ .125		188									
							187									
186.1																
13.7	SHALE, highly weathered, thinly bedded, reddish brown, grey limestone layers		12	SS	126		186									
							185									
184.5			13	SS	100/ .075											
15.3	END OF BOREHOLE AT 15.32 m. BOREHOLE OPEN AND WATER LEVEL AT 13.1 m UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 13.11.06 6.0 193.8															

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

RECORD OF BOREHOLE No H3

1 OF 1

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION Proposed Hurontario St. Underpass N 4 832 235.7 E 289 881.3

ORIGINATED BY GA

HWY 401

BOREHOLE TYPE Solid Stem Augers

COMPILED BY JHL

DATUM Geodetic

DATE 2006-11-12 - 2006-11-12

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
194.4	TOPSOIL: (50 mm)		1	SS	8		194							
193.8	Sandy SILT, trace clay, trace gravel Brown Damp (FILL)		2	SS	34		193							
190.6	Silty CLAY, some sand, trace gravel Hard Brown (TILL)		3	SS	48		192							3 33 44 20
			4	SS	50/ .125		191							
			5	SS	50/ .075		190							2 44 45 9
190.5	SHALE, highly weathered, thinly bedded, reddish brown, grey limestone layers		6	SS	50/ .100									
189.9	END OF BOREHOLE AT 4.57 m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.													
4.6	WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 29.01.07 1.6 192.8													

+ 3 . X 3 Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No H4

1 OF 3

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION

Proposed Hurontario St. Underpass N 4 832 249.1 E 289 906.0

ORIGINATED BY JHL

HWY 401

BOREHOLE TYPE

Solid Stem Augers/INQ Coring

COMPILED BY JHL

DATUM Geodetic

DATE

2006-10-31 - 2006-11-01

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	
200.0												
0.0	TOPSOIL: (100 mm)											
0.1	SAND, trace silt, trace to some gravel, occasional shale fragments Compact Brown Moist (FILL)		1	SS	28		200					
			2	SS	29		199					
			3	SS	13		198					
	occasional cobbles		4	SS	13							
197.0												
3.0	SAND AND GRAVEL, trace silt, trace clay, occasional cobbles Compact Brown Moist (FILL)		5	SS	14		197					37 54 9 (SI+CL)
			6	SS			196					
							195					
193.9												
6.1	Silty SAND, some clay, trace gravel, occasional cobbles Very Loose Brown Wet (FILL)		7	SS	0		194					4 47 33 16
							193					
192.3												
7.8	Silty CLAY, some sand, trace gravel, trace rootlets, occasional shale fragments, occasional oxide staining Very Stiff to Hard Grey (TILL)		8	SS	22		192					
			9	SS	63		191					1 35 42 22

Continued Next Page

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

METRIC

ORIGINATED BY JHL

COMPILED BY JHL

CHECKED BY RPR

Continued Next Page

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No H4

3 OF 3

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION

Proposed Hurontario St. Underpass N 4 832 249.1 E 289 906.0

ORIGINATED BY JHL

HWY 401

BOREHOLE TYPE

Solid Stem Augers/NO Coring

COMPILED BY JHL

DATUM Geodetic

DATE

2006-10-31 - 2006-11-01

CHECKED BY RPR

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20						40	60	80	100	W P	W	W L
	Continued From Previous Page																			
	END OF BOREHOLE AT 19.81 m. BOREHOLE GROUTED WITH BENTONITE. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 14.11.06 8.1 191.9																			

METRIC

CHECKED BY RPR

+3, X3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No H6

1 OF 2

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION

Proposed Hurontario St. Underpass N 4 832 190.6 E 289 977.1

ORIGINATED BY GA

HWY 401

BOREHOLE TYPE

Solid Stem Augers/NQ Coring

COMPILED BY JHL

DATUM Geodetic

DATE

2006-11-06 - 2006-11-07

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
199.7 0.0	ASPHALT: (150 mm)												
0.2	SAND AND GRAVEL Dense Brown Moist (FILL)		1	SS	32								
198.9 0.8	Sandy SILT, trace gravel, occasional shale fragments Dense Brown (FILL)		2	SS	32								
198.1 1.5	Silty CLAY, some sand to sandy, trace gravel Stiff to Very Stiff Reddish Brown (FILL)		3	SS	12								
			4	SS	16								
			5	SS	13								
			6	SS	10								
			7	SS	15								
			8	SS	45								
191.1 8.5	Silty CLAY, some sand, trace gravel Hard Brown (TILL)		9	SS	57								
	occasional asphalt fragments Hard												

Continued Next Page

+ 3 X 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No H6

2 OF 2

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION Proposed Hurontario St. Underpass N 4 832 190.6 E 289 977.1

ORIGINATED BY GA

HWY 401

BOREHOLE TYPE Solid Stem Augers/NQ Coring

COMPILED BY JHL

DATUM Geodetic

DATE 2006-11-06 - 2006-11-07

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
	Continued From Previous Page							20 40 60 80 100							
	Silty CLAY, some sand, trace gravel Hard Brown (TILL)		10	SS	144		189								
187.9							188								
11.7	SHALE, highly to moderately weathered, thinly bedded, reddish brown, grey limestone layers		11	SS	106										
							187								
			12	SS	100/ .125		186								
	Rubble zones from 14.10m to 14.14m, 14.53m to 14.56m														
	Limestone interbeds at 14.51m to 14.56m, 14.68m to 14.71m, 14.75m to 14.80m, 14.99m to 15.04m		1	RUN			185								
	Limestone interbeds at 15.73m to 15.80m, 16.00m to 16.05m		2	RUN			184								
182.8							183								
16.8	END OF BOREHOLE AT 16.84 m. BOREHOLE OPEN TO BOTTOM UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 14.11.06 7.1 192.6														

ONTMT4S 2311.GPJ 3/4/08

+³ × 3³ Numbers refer to
Sensitivity

20
15 10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No H7

1 OF 1

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION

Proposed Hurontario St. Underpass N 4 832 287.0 E 289 859.4

ORIGINATED BY JHL

HWY 401

BOREHOLE TYPE Solid Stem Augers

COMPILED BY JHL

DATUM Geodetic

DATE

2006-10-31 - 2006-10-31

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
199.2								20	40	60	80	100				
0.0	ASPHALT: (100 mm)															
0.1	SAND, trace silt Very Dense Brown Moist (FILL)		1	SS	56		199									
198.3			2	SS	56											
0.9	Silty CLAY, some sand, trace gravel, occasional shale fragments Firm to Hard Brown (FILL)		3	SS	18		198									
			4	SS	21		197									
	occasional rock fragments Becoming Reddish Brown to Brown		5	SS	8		196									2 38 40 20
			6	SS	11		195									
							194									
193.1			7	SS	14		193									1 18 44 37
6.1	Silty CLAY, some sand, trace gravel Stiff to Very Stiff Grey (TILL)						192									
			8	SS	28											
191.1																
8.1	END OF BOREHOLE AT 8.08 m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE. WATER NOT OBSERVED IN BOREHOLE UPON COMPLETION OF DRILLING.															

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No H8

1 OF 2

METRIC

G.W.P. 2149-01-00 & 2150-01-00

LOCATION

Proposed Hurontario St. Underpass N 4 832 133.0 E 289 989.3

ORIGINATED BY GA

HWY 401

BOREHOLE TYPE

Solid Stem Augers

COMPILED BY JHL

DATUM Geodetic

DATE

2006-11-09 - 2006-11-09

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)					
								20	40						60	80	100	20	40	60
								○ UNCONFINED	+						FIELD VANE	×	LAB VANE			
198.0																				
0.0	TOPSOIL: (225 mm)																			
0.2	Silty CLAY, some sand, some asphalt fragments, organics and rootlets Firm to Very Stiff Brown to Reddish Brown (FILL)		1	SS	8															
			2	SS	7															
			3	SS	15															
			4	SS	14															
			5	SS	10															
			6	SS	11															
			7	SS	20															
191.0																				
7.0	Silty CLAY, some sand, trace gravel Hard Brown (TILL)		8	SS	42									0 27 46 27						
189.2																				
8.8	SHALE, highly weathered, thinly bedded, reddish brown, grey limestone layers		9	SS	106									6 21 46 27						
188.6																				
9.4	END OF BOREHOLE AT 9.45 m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.																			

