

**FOUNDATION INVESTIGATION AND DESIGN REPORT
MUSKOKA ROAD OVERPASS SBL
HIGHWAY 11 BURK'S FALLS TO SOUTH RIVER
G.W.P. 759-93-00, W.P. 758-93-01, SITE: 44-420/2**

Geocres Number: 31E - 258

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 the proposed single-span Muskoka Road Overpass SBL structure on the proposed four-laning of Highway 11 in the Township of Strong, Ontario. A previous foundation investigation was carried out by Thurber at this site for a previous three-span structure design. The design of the structure was subsequently changed to single-span and an additional borehole was drilled to reflect these changes. The factual data from both investigations have been used in preparing this report. An earlier, preliminary report was prepared by Golder Associates and was also referenced in the preparation of this report. The borehole logs for Golder's investigation are included in Appendix C.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the present and previous investigations.

Thurber carried out the investigation as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000188.

2 SITE DESCRIPTION

The site is located south of the existing at grade intersection of Highway 11 and Muskoka Road in the Township of Strong. Bedrock outcrops and a thick cover of vegetation are evident on the west side of the existing Highway 11. On the east side of the highway there is a cleared area that was previously developed as a motel but the buildings have been removed. Beyond the developed area is a low lying swampy area with occasional mature trees.

The general site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges where exposed. Locally, however, the site lies in a gently rolling area with the bedrock obscured by glacio-fluvial soil deposits.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out on November 14, 2005 for the present investigation and between April 13 and 28, 2004 for the previous investigation. One borehole number 420-41 was drilled for the present investigation to a depth of 7.5 m. The previous investigation consisted of drilling and sampling twelve boreholes numbered 420-27, 420-29, 420-31, 420-34, 420-38, 420-39, 420-40, 420-42, 420-45, 420-46, 420-51 and 420-52 to depths ranging from 3.7 m to 10.5 m. The locations of all of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G. Some of these boreholes were drilled from the shoulders and travelled lanes of existing Highway 11.

A combination of hollow-stem auger drilling techniques and casing and washboring methods were used to advance the boreholes and samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. Boreholes at each abutment were also advanced 2.6 m to 3.2 m into bedrock by NQ size diamond coring techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. At each abutment a standpipe piezometer consisting of 19 mm PVC pipe with a slotted screen was installed and enclosed in filter sand to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1. Additional piezometers were installed in boreholes outside of the foundation elements during the previous investigation. Details of these piezometers are shown on the Record of Borehole sheets in Appendix A. The boreholes in which no piezometers were installed were generally backfilled with bentonite and drill cuttings upon completion.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the soil and rock samples for transport 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.

Table 3.1 – Piezometer Installation Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
420-31	7.6/358.3	Piezometer with 1.52 m slotted screen installed with sand filter to 5.8 m, bentonite seal from 5.7 m to 0.6 m and drill cuttings from 0.6 m to ground surface.
420-34 South Abutment	7.5/358.8	Piezometer with 1.52 m slotted screen installed with sand filter to 5.7 m, bentonite seal from 5.7 m to 0.6 m and drill cuttings from 0.6 m to ground surface.
420-45 North Abutment	7.2/358.5	Piezometer with 1.52 m slotted screen installed with sand filter to 5.5 m, bentonite seal from 5.5 m to 0.15 m and drill cuttings from 0.15 m to ground surface.
420-46	8.5/358/.5	Piezometer with 1.52 m slotted screen installed with sand filter to 6.7 m, bentonite seal from 6.7 m to 0.3 m and drill cuttings from 0.3 m to ground surface.

All remaining boreholes were abandoned in accordance with Reg903, i.e. all boreholes deeper than 3 m were grouted using bentonite grout.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the soil and rock samples for transport 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.

4 LABORATORY TESTING

All the soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples were also subjected to gradation analysis and 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. The results of point load tests on rock cores retrieved from the boreholes are shown in Table B1 in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil

Strata" drawings 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 ten boreholes, the stratigraphy was found to consist of 3.9 m to 7.6 m of overburden soils overlying Pre-Cambrian bedrock. In the remaining three boreholes, bedrock was not proved, but auger refusal was encountered at depths ranging from 3.7 to 5.5 m. The overburden soils generally consist of sand and gravel fill, sands and silts, sand some gravel and occasional cobbles and boulders.

5.1 Pavement and Granular Fill

Boreholes were drilled through the paved lanes and granular shoulders of the present Highway 11. Asphalt concrete ranging from 200 mm to 250 mm was encountered in the boreholes drilled through the paved lanes (BH 420-29, 38, 39, 40, 41 and 42).