Continued Next Page

+ 3 x 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No H8

2 OF 2

METRIC

G.W.P. 2149-01-00 & 2150-01-00 LOCATION Proposed Hurontario St. Underpass N 4 832 133.0 E 289 989.3 ORIGINATED BY GA
 HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY JHL
 DATUM Geodetic DATE 2006-11-09 - 2006-11-09 CHECKED BY RPR

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20					
	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) 13.11.06 7.7 190.3 12.12.06 7.1 190.9 29.01.07 7.4 190.6												

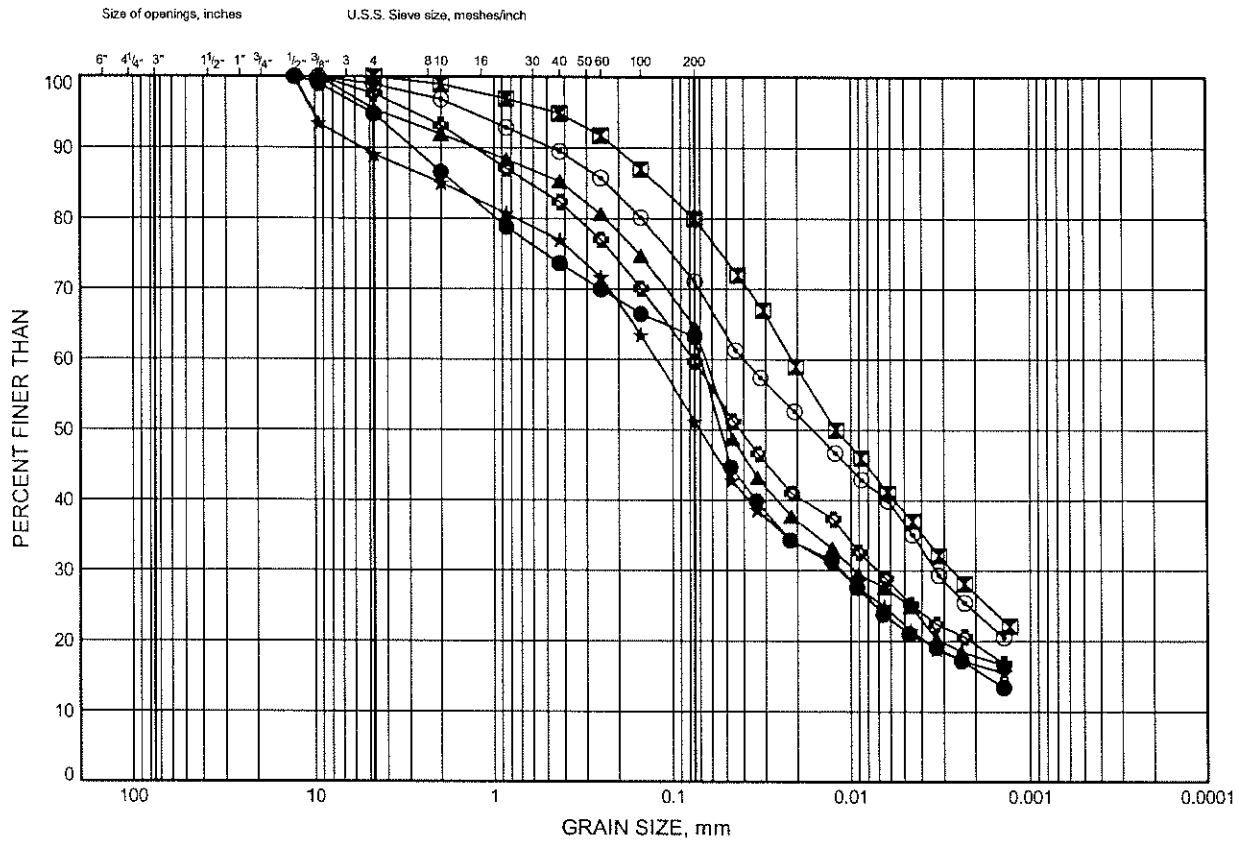
Appendix B

Laboratory Test Results

Hwy 401/410 to Credit River GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY / CLAYEY SILT FILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	H1	2.59	194.70
⊠	H1	6.40	190.89
▲	H2	1.75	198.01
★	H2	6.32	193.44
⊙	H6	2.59	197.08
⊛	H7	3.28	195.90

Date February 2008
Project 2149-01-00 & 2150-01-00

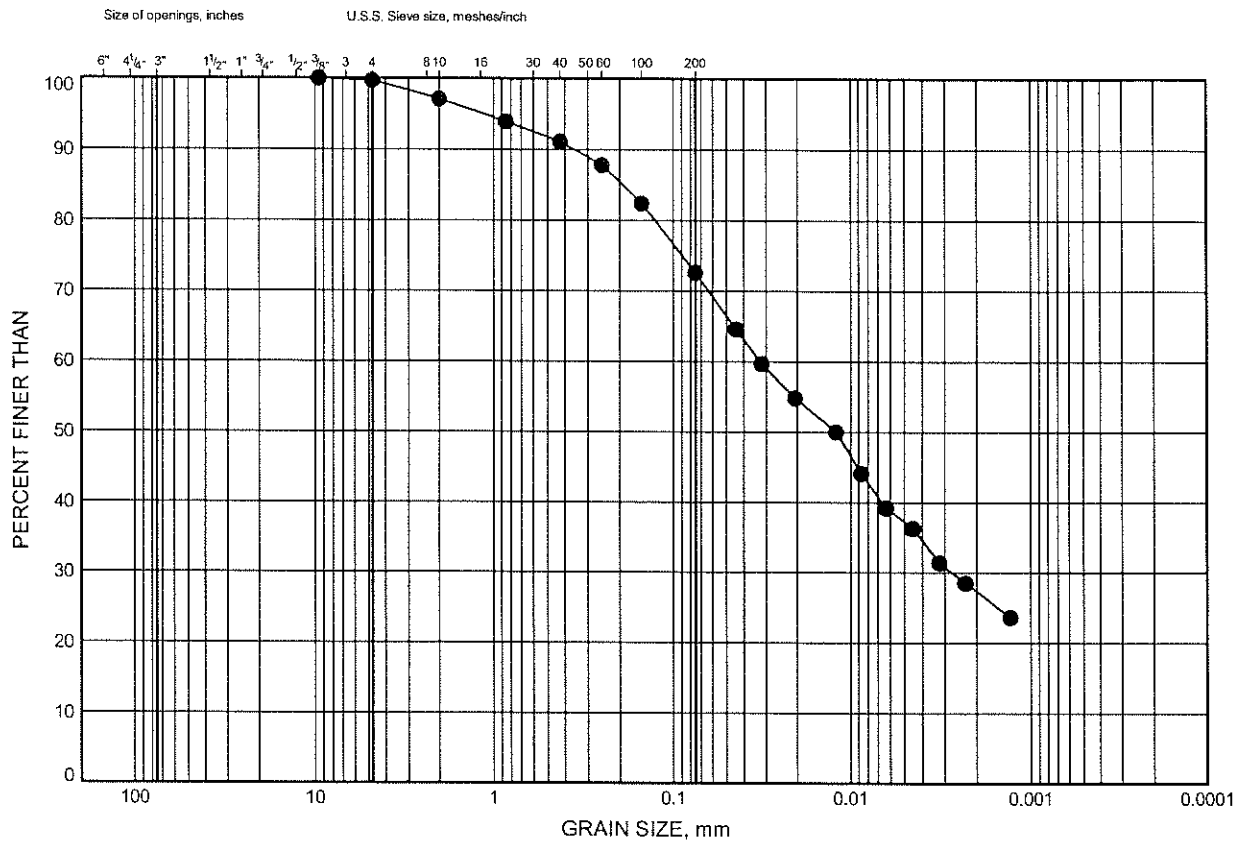


Prep'd MFA
Chkd. RPR

Hwy 401/410 to Credit River
GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY / CLAYEY SILT FILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	H8	4.88	193.15



Date February 2008

Project 2149-01-00 & 2150-01-00

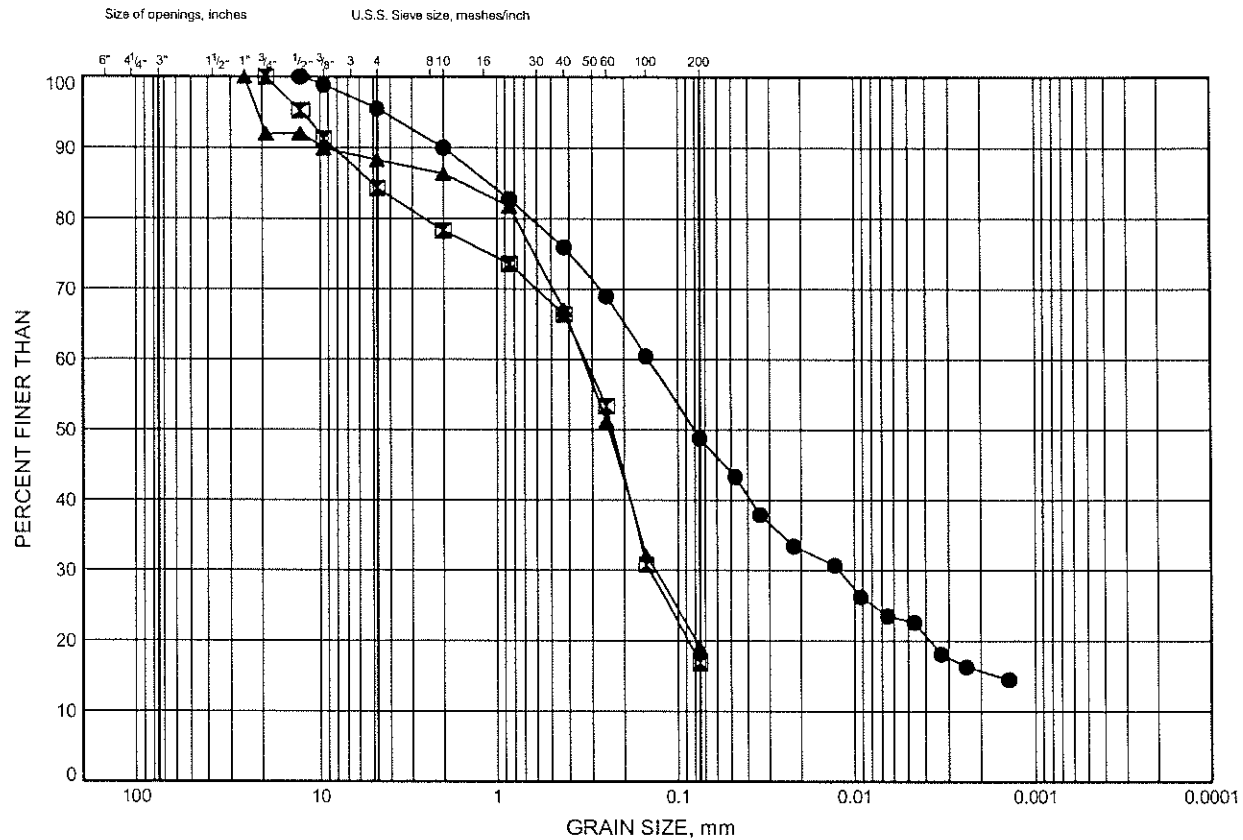
Prep'd MFA

Chkd. RPR

Hwy 401/410 to Credit River GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND, SILTY SAND FILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	H4	6.32	193.72
⊠	H5	2.59	197.54
▲	H5	5.79	194.34

Date February 2008

Project 2149-01-00 & 2150-01-00



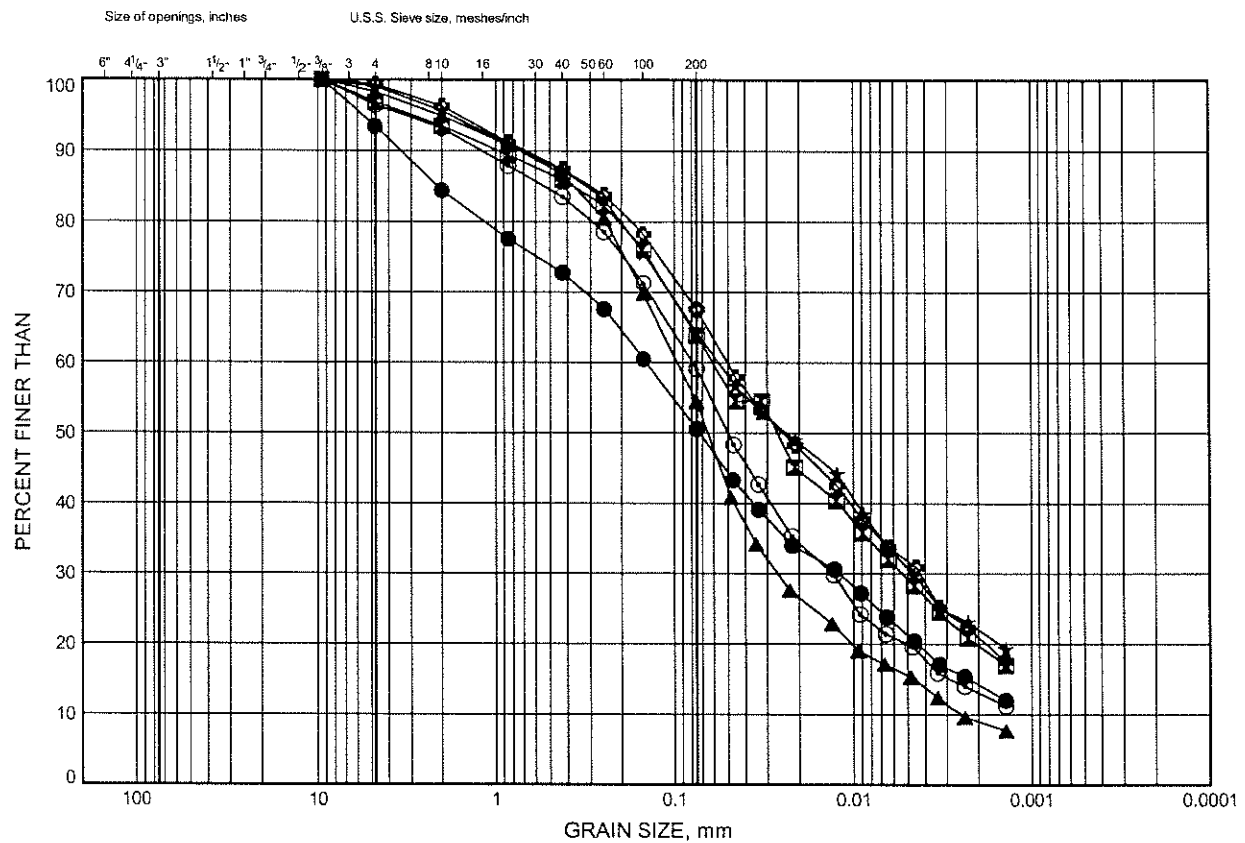
Prep'd MFA

Chkd. RPR

Hwy 401/410 to Credit River GRAIN SIZE DISTRIBUTION

FIGURE B5

SILTY CLAY TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	H2	10.73	189.04
⊠	H3	1.83	192.61
▲	H3	3.16	191.28
★	H4	9.37	190.67
⊙	H4	10.77	189.28
⬢	H6	9.45	190.22

Date February 2008.....

Project 2149-01-00 & 2150-01-00



Prep'd MFA.....

Chkd. RPR.....

FIGURE B6

Size of openings, inches

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

PERCENT FINER THAN

GRAIN SIZE, mm

Grain Size (mm)	Percent Finer Than (Solid Circles)	Percent Finer Than (Open Squares)
100	100	100
75	100	100
60	100	100
48	100	100
40	100	100
30	100	100
25	100	100
20	100	100
16	100	100
12	100	100
10	100	100
8	100	100
6	100	100
5	100	100
4	100	100
3	100	100
2.5	100	100
2	100	100
1.5	100	100
1.25	100	100
1	98	95
0.85	95	92
0.75	92	88
0.6	88	85
0.5	85	82
0.425	82	78
0.35	78	75
0.3	75	72
0.25	72	68
0.2	68	65
0.15	65	62
0.125	62	58
0.106	58	55
0.09	55	52
0.075	52	48
0.063	48	45
0.053	45	42
0.045	42	38
0.0375	38	35
0.0315	35	32
0.025	32	28
0.02	30	25

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	H7	6.32	192.85
☒	H8	7.92	190.10