Sand and gravel fill was encountered below the asphalt, where it was encountered, and otherwise from the ground surface. This layer of fill extends to depths ranging from 1.3 m to 2.2 m or from elevations ranging between 365.7 m and 362.8 m. This sand and gravel fill generally grades into a fill consisting of sand with trace gravel at a depth of 0.7 m below ground surface.

Three samples of this fill material below a depth of 0.7 m were subjected to grain size distribution tests and the results are illustrated in Figure B1.

SPT 'N' values ranged from 4 to 55 blows for 0.3 m penetration but generally, most recorded 'N' values ranged from 12 to 40. Based on these results the fill is considered to have a compact to dense relative density with occasional loose and very dense zones. The moisture content of samples from this deposit ranged from 1% to 16%.

In Borehole 420-41, underlying the sand with trace gravel, a layer of sandy silt fill extends to a depth of 2.1 m. An SPT 'N' value of 11 blows per 0.3 m indicates that this material has a compact relative density. The moisture content of this material was approximately 36%.

5.2 Sandy Silt

A layer of sandy silt was identified across the south portion of the site. The sandy silt was encountered below the sand and gravel fill at depths ranging from 1.4 m to 2.2 m below ground surface. This deposit extended to depths of 4.0 m to 5.6 m or from elevations varying from 361.9 m to 361.0 m.

Four samples from this deposit were subjected to grain size distribution tests and the results are presented in Figure B2. These results show a soil consisting of 0 to 4% gravel, 19 to 53% sand, 40 to 75% silt and 3 to 7% clay sized particles. Based on these results the deposit is essentially a cohesionless soil ranging from a sandy silt to sand and silt. The

upper part of this layer contains trace rootlets and organics. Occasional cobbles were noted in this layer below a depth of 2.2 m.

SPT 'N' values ranged from 7 to 26 blows for 0.3 m penetration in this layer indicating a loose to compact relative density. The moisture content of samples from this deposit generally ranged from 11% to 24% and in Borehole 420-31 a moisture content of 32% was recorded in the organic rich upper zone of this deposit.

5.3 Sand

A deposit of sand to silty sand was generally encountered across northern portion of the site. This cohesionless layer was encountered at depths ranging from 1.3 m to 2.2 m below ground surface or from elevations ranging between Elev. 365.6 m and Elev. 364.1 m. The deposit extends to depths of 2.2 m to 5.5 m or from elevations ranging between Elev. 364.3 m and Elev. 361.2 m.

Samples from this deposit were subjected to grain size distribution tests and the results are shown in Figures B3a and B3b. The results show a soil consisting of 0 to 8% gravel, 66 to 90% sand and 3 to 32% silt and clay. The upper part of this deposit contains rootlets and organics. Occasional cobbles and boulders were noted in this layer below a depth of about 1.7 m.

Standard Penetration tests in this deposit gave 'N' values ranging from 12 to more than 50 blows per 0.3 m penetration. Based on these results the deposit is considered to have a compact to very dense relative density. The moisture content of samples from this stratum generally varies between 3% and 29%. In Boreholes 420-29 and 420-42 moisture content values of 49% and 40% respectively, were recorded in the organic rich upper zone of this layer.

5.4 Sand and Gravel with Cobbles and Boulders

In some boreholes, the sand and silt were underlain by bedrock and in the remainder by a discontinuous layer of sand and gravel with occasional cobbles and boulders at depths ranging from 2.1 m to 5.6 m below ground surface. At some of the boreholes this layer was fully penetrated and the deposit was found to extend to depths ranging from 3.9 m to 7.6 m below ground surface or from elevations ranging from Elev. 362.6 m to Elev. 359.5 m. It should be noted that cobbles and boulders are inferred to exist in this deposit based on the resistance to augering that was observed while drilling through this deposit.

Two samples from this deposit were subjected to grain size distribution tests and the results are illustrated in Figure B4.

Standard Penetration tests in this deposit gave 'N' values of more than 50 blows per 0.3 m penetration indicating a very dense relative density. The moisture content of samples from this stratum varies between 10% and 22%.

5.5 Bedrock

The overburden soils described above are underlain by gneiss bedrock. Bedrock was proved by coring at the north and south abutments. Table 5.1 summarizes the bedrock depth and the elevations to the top of bedrock at the foundation elements.

TABLE 5.1 – Depth to Bedrock at Foundation Elements

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
South Abutment	420-34	4.3	362.0
	420-38	5.5*	361.4*
North Abutment	420-41	4.9	362.1
	420-45	4.0	361.7

* Possible bedrock, based on auger refusal.

During the previous investigation, bedrock was cored at several other locations in the vicinity of the structure. The bedrock depth and the elevations to the top of bedrock are shown on the Record of Borehole sheets in Appendix A and on the “Borehole Locations and Soil Strata” drawing in Appendix G.