Date February 2008
Project 2149-01-00 & 2150-01-00

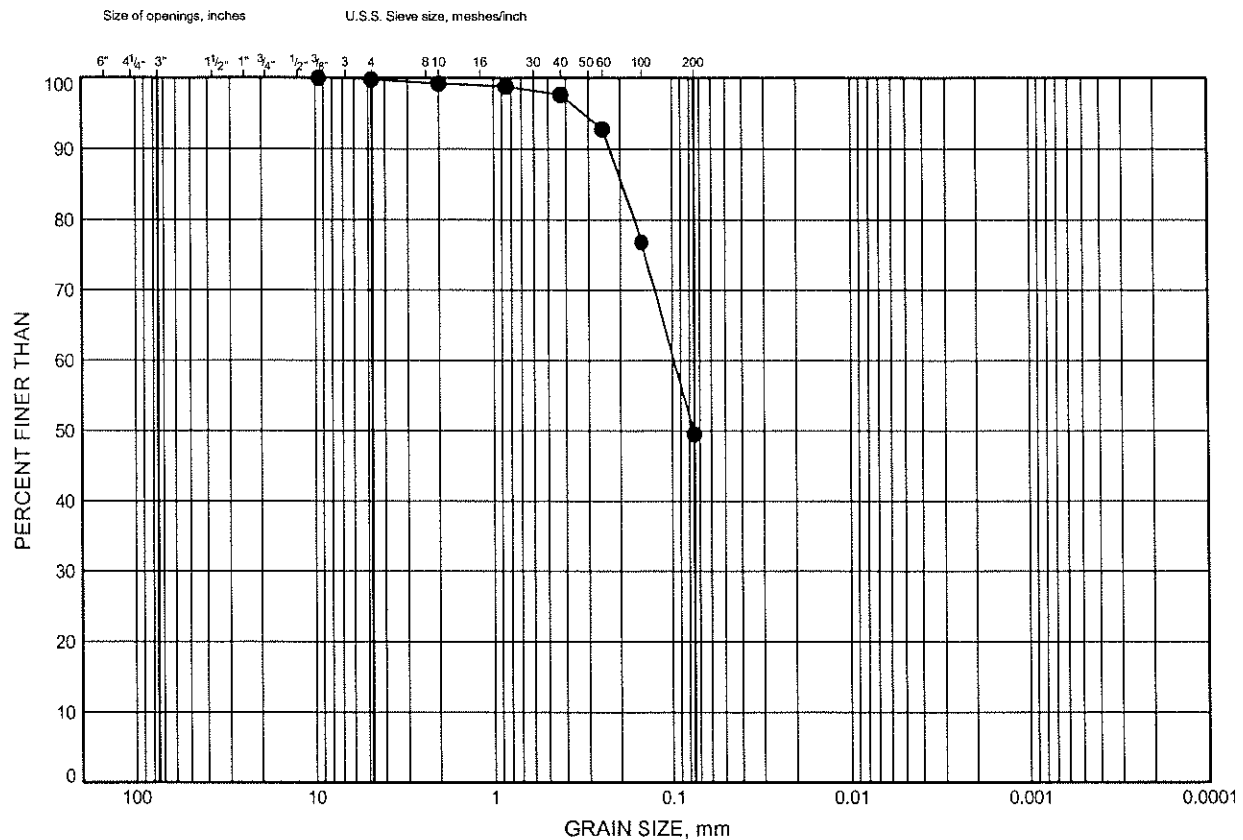


Prep'd MFA
Chkd. RPR

Hwy 401/410 to Credit River GRAIN SIZE DISTRIBUTION

FIGURE B7

SILTY SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	H1	7.92	189.36

Date February 2008

Project 2149-01-00 & 2150-01-00



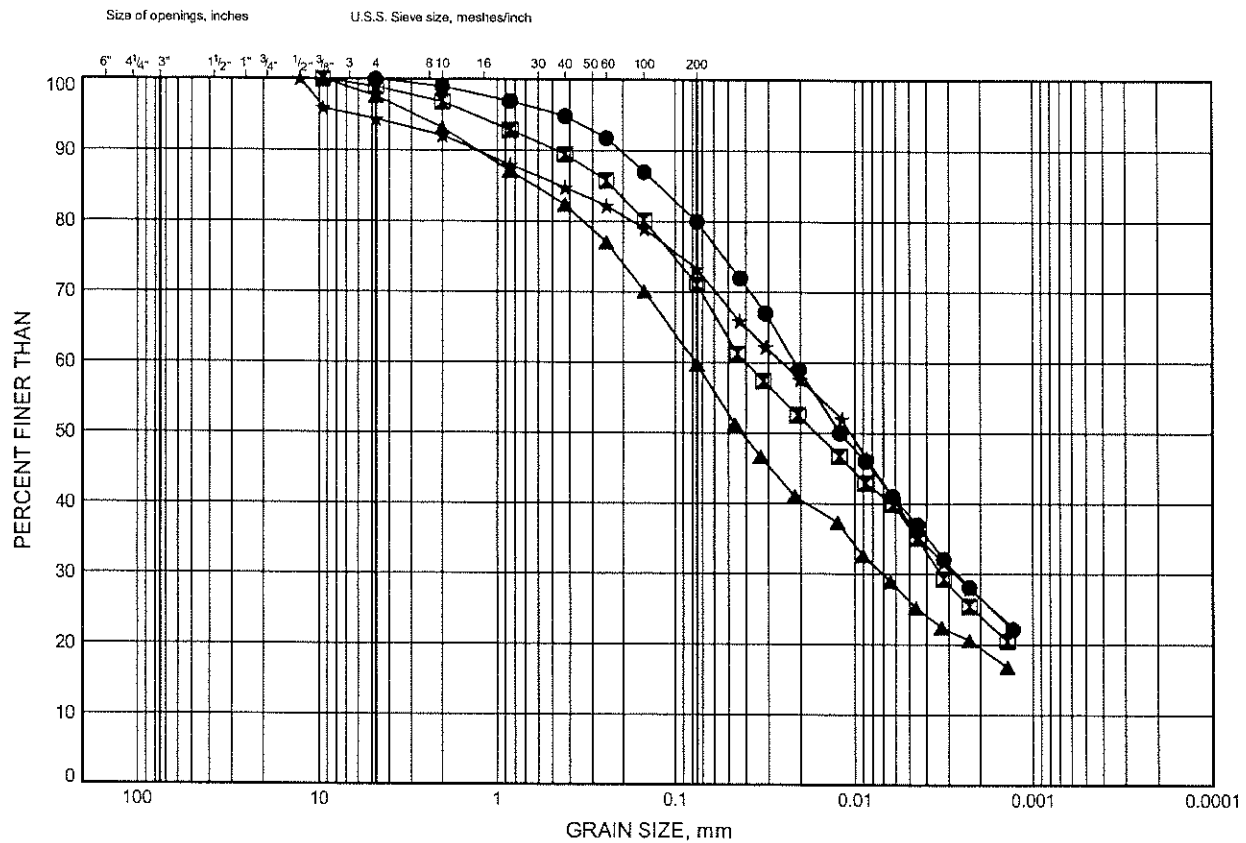
Prep'd MFA

Chkd. RPR

Hwy 401/410 to Credit River
GRAIN SIZE DISTRIBUTION

FIGURE B8

SILTY CLAY FILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	H1	6.40	190.89
⊠	H6	2.59	197.08
▲	H7	3.28	195.90
★	H8	7.92	190.10

Date February 2008
 Project 2149-01-00 & 2150-01-00

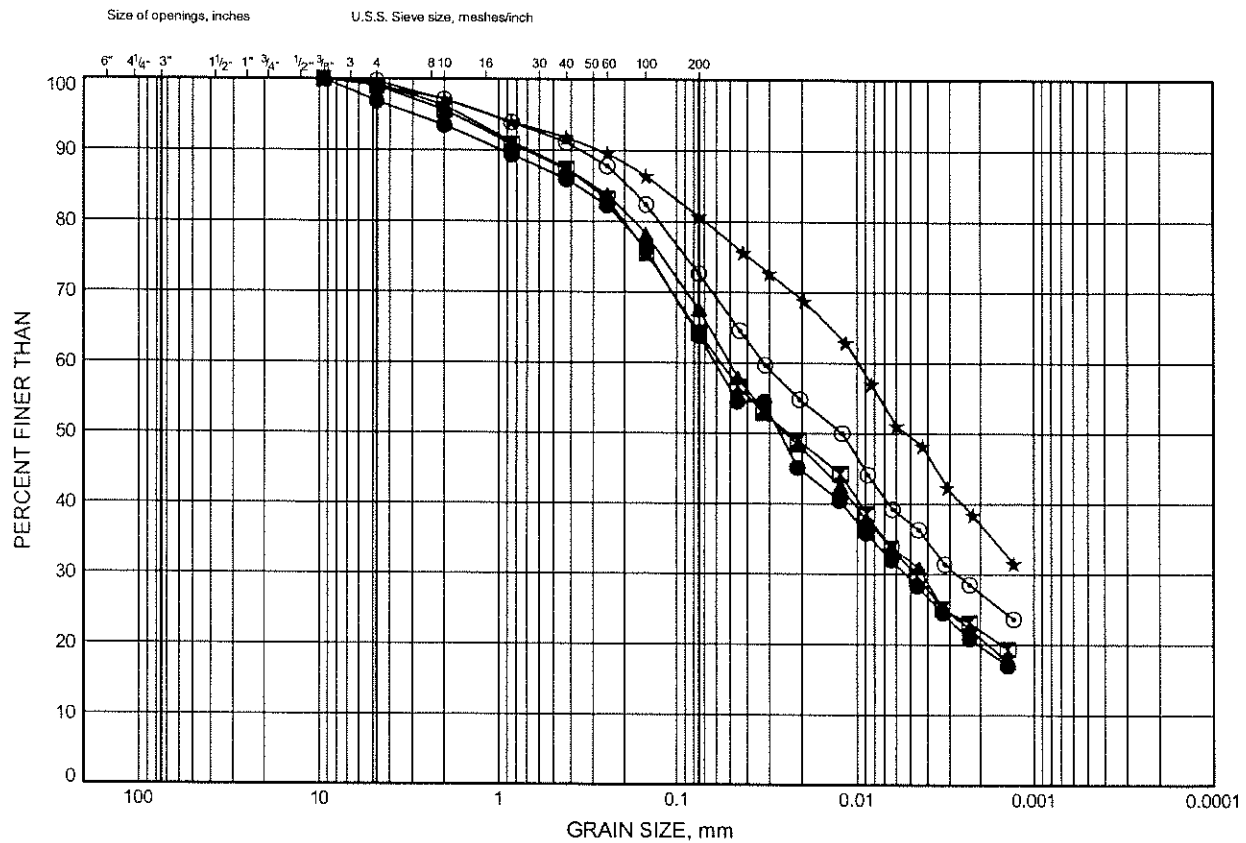


Prep'd MFA
 Chkd. RPR

Hwy 401/410 to Credit River
GRAIN SIZE DISTRIBUTION

FIGURE B9

SILTY CLAY TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	H3	1.83	192.61
⊠	H4	9.37	190.67
▲	H6	9.45	190.22
★	H7	6.32	192.85
⊙	H8	4.88	193.15

Date February 2008
 Project 2149-01-00 & 2150-01-00



Prep'd MFA
 Chkd. RPR

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	06-1116-040	SAMPLE NUMBER	NQ Run 1
BOREHOLE NUMBER	H4	SAMPLE DEPTH, m	17.2-17.3

TEST CONDITIONS

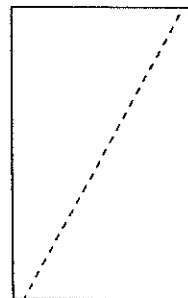
MACHINE SPEED, mm/min	0.00	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	1.90

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	8.92	WATER CONTENT, (specimen) %	3.00
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m ³	24.62
SAMPLE AREA, cm ²	17.35	DRY UNIT WT., kN/m ³	23.90
SAMPLE VOLUME, cm ³	154.76	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	388.66	VOID RATIO	0.11
DRY WEIGHT, g	377.34		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	9.5
----------------------	---	-------------------------	-----

REMARKS: Sample cracked horizontally through
middle of sample before test.

DATE:

11/16/2006



UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	06-1116-040	SAMPLE NUMBER	NQ Run 2
BOREHOLE NUMBER	H6	SAMPLE DEPTH, m	16.2-16.4

TEST CONDITIONS

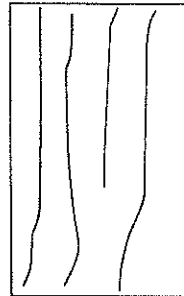
MACHINE SPEED, mm/min	0.00	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.27

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.69	WATER CONTENT, (specimen) %	2.10
SAMPLE DIAMETER, cm	4.71	UNIT WEIGHT, kN/m ³	25.03
SAMPLE AREA, cm ²	17.42	DRY UNIT WT., kN/m ³	24.52
SAMPLE VOLUME, cm ³	186.26	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	475.58	VOID RATIO	0.08
DRY WEIGHT, g	465.80		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	29.7
----------------------	---	-------------------------	------

REMARKS: Sample cracked horizontally through
middle of sample before test.

DATE:

11/16/2006

Appendix C

**Record of Borehole Sheets
And
Laboratory Test Results
(previous investigation)**

RECORD OF BOREHOLE No 1

METRIC

W P 54-82-02 LOCATION Co-ords. N 4 831 951.5; E 289 917.2 ORIGINATED BY RM
 DIST 6 HWY 10 & 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY RM
 DATUM Geodetic DATE 1983 08 24 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
194.7	Ground Surface																
0.0	Fill Silty Clay, some sand, trace gravel trace organics Stiff to Very Stiff		1	SS	37		194										
			2	SS	14												
			3	SS	21		192										
			4	SS	18												
190.3			5	SS	25												
4.4	Shale Bedrock Very Weathered Red		6	SS	80	12 cm	190										
			7	SS	115	15 cm											
187.8			8	SS	100	10 cm	188										
6.9	End of Borehole																

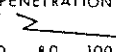

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 2

METRIC

W P 54-82-02 LOCATION Co-ords. N 4 831 975.5; E 289 898.5 ORIGINATED BY RM
DIST 6 HWY 10 & 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY RM
DATUM Geodetic DATE 1983 08 24 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH									WATER CONTENT (%) 20 40 60
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	x LAB VANE						
192.7	Ground Surface																
0.0	Fill		1	SS	17												
191.3	Silty Clay, some sand, trace gravel Very Stiff																
1.4	Heterogenous Mixture		2	SS	45												
	Silty Clay		3	SS	90/	10 cm											
	some sand and gravel (Glacial Till)		4	SS	70/	8 cm											
	Hard		5	SS	112												
188.3			6	SS	71/	12 cm											
4.4	Shale Bedrock Very Weathered Red		7	SS	70/	8 cm											
186.2																	
6.5	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 3

METRIC

W P 54-82-02 LOCATION Co-ords. N 4 832 022.5; E 289 852.5 ORIGINATED BY RM
 DIST 6 HWY 10 & 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY RM
 DATUM Geodetic DATE 1983 08 24 CHECKED BY CP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
195.7	Ground Surface																
0.0	Fill		1	SS	12												
	Silty Clay with sand		2	SS	19												
	some gravel		3	SS	35												
	Stiff to Very Stiff		4	SS	17												
	Asphaltic		5	SS	24												
	Concrete		6	SS	30												
191.0	Organic Matter		7	SS	100/	10 cm											
4.7	Heterogenous Mixture																
	Silty Clay																
	some sand and gravel																
	Hard																
188.7	Shale Bedrock		8	SS	100/	8 cm											
7.0	Very Weathered																
	Red																
186.3			9	SS	80/	8 cm											
9.4	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 4

METRIC

W P 54-82-02 LOCATION Co-ords. N 4 832 029.2; E 289 870.5 ORIGINATED BY RM
DIST 6 HWY 10 & 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY RM
DATUM Geodetic DATE 1983 08 25 CHECKED BY CP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
199.9	Ground Surface															
0.0	Fill		1	SS	25											
	Fine to Medium Sand, some silt		2	SS	15											
	--- Compact		3	SS	5											
	Silty Clay with sand some gravel		4	SS	5											
	traces of organic matter		5	SS	8											
	Firm to Very Stiff		6	SS	9											
	Silty Sand		7	SS	17											
192.2	Organic Matter		8	SS	31											
7.7	Heterogenous Mixture Silty Clay, some sand and gravel		9	SS	87											
190.1	(Glacial Till) Hard		10	SS	100	5 cm										
9.8	Shale Bedrock		11	SS	123											
	Very Weathered		12	SS	100	8 cm										
	Red		13	SS	100	5 cm										
184.6	End of Borehole															
15.3																

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 5

METRIC

W P 54-82-02 LOCATION Co-ords. N 4 831 977.2; E 289 922.2 ORIGINATED BY RM
 DIST 6 HWY 10 & 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY RM
 DATUM Geodetic DATE 1983 08 25 & 26 CHECKED BY *ep*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
200.0	Ground Surface													
0.0	Fill													
	Fine to Medium Sand Some Silt		1	SS	22		198							0 82 14 4
			2	SS	31									
			3	SS	14									
			4	SS	5									
	Loose to Dense Stiff to V.Stiff		5	SS	5		196							5 32 48 15
	Silty Sand		6	SS	16									
	Silty Clay with sand some gravel traces of organic matter		7	SS	13		194							
192.1			8	SS	30		192							
7.9	Heterogenous Mixture of Silty Clay some sand and gravel (Glacial Till) Hard		9	SS	63									3 30 47 20
189.3			10	SS	100		190							
10.7	Shale Bedrock Very Weathered Red					12 cm	188							
187.6			11	SS	100	3 cm								
12.4	End of Borehole													

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 6

METRIC

W P 54-82-02 LOCATION Co-ords. N 4 831 960.0; E 289 939.2 ORIGINATED BY RM
DIST 6 HWY 10 & 401 BOREHOLE TYPE Solid Sem Augers COMPILED BY RM
DATUM Geodetic DATE 1983 08 26 CHECKED BY GP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
199.8	Ground Surface															
0.0	Fill															
	Fine to Medium		1	SS	15											
	Sand		2	SS	22											
	some silt		3	SS	6											
	silt		4	SS	13											
	Compact		5	SS	13											
	Stiff		6	SS	9											
	Silty Sand		7	SS	12											
	Silty Clay with sand some gravel		8	SS	26											
192.0	Heterogenous Mixture of Silty Clay, some sand and gravel (Glacial Till)		9	SS	117											
7.8	Hard		10	SS	100											
188.9	Shale Bedrock		11	SS	125											
10.9	Very Weathered															
187.8	Red															
12.0	End of Borehole															