The gneiss bedrock is generally described as fresh to slightly weathered. Its colour is grey white to light pink with black blotches and occasional black bands visible in most cores.

Outside of the foundation elements, in Borehole 420-46 biotite rich inclusions were found in the structure of the gneiss bedrock. A biotite schist layer was encountered from 5.0 m to 5.1 m above the surface of the gneiss. In run 2 a layer of biotite was encountered from 7.0 m to 8.2 m with no recovery from 7.5 m to 7.9 m in this layer. Underlying this layer of biotite, biotite schist was encountered from 8.2 m to 8.6 m.

Core recovery in the bedrock was generally between 95% and 100%. However, in Borehole 420-46 a core recovery of 73% was recorded in run 2 where the biotite layer was encountered. The RQD values generally ranged from 53% to 100% indicating fair to excellent rock quality. Outside of the foundation elements, in Borehole 420-39 an RQD value of 45% was recorded in the first run indicating poor rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally low ranging from 0 to less than 5. Fracture Indices greater than 5 were obtained in some core runs indicating the presence of rubble zones within the rock mass. Sub-vertical to vertical joints were encountered within the rock mass. They were mostly tight with no infilling or secondary weathering material.

The unconfined compressive strength of the rock cores is estimated to range between 59 and 148 MPa indicating strong to very strong intact rock. Outside of the foundation elements, in Borehole 420-46 the gneiss bedrock immediately above the biotite layer is

considered to be moderately strong based on an estimated unconfined compressive strength value of 23 MPa. These estimated rock strength values are based on point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Table B1 in Appendix B.

5.6 Groundwater Conditions

A standpipe piezometer was installed at each foundation element in a selected borehole and water levels were measured on separate visits made after the completion of drilling. The water level readings are presented in Table 5.2.

Table 5.2: Water Level Measurements

Date	BH 420-31		BH 420-34		BH 420-45		BH 420-46	
	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)
April 15, 2004							1.6	365.4
April 16, 2004	1.3	364.6	1.4	364.9	1.1	364.6	1.5	365.5
June 18, 2004	*	*	*	*	1.4	364.3	*	*
October 14, 2004	*	*	*	*	2.0	363.7	*	*
November 29, 2005	*	*	*	*	1.2	364.5	*	*

* Piezometer Destroyed

Additional water level readings from the piezometers installed in the previous investigation are shown on the Record of Borehole sheets in Appendix A. Based on these observations, local groundwater levels exist at Elevations 363.7 to 365.5. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

6 MISCELLANEOUS

The borehole locations were marked in the field by surveyors from Marshall Macklin Monaghan Ltd. who also provided Thurber with the coordinates and geodetic elevations of the boreholes after drilling was completed. Thurber obtained utility clearances prior to drilling.

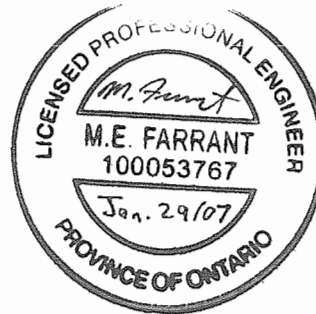
All-Terrain Drilling of Waterloo, Ontario supplied truck and track mounted CME 75 drill rigs and conducted the drilling, sampling and in-situ testing operations.

The drilling and sampling operations in the field were supervised on a full time basis by Mr. George Azzopardi and Mr. Stephane Loranger of Thurber.

Mr. Alastair E. Gorman, P.Eng. and Mr. Mark E. Farrant, P.Eng., directed the field program and prepared the report..

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

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

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 embankments for the proposed structure.

It is understood that Highway 11 SBL will cross over Muskoka Road via a single-span structure with a 38 m span between the abutments. Muskoka Road will be realigned south of its existing alignment to pass under the new Highway 11 SBL at Sta. 18 + 063.

At the south abutment, the finished grade of Highway 11 will be at Elevation 371.6 and the existing ground surface, or highway grade, lies at average Elevation 366.6. The resulting embankment height above existing Highway 11 level will, therefore, be in the order of 5.0 m at the south abutment. The underside of abutment will lie at Elevation 366.8.

At the north abutment, the finished grade of Highway 11 will be at Elevation 372.4 and the existing ground surface averages Elevation 366.3 at the abutment, resulting in an approach embankment in the order of 6.1 m high. The underside of abutment will lie at Elevation 367.4.

The grade of Muskoka Road will lie approximately at Elevation 364.

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 this investigation, including boreholes drilled for an earlier version of the design. Reference has also been made to four boreholes drilled in a previous investigation by Golder Associates.

8 STRUCTURE FOUNDATIONS

The proposed bridge is a single-span overpass structure with two abutment foundation elements.