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 7

METRIC

W P 54-82-02 LOCATION Co-ords. N 4 831 998.4; E 289 899.5 ORIGINATED BY RM
DIST 6 HWY 10 & 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY RM
DATUM Geodetic DATE 1983 08 29 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
192.5	Ground Surface															
0.0	Fill - Silty Clay with sand some gravel Stiff		1	SS	16		192									
191.1			2	SS	31											
1.4	Het. Mixture of Silty Clay, some sand and gravel (Glacial Till) Hard		3	SS	58		190									
189.0			4	SS	70	5 cm										
3.5			5	SS	72											
	Shale Bedrock Very Weathered Red		6	SS	50	8 cm	188									
185.9			7	SS	100	13 cm										
6.6	End of Borehole		8	SS	100	8 cm	186									

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 8

METRIC

W P 54-82-02 LOCATION Co-ords. N 4 831 988.3; E 289 886.2 ORIGINATED BY RM
DIST 6 HWY 10 & 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY RM
DATUM Geodetic DATE 1983 08 29 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
192.6	Ground Surface												
0.0	Fill												
	Fine to Medium Sand trace clay		1	SS	29		192						
191.2	Compact												
1.4	Silty Clay, some sand and gravel		2	SS	64								
190.1	(Glacial Till) Hard		3	SS	151								
2.5	Shale Bedrock Very Weathered Red		4	SS	50/	5 cm	190						
188.2			5	SS	100/	13 cm							
4.4	End of Borehole												
	Note: Groundwater Level not Established												

Appendix D

Foundation Comparison

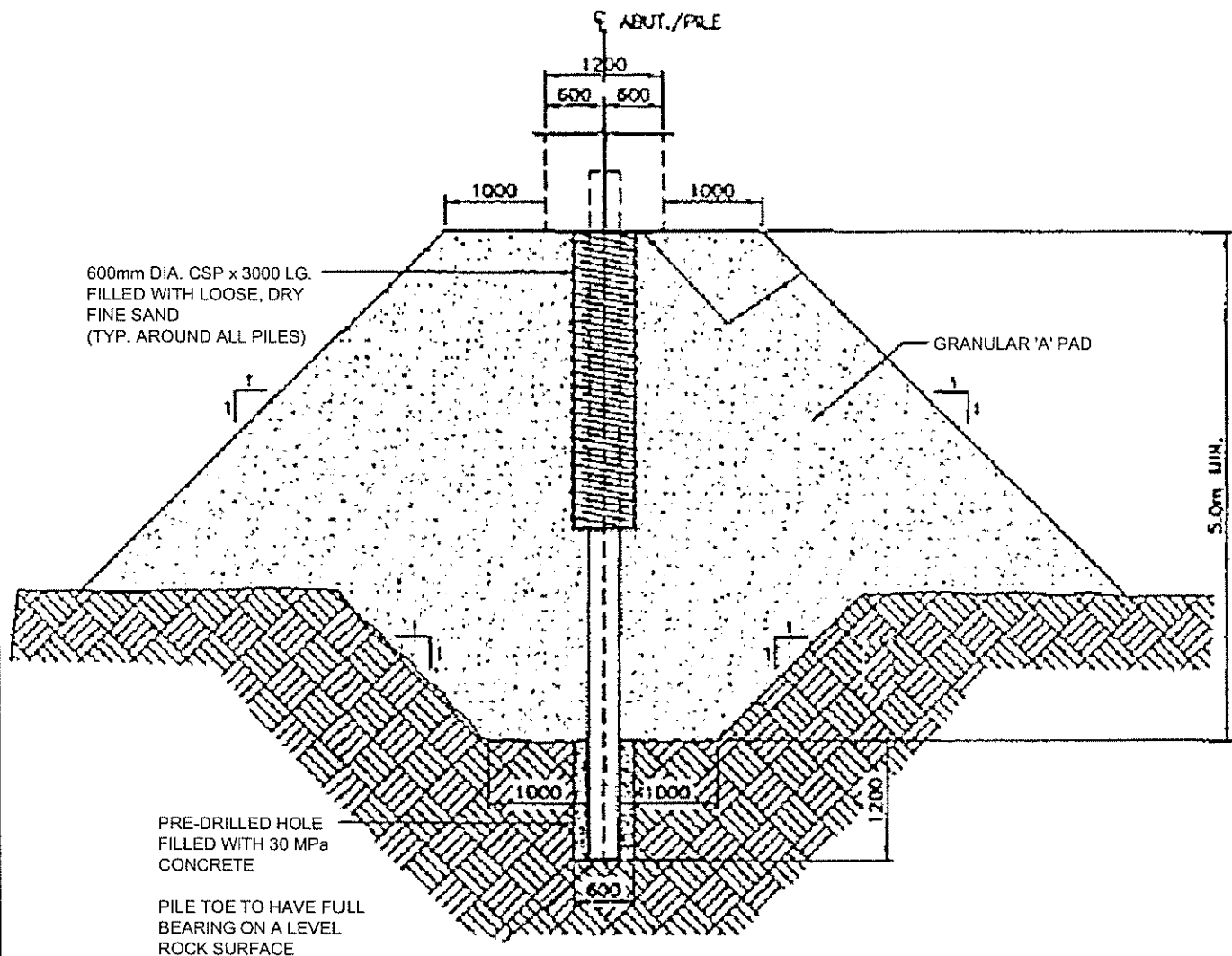
COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil or Shale	Driven Piles	Caissons
<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. High values of geotechnical resistance are available on the bedrock/hard till deposits. iii. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost of excavation into bedrock. ii. Mass concrete fill may be required to raise the founding subgrade level. iii. Cannot be incorporated into an integral <p>FEASIBLE</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by driving piles to achieve resistance in the very dense till soil overlying bedrock or to bedrock. ii. Construction of piles could continue in freezing weather. iii. Readily installed. iv. Foundation construction requires less excavation than footings. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost than footings. ii. Possibility that cobbles and boulders may be encountered in till. <p>RECOMMENDED AT THE ABUTMENTS</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Less disruption to traffic particularly at the piers. ii. High resistance is available for caissons socketed in bedrock. iii. Construction of caissons could continue in freezing weather. iv. Subexcavation of fill and variable material not required. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost than spread footings ii. Possibility of boulders being encountered during augering. iii. Potential difficulties penetrating hard limestone layers in shale. iv. Dewatering may be required v. Potential difficulty in cleaning and inspecting bases. <p>RECOMMENDED AT THE PIERS</p>


Appendix E

Figure

- **Foundation details for abutment construction in shallow bedrock**



PILE DETAIL

ENGINEER	AEG	<p>FOUNDATION DETAILS FOR INTEGRAL ABUTMENT CONSTRUCTION IN SHALLOW BEDROCK</p>	
DRAWN	MFA		
DATE	FEBRUARY 2008		
APPROVED	PKC		
SCALE	NTS		

DWG. NO.

FIGURE 1

Appendix F

List of SPs and OPSS

Suggested Text for Selected NSSP

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

- SP 902 S01
- OPSD 3501.150.
- OPSD 3102.100.
- OPSS 902
- Special Provision No. 903S01
- SP 105S19
- SP 105S01

OPSS 206, as amended by Special Provision “Amendment to OPSS 206, December 1993”, dated November 2002.

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

1. Suggested Text for NSSP on “Impact on Adjacent Structure”

It is critical that Contractor’s excavation and construction activities do not undermine or have any adverse impact on the integrity and performance of the following adjacent structures:

- *The lanes of the Highway 401 during excavation and foundation construction at the new north and south abutments.*
- *Protection of the existing underpass foundations during excavation and removal of existing abutment.*

Protection of existing approach fills.

2. Suggested Text for NSSP on “Rock Excavation and Drilling of Caisson Sockets”

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

Caisson installation through the till may encounter cobbles or boulders and the installation equipment should be capable of dislodging and removing such obstructions.