At the south abutment the stratigraphy consists of 4.3 to 5.5 m of loose to compact, cohesionless overburden soils overlying bedrock.

At the north abutment, the stratigraphy consists of 4.0 to 4.9 m of cohesionless soils overlying bedrock.

The overburden consists of asphalt pavement and granular fill underlain by sands silts, sand and gravel and cobbles and boulders.

The groundwater level exists at approximate Elevation 364.9 at the south abutment and 365.5 at the north abutment.

Initial consideration was given to the following foundation types:

- Spread footings (on native soil, engineered fill or bedrock)
- Augered Caissons (drilled shafts)
- Driven piles

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

8.1 Spread Footings on Native Soil

Provided a minimum footing width of 2.0 m is maintained, footings may be designed at or below the elevations and for the geotechnical resistances given in Table 8.1.

The geotechnical resistances quoted below 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.

Table 8.1 – Foundations on Native Soil

Location	Relevant Boreholes	Geotechnical Resistance (kN)		Founding Elevation
		ULS _r	SLS	
South Abutment	BH 420-34, 420-38	300	200	364.0
North Abutment	BH 420-41, 420-45	500	350	364.0

At this specific location and considering the depth to bedrock, footings designed on the basis of the geotechnical resistance values given above are expected to experience total settlements not exceeding 20 mm. This settlement is expected to be substantially complete by the end of construction.

The sliding resistance of mass concrete poured on compact to dense sand and silt, or the very dense gravel and sand or cobbles and boulders may be computed on the basis of an ultimate coefficient of friction of 0.60.

Temporary excavations required to construct these footings will extend in granular soils below the water table. Dewatering prior to excavation will be required to construct the footings in the dry and to prevent sloughing of the sides or disturbance of the base of the

excavation due to the inflow of groundwater. The dewatering method adopted must depress the groundwater level to at least 0.6 m below the base of excavation and must maintain a stable, unwatered excavation throughout the duration of the footing construction. Dewatering must remain operational and effective until the footing is constructed and backfilled.

8.2 Spread Footings on Engineered Fill

If it is beneficial to the overall design, spread footings may be founded on an engineered fill pad.

At this site, the engineered fill pads will be founded on compact to very dense, cohesionless soils and it is recommended that the engineered fill pads be at least 2 m thick. The underside of the engineered fill pad must not be higher than the elevations given in Table 8.2. In this case the footings may be designed on the basis of a geotechnical resistance at factored ULS of 900 kPa and of 350 kPa at SLS.

Table 8.2 - Maximum Elevation for Engineered Fill

Location	Relevant Boreholes	Founding Elevation
South Abutment	BH 420-34, 420-38	364.0
North Abutment	BH 420-41, 420-45	364.0

If an engineered fill pad is used at this site, all topsoil, organics and loose/soft soils should be stripped from below the footprint of the engineered fill pad and the native soil should be stripped at least to the elevations in Table 8.2.

Temporary excavations required to construct the engineered fill pads may extend in granular soils below the water level prevailing at the time of construction. Dewatering prior to excavation may be required to construct the fill pad and the footing in the dry and to prevent sloughing of the sides or disturbance of the base of the excavation due to the inflow of groundwater. The dewatering method adopted must depress the groundwater level to at least 0.6 m below the base of excavation and must maintain a stable, unwatered excavation throughout the duration of the fill pad and footing construction. Dewatering must remain operational and effective until the footing is constructed and backfilled or until the engineered fill pad is completed to a level at least 500 mm above the groundwater level.

The engineered fill must consist of OPSS Granular "A" compacted to 100% of its Standard Proctor maximum dry density (SPMDD) at $\pm 2\%$ of optimum moisture content (OPSS 501, Section 501.08.02, Method A) and generally conforming to the geometry illustrated in Figure 1.

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.

At this specific location and considering the depth to bedrock, footings designed on the basis of the geotechnical resistance values given above are expected to experience total settlements not exceeding 20 mm. This settlement is expected to be substantially complete by the end of construction.

The sliding resistance of mass concrete poured on a compacted Granular "A" pad may be computed on the basis of an ultimate coefficient of friction of 0.70.

8.3 Spread Footings on Bedrock

The top of bedrock elevations at the abutments established in the course of the investigation are shown in Table 5.1. Based on these elevations, it is estimated that bedrock will be approximately 5 m below the design underside of the footing at both abutments.

Two design options that can be considered for the support of footings on bedrock are:

- Design the footing to bear directly on bedrock
- Design the footing to bear at an elevation appropriate to the structure and place mass concrete fill between the underside of the footing and the bedrock.

Footings bearing directly on the bedrock may be designed on the basis of a factored geotechnical resistance at ULS of 10,000 kPa. The SLS condition will not govern for a footing bearing on bedrock.

Footings bearing on mass concrete fill may be designed on the basis of a factored geotechnical resistance at ULS of 10,000 kPa, provided the concrete fill will safely support this loading. It is recommended that the fill consist of 30 MPa concrete and that the plan dimensions of the fill be at least 0.6 m larger than the footing dimensions in all directions to mitigate stress concentrations in the unreinforced concrete. The SLS condition will not govern for a footing bearing on mass concrete as described herein.

The stated bearing resistance is for vertical, concentric loads. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4.

In either of the above cases, all overburden must be stripped from the bedrock within the footprint of the footing or the mass concrete fill and any broken, disturbed rock must be removed. The excavation must be unwatered prior to placing concrete.

The concrete may be placed directly in contact with the bedrock after it has been exposed and cleaned. It is also acceptable for the Contractor to bench the bedrock to facilitate his construction process.

The use of spread footings bearing on bedrock is not a preferred option at this site on account of:

- The depth of excavation of 4 to 5 m below existing grade and 2 to 3 m below the groundwater table
- The possibility of encountering cobbles and boulders, especially at the bedrock contact
- The bedrock surface is sloping and uneven
- The difficulty that will probably be encountered in any attempt to unwater the excavation.

These factors will make excavation and construction of footings bearing on bedrock a high risk operation. There is a high probability that a Contractor would anticipate serious difficulties in maintaining a stable, unwatered excavation and would include a large risk premium in his pricing.

From a geotechnical and cost effectiveness perspective, spread footings on bedrock are not recommended.

8.4 Caissons

Caissons are not considered to be a viable alternative at this site due to the shallow cohesionless soil with cobbles and boulders, the high groundwater table and the uneven bedrock surface.

8.5 Steel Piles Bearing on Bedrock

The foundations may be supported on steel H-piles bearing on bedrock.

The stratigraphy encountered at the site consists of relatively thin overburden deposits overlying flat to gently sloping bedrock. Table 8.3 below gives details on the encountered bedrock elevations and the estimated pile lengths from underside of abutment to the top of bedrock.

If an integral abutment design is considered, the upper 3.0 m length of the pile must be unrestrained in order to allow sufficient flexibility. Beyond the 3.0 m required for flexibility, the pile must have sufficient embedment to develop the geotechnical resistance and to maintain the position of the pile tip horizontally. As shown in Table 8.3, piles driven to the top of the bedrock will not achieve this minimum length.

Table 8.3 – Estimated Pile Lengths

Location	Borehole No.	Depth to Bedrock* (m)	Top of Bedrock Elevation (m)	Underside of Abutment Stem Elevation (m)	Estimated Length of Pile (m)
South Abutment	420-34	4.3	362.0	367.7**	5.7
	420-38	5.5	361.4		6.3
North Abutment	420-41	4.9	362.1	368.9**	6.8
	420-45	4.0	361.7		7.2

* From ground surface existing at the time of investigation

** From the General Arrangement Drawing

The recommended minimum length of pile below the underside of abutment is 6 m, consisting of 3 m in loose sand and a minimum of 3 m driven into resisting material below. At this site, it is considered that the minimum pile length of 5.7 m will provide the required embedment in the founding material.

8.5.1 Axial Resistance

Four steel pile sections believed to be currently available have been considered for use in the proposed foundations. The factored, vertical, concentric, geotechnical resistances at ULS for these pile sections, when bearing on bedrock, are as follows:

- 2,000 kN for HP 310 x 110
- 2,400 kN for HP 310 x 132
- 2,750 kN for HP 310 x 152
- 2,400 kN for HP 360 x 132

The SLS condition will not govern for piles founded on bedrock.

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

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used in the fills through which the piles will be driven.

8.5.2 Downdrag

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

8.5.3 Integral Abutment Considerations

From a geotechnical perspective, the subsurface conditions at this site are considered to be suitable for the construction of conventional, semi-integral or integral abutments.

However, the recommended foundation system of H-piles makes integral abutments a feasible option.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. Since the upper 3 m of the piles will lie partially or completely in compacted fill, the upper 3 m of the piles should be surrounded by one of the following systems:

- For a “conventional integral abutment” supported on top of the piles - a 600 mm diameter CSP filled with sand, or
- For “false abutment” - concentric CSPs in accordance with standard integral abutment design procedures.

The sand must be placed in the CSP after the pile has been driven to avoid the danger of the sand being densified by pile driving.

Backfill sand should meet the gradation shown in Table 8.4.

Table 8.4 – 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%

The Contract drawings or Special Provisions must contain instructions to the Contractor for the installation of the CSP's.