3. Suggested Text for NSSP on “Pile Installation” should contain the following:

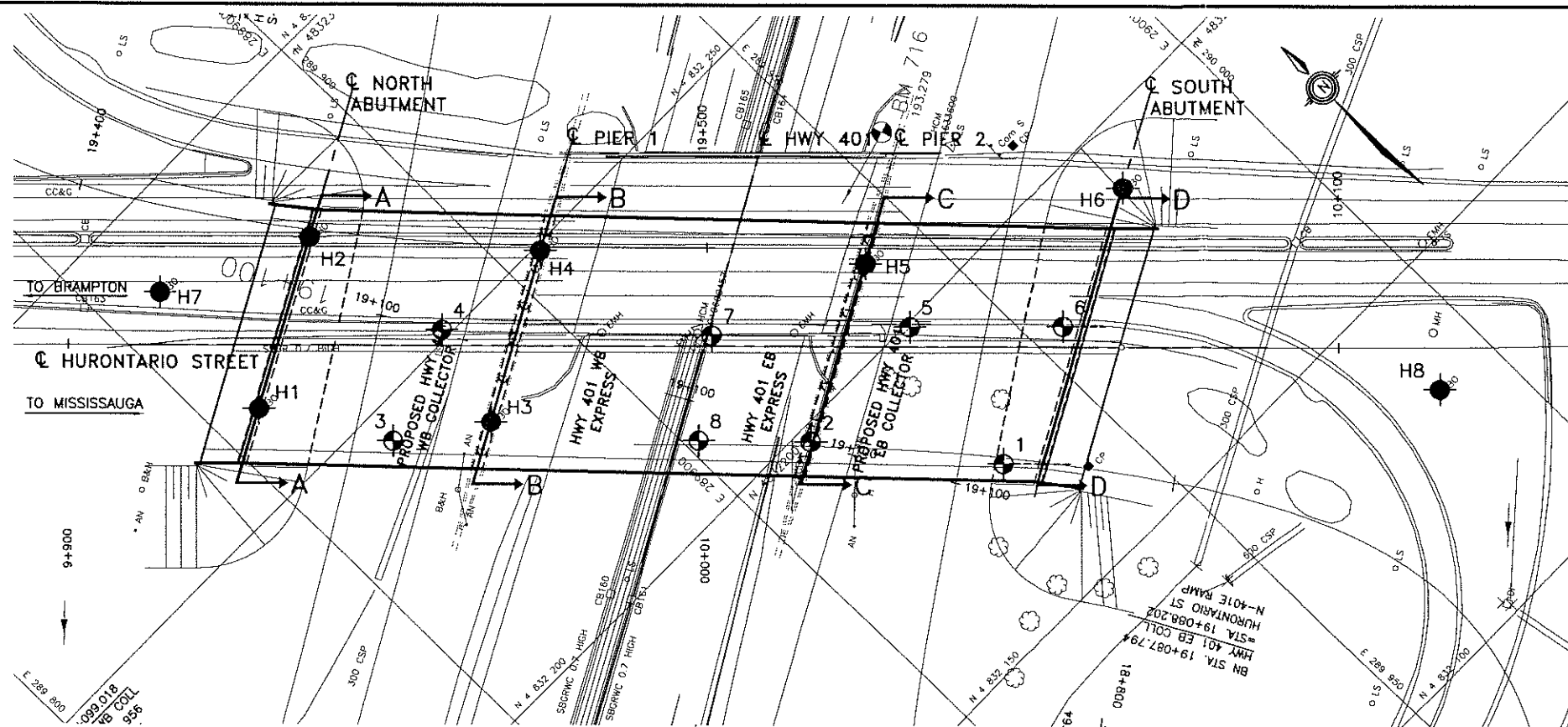
“The glacial till soil overlying the bedrock contains cobbles and boulders. The existing fill may contain obstructions such as rubble and, rock slabs. The presence of these obstructions will potentially have an impact on the installation of piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- The obstructions in the fill or cobbles and boulders in the till may impede the driving of the piles resulting in more arduous driving*
- 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*

Appendix G

Borehole Locations and Soil Strata Drawing

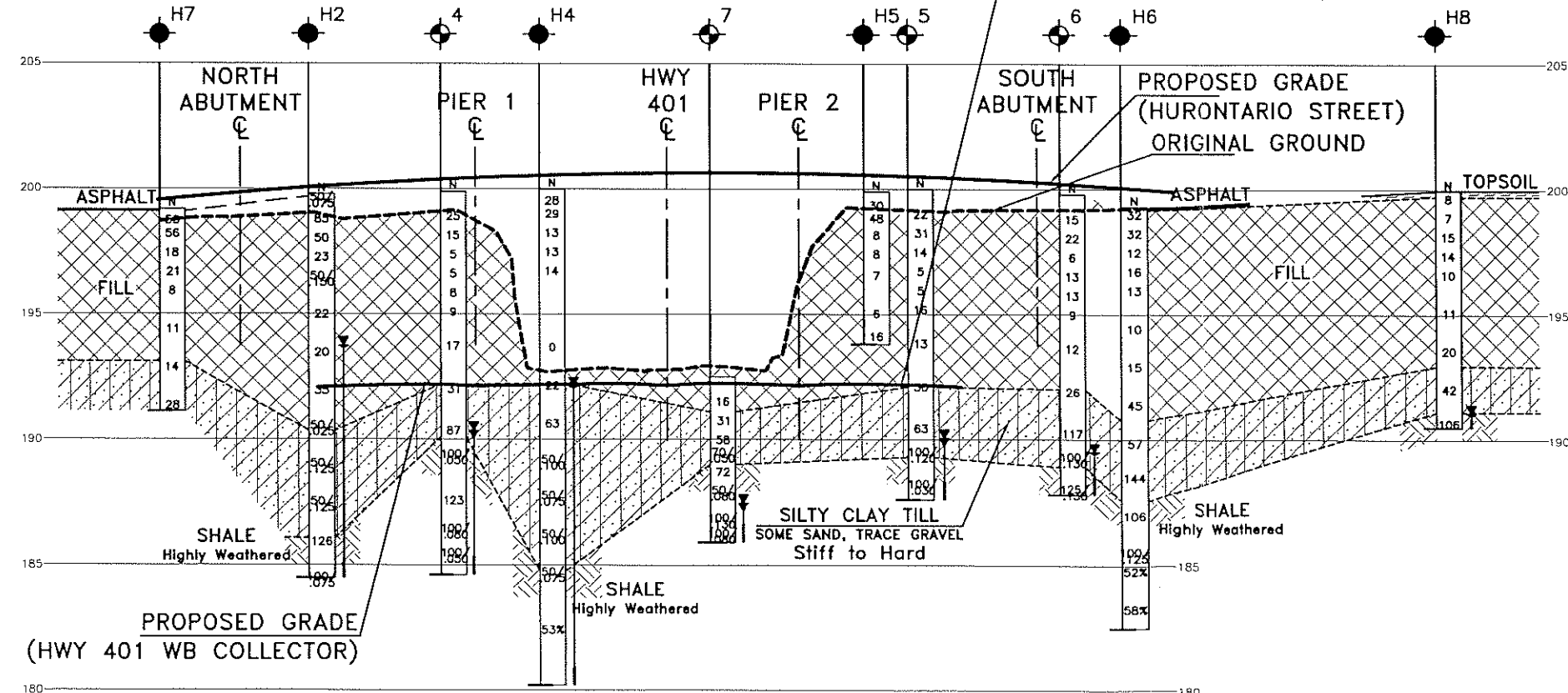
PROJECT SCALE 1:1
DATE: 01-01-2008
DRAWN BY: JHL
CHECKED BY: PKC
DESIGNED BY: RFR
SITE: 101



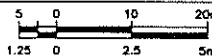
PLAN



PROPOSED GRADE
(HWY 401 EB COLLECTOR)



PROFILE OF HURONTARIO STREET



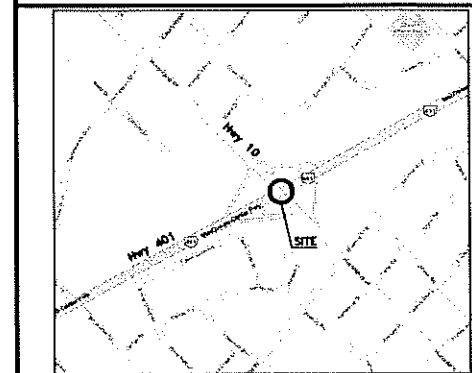
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

HWY 401
CONT No
GWP No 2149-01-00 & 2150-01-00



HWY 10
IC UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEYPLAN

LEGEND

◆	BoreHole
◆	Previous BoreHole
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
H1	197.3	4 832 263.1	289 857.2
H2	199.8	4 832 276.4	289 882.0
H3	194.4	4 832 235.7	289 881.3
H4	200.0	4 832 249.1	289 906.0
H5	200.1	4 832 211.2	289 940.2
H6	199.7	4 832 190.6	289 977.1
H7	199.2	4 832 287.0	289 859.4
H8	198.0	4 832 133.0	289 989.3
1	194.7	4 832 173.7	289 933.0
2	192.7	4 832 197.7	289 914.3
3	195.7	4 832 244.7	289 868.2
4	199.9	4 832 251.4	289 886.2
5	200.0	4 832 199.4	289 938.0
6	199.8	4 832 182.2	289 955.0
7	192.5	4 832 220.6	289 915.3
8	192.6	4 832 210.5	289 902.0

N.B. COORDINATES FOR
BOREHOLES 1 TO 8 HAVE
BEEN CONVERTED FROM
AN EARLIER DATUM.

NOTES:

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

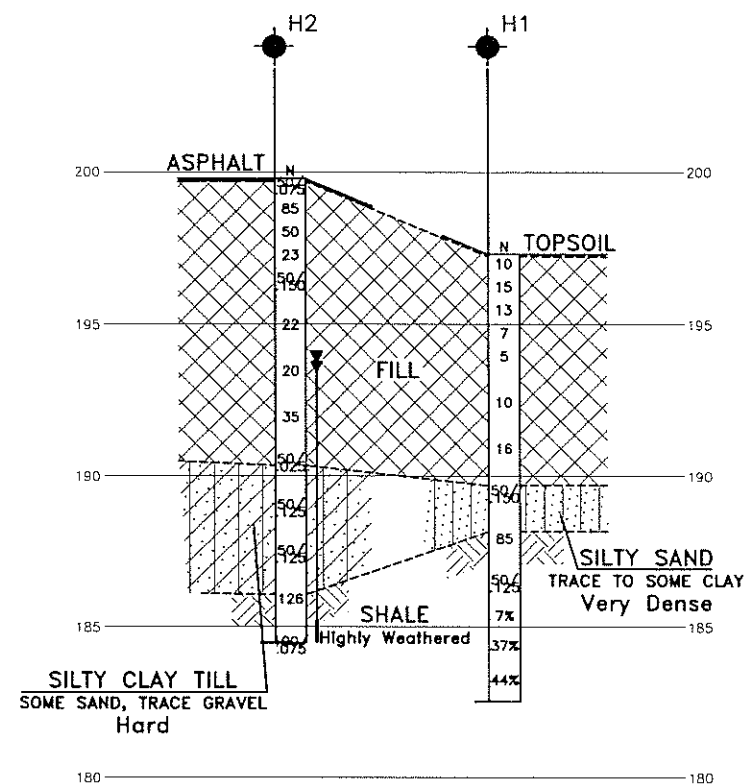
GEOCRES No. 30M12-268

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DRAWN	JHL	CHK	PKC
SITE			
STRUCT.			
DWG.			

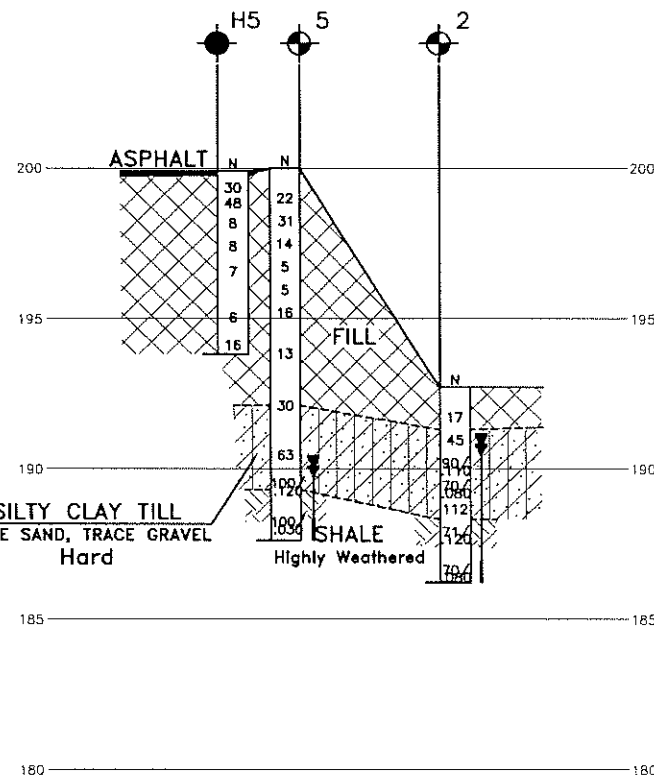
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CREATED:

DRAWING NOT TO BE SCALED
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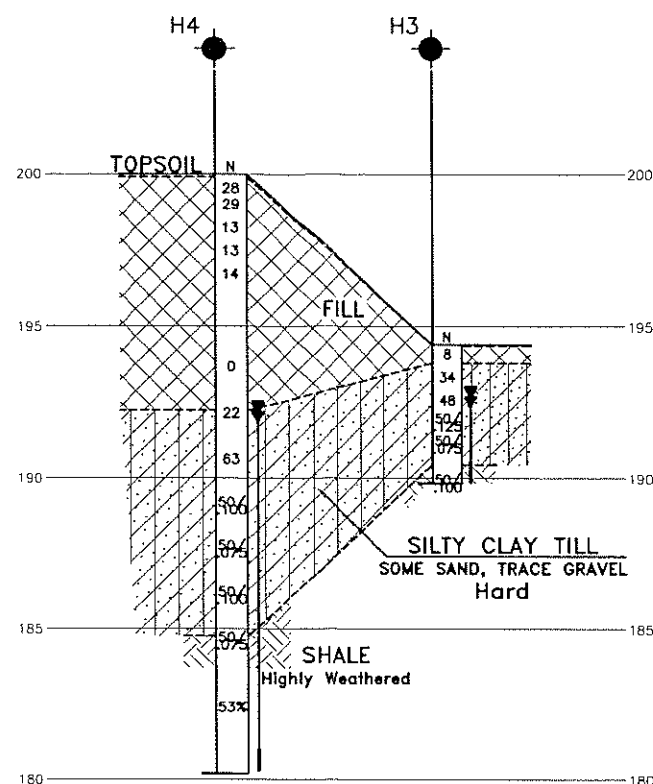
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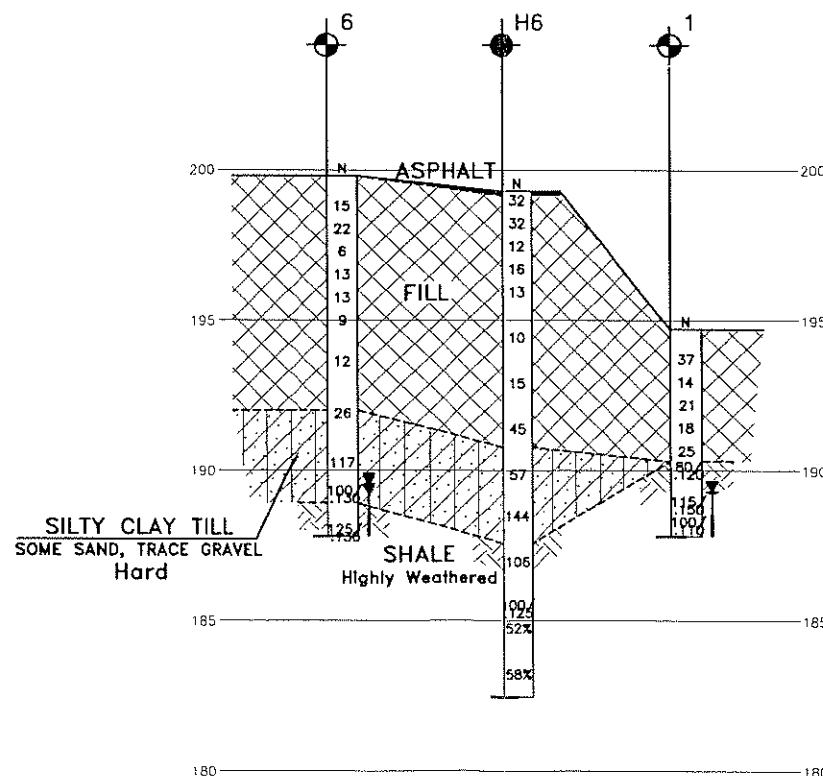
SECTION A-A
NORTH ABUTMENT



SECTION C-C
PIER 2



SECTION B-B
PIER 1



SECTION D-D
SOUTH ABUTMENT

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

HWY 401
CONT No
GWP No 2149-01-00 & 2150-01-00

HWY 10
IC UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

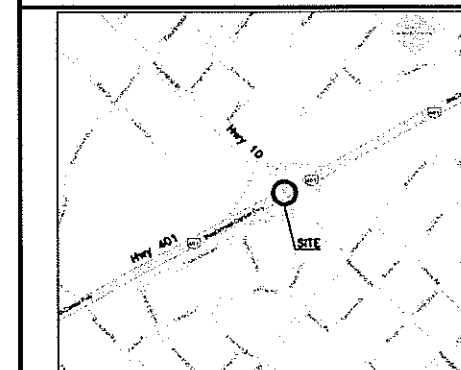


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KEYPLAN

LEGEND

- | | |
|---|---------------------------------------|
|  | BoreHole |
|  | Previous BoreHole |
| 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
H1	197.3	4 832 263.1	289 857.2
H2	199.8	4 832 276.4	289 882.0
H3	194.4	4 832 235.7	289 881.3
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H7	199.2	4 832 287.0	289 859.4
H8	198.0	4 832 133.0	289 989.3
1	194.7	4 832 173.7	289 933.0
2	192.7	4 832 197.7	289 914.3
3	195.7	4 832 244.7	289 868.2
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8	192.6	4 832 210.5	289 902.0

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BOREHOLES 1 TO 8 HAVE
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-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
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GEOCRES No. 30M12-268

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