8.5.4 Lateral Resistance

The lateral resistance of a pile may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

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

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

where z = depth of embedment of pile in metres

D = pile width in metres

n_h = coefficient of horizontal subgrade reaction (Table 8.5)

γ = unit weight (Table 8.5)

K_p = passive earth pressure coefficient (Table 8.5)

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

Table 8.5 – Recommended Soil Parameters

Location	Elevation	n_h (kN/m^3)	K_p	Unit Weight* (kN/m^3)	Soil Conditions
South Abutment	Granular B-I Fill	15,000	3.3	22	Compacted fill.
	OGL to 365.5	5,000	3.3	21	Gravel and sand fill, compact
	365.5 to 362.0 (or BDR)	3,000	3.0	10	Sand, trace gravel and cobbles, loose to compact
North Abutment	Granular B-I Fill	15,000	3.3	22	Compacted fill.
	OGL to 364.5	3,000	2.8	11	Gravel/Sand fill, loose to compact
	364.5 to 362.0 (or BDR)	8,000	3.3	11	Gravel and sand with cobbles, dense

*Buoyant unit weight below the water table.

The spring constant, K_s , for analysis may be obtained by the expression, $K_s = k_s \times L \times D$ (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, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$. 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.

Since the piles are end bearing on rock, the vertical resistance will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the equation for k_s and p_{ult} quoted above may be used in conjunction with appropriate reduction factors.

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

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

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

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

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

Intermediate values may be obtained by interpolation.

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

8.5.5 Pile Tips

Due to the possible presence of cobbles and boulders above bedrock, the tips of all piles should be fitted with H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or Pruyn Points or approved equivalent.

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

- The piles will be driven through soil containing cobbles and boulders, which requires a higher level of protection than driving into soils containing only smaller particle sizes
- Some piles may achieve refusal on large boulders, which will require the same pile tip protection and reinforcement as founding on bedrock
- Some piles may fully penetrate the zone of cobbles and boulders and achieve refusal on 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.

8.5.6 Pile Installation

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

The contract documents should include a NSSP alerting the Contractor to the presence of cobbles and boulders in the lower sand layer. Suggested wording for this NSSP is contained in Appendix E.

8.5.7 Pile Driving

The appropriate note for the foundation drawing is Note 5, i.e. "Piles to be driven to bedrock".

8.6 Recommended Foundation

The recommended foundation system for this structure is abutments supported on steel H-piles grouted in, or driven to bedrock.

8.7 Frost Cover

Pile caps and footings on earth must be provided with a minimum of 1.9 m of earth cover over the footing base (founding elevation). Frost penetration is not an issue for footings bearing on bedrock or mass concrete fill.

9 EXCAVATION AND BACKFILL

9.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site may be classified as Type 3 soils above the water table and Type 4 soils below the water table. Excavation below the groundwater level is not recommended without prior dewatering. Provided

dewatering is carried out as described below, temporary excavations may be sloped at 2H:1V.

9.2 Foundations

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

Bidders must be alerted to the fact that excavation must be carried out through cohesionless soils, which may include cobbles and boulders. Excavation to anticipated founding elevations for footings may require prior dewatering.

If founding on bedrock is selected, bidders must be alerted to the fact that the bedrock surface is uneven. The methods used to excavate, control groundwater and maintain a stable excavation must be selected by the Contractor. However, when different options are evaluated, it must be recognized that there may be difficulties in depressing the groundwater level to the bedrock surface or, alternately, in obtaining a seal between driven sheeting and the bedrock to prevent the inflow of groundwater carrying soil with it. The Contractor may have to consider some or all of the following:

- An oversize sheeted excavation to allow space to pack filter material at the toe of the sheeting
- An oversize excavation to allow space to collect and remove seepage water
- Placing a mud slab within a sheeted excavation to prevent the continued migration of soil into the excavation.

10 GROUNDWATER CONTROL

The groundwater level is close to the ground surface at this site. The groundwater must be controlled during construction to maintain a stable excavation and to allow concrete to be placed in an unwatered excavation.

The design of the groundwater control system is the responsibility of the Contractor. However, suitable systems that might be considered include pumping from filtered sumps for nominal penetration below the groundwater level or the use of a sheeted excavation to bedrock. The effectiveness of dewatering wells may be limited by the presence of bedrock at shallow depth.

Any accumulation of water from the base of the excavation should be removed prior to placing concrete or compacting granular fill. Placement of concrete or compacting engineered fill must be done in the dry.

11 APPROACH EMBANKMENTS

Approach embankment construction using either earth fill or rock fill is feasible on the foundation soils encountered at this site. Settlement in the order of 25 mm should be expected under the

loading imposed by the 5 to 6 m of approach fill but due to the non-plastic nature of the foundation soils, the settlement will be immediate and essentially complete when construction of the fill is completed. Abutment piles should be installed after completion of the abutment approach fills up to the underside of the abutment stem.

Embankment design must make allowance for the settlement within the mass of the fill. Appropriate values for internal consolidation are:

Earth fill	1% of the fill height
Granular fill	0.5% of the fill height
Rock fill	0.5% of the fill height

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry and also to a large degree on the material used to construct the embankment. If the embankment is constructed of blast rock fill, it may be assumed that the side slopes will be stable at inclinations up to 1.25H:1V. Embankments constructed using granular material or select subgrade material will have stable side slopes at inclinations of up to 2H:1V. Earth fill embankments will also generally have stable side slopes at 2H:1V if constructed of cohesionless earth fill compacted in accordance with OPSS 501.

For the purpose of embankment stability analyses, the commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used. The Bishop's simplified method for stability analysis was employed.

Global stability analyses were conducted for 2H:1V SSM or earth fill embankments and for 1.25H:1V rock fill embankments. In each case the factor of safety against global failure was greater than 1.3. Typical computer output is shown in Appendix F.

The factors of safety obtained in the course of the analysis are summarized in Table 11.1.

Table 11.1 – Approach Embankment Factors of Safety

Location / Material	Condition	Factor of Safety	Figure
South Approach			
Earth Fill	Normal	1.5	F1
Earth Fill	With Seismic	1.2	F2
Rock Fill	Normal	1.4	F3
Rock Fill	With Seismic	1.2	F4
North Approach			
Earth Fill	Normal	1.4	F5
Earth Fill	With Seismic	1.1	F6
Rock Fill	Normal	1.3	F7
Rock Fill	With Seismic	1.1	F8

It is recommended that all topsoil, organics, loose soils and other deleterious material be removed from the footprint of the approach fills. Embankment construction should be in accordance with the latest version of SSP 206S03.

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

12 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used subject to the requirements presented in this section.

RSS walls must be specified to be “High Performance” and “High Appearance”. The contract drawings must 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.

12.1 Foundation

The performance of an RSS is dependent, among other factors, on 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 and levelling pad must be founded at or below the elevations given in Table 8.1. Alternatively, the RSS and levelling pad may be founded on a pad of Granular “A” engineered fill founded at the elevations given in Table 8.2.

The geometry of the engineered fill must conform to the limits illustrated in Figure 2.

The subgrade should be competent and free of organics, soft or deleterious soils. The native soil under the RSS foundation should be re-compacted. Acceptance of the re-compaction should be based on OPSS 501, Method A modified by a NSSP. Suggested wording for the NSSP is provided in Appendix E.

Dewatering will be required to prepare the subgrade for placement and compaction of the engineered fill pads.

The following parameters may be used for the design of the RSS:

- Factored geotechnical resistance of 300 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 200 kPa at Serviceability Limit States (SLS)
- Ultimate coefficient of sliding resistance of cast in-situ concrete levelling pad on native soil = 0.6
- Ultimate coefficient of sliding resistance of RSS mass on Granular A = 0.6

Settlement under a RSS mass constructed as outlined above is expected to be less than 25 mm and to occur essentially as the RSS is constructed.

The RSS is a proprietary system and the supplier must design for internal, sliding and overturning stability and for any other failure modes identified by the supplier.

12.2 Global Stability

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.

If a RSS wall system is selected, the global stability must be analyzed after the location of the wall is known. Typically, however, a wall founded close to the elevation of Muskoka Road, on foundations prepared as described in this section will possess acceptable global stability.

13 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment must be granular material. In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to limit rock fill used as abutment backfill to fragments no greater than 300 mm and including adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular "B" Type II.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3501.000, and rock backfill should be placed to the extents shown in OPSD 3505.000.

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 should be restricted in accordance with SSP 105S10.

The design of the abutment should incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

14 EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3501.000 or OPSD 3505.000, as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill and 1.25H:1V for rock backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used.

Earth pressures acting on the structure for a fully drained backfill should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

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

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table below)

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 at a depth of 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 given in Table 14.1.

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 or semi-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 wall moves toward the soil mass. However, the use of Granular "B" Type I may be restricted if the approach embankment consists of rock fill.

The factors in the 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 CHBDC, 2000.

Table 14.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ; \gamma = 19.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	5.0	-

* For wing walls.

15 SEISMIC CONSIDERATIONS

15.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 2. The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 2
- Zonal Velocity Ratio 0.1
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.1
- Peak Horizontal Acceleration 0.11

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

15.2 Liquefaction Potential

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

Using this method and assuming an earthquake of magnitude 7.5, it is estimated that under the existing conditions there is negligible potential for liquefaction of the foundation soils below the abutments. Therefore, the vertical geotechnical resistance of the foundations and embankments will not be compromised.

The embankments themselves will be constructed above the groundwater level and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur but it is expected to be of limited nature and readily repairable.

15.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

Table 15.1 – 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 $\phi = 32^\circ$; $\delta = 16^\circ$ $\gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ$; $\delta = 21^\circ$ $\gamma = 19.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.3	0.45	0.33	0.54	0.23	0.31
Passive (K_{PE})	6.3	6.3	5.4	5.4	12.0	12.0
At Rest (K_{OE})**	0.59		0.63		0.33	

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

** After Woods

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 15.1 may be used.

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.

16 CONSTRUCTION CONCERNS

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

1. Unwatering Excavations

If temporary excavation below the groundwater level is required, the contractor may have difficulty maintaining a stable, unwatered excavation. These difficulties will become more pronounced if excavation to the uneven, sloping bedrock surface is contemplated.

2. RSS Wall Foundations

The performance and appearance of a RSS wall are dependent, in part, on the preparation of the foundation. It is important that RSS foundations be treated with the same care as structure foundations and that they be designed and constructed as recommended in this report.

3. Control of Pile Driving

The uneven bedrock surface will result in piles of varying length being driven. It is important that the field staff responsible for pile driving be alert to this fact and control the driving process so as not to permit pile damage to occur.

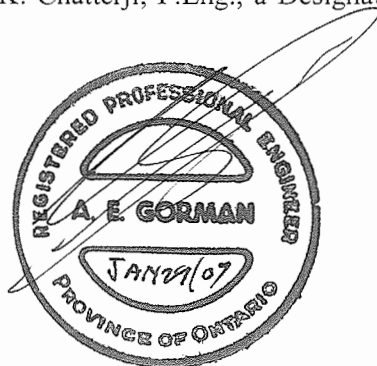
17 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng.

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

Thurber Engineering Ltd.

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



Appendix A
Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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

 Water Level

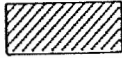

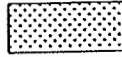


C_{pen} Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.

TERMS		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.				
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No 420-27

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 508.8 E 310 779.0 Muskoka Road Overpass (SBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 16.04.04 - 16.04.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE 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		W _p	W	W _L		
365.0 0.0	GRAVEL and SAND Compact Brown Dry to Moist (FILL)		1	SS	19		365	20 40 60 80 100	○ UNCONFINED + FIELD VANE					
			2	SS	14		364	20 40 60 80 100	● QUICK TRIAXIAL × LAB VANE					
			3	SS	4		363	20 40 60 80 100						
362.8 2.2	SILT, some sand to sandy, trace clay Compact Grey- Brown Moist to Wet		4	SS	20		362	20 40 60 80 100						0 19 75 7
			5	SS	17			20 40 60 80 100						
361.0 4.0	END OF BOREHOLE AT 3.96m. AUGER REFUSAL AT 3.96m PROBABLY ON COBBLES OR BOULDERS. WATER LEVEL IN OPEN BOREHOLE AT 2.4m DEPTH UPON COMPLETION.													

ONTMT4S 420MUSKOKA-1.GPJ 26/06/06

RECORD OF BOREHOLE No 420-29

1 OF 2

METRIC

W.P. 758-93-01 LOCATION N 5 066 519.8 E 310 798.5 Muskoka Road Overpass (SBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 15.04.04 - 15.04.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								20 40 60 80 100				
								20 40 60 80 100				
366.7						○ UNCONFINED	+ FIELD VANE					
0.0	ASPHALT (250mm)					● QUICK TRIAXIAL	× LAB VANE					
0.2	GRAVEL and SAND Compact, Brown, Dry (FILL)		1	SS	29							
366.0												
0.7	SAND, trace gravel Compact to Very Dense Brown Dry to Moist (FILL)		2	SS	29							
			3	SS	55							
364.5												
2.2	SAND, some silt, occasional rootlets and trace organics to 2.9m Compact Brown Moist to Wet		4	SS	18							
			5	SS	22							
			6	SS	24							
361.2												
5.5	SAND with cobbles and boulders Very Dense inferred Brown Moist		7	SS	50/00							
359.7												
7.0	AUGER REFUSAL AT 7.01m. GNEISS BEDROCK Fresh to slightly weathered, massive, grey- white with black blotches, strong		1	RUN								
			2	RUN								

Continued Next Page

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

RECORD OF BOREHOLE No 420-29

2 OF 2

METRIC

W.P. 758-93-01 LOCATION N 5 066 519.8 E 310 798.5 Muskoka Road Overpass (SBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 15.04.04 - 15.04.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
356.6 10.1	END OF BOREHOLE AT 10.1m. BOREHOLE FILLED WITH DRILL WATER UPON COMPLETION OF CORING.	X					356							

ONTMT4S 420MUSKOKA-1.GPJ 26/06/06

RECORD OF BOREHOLE No 420-31

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 522.7 E 310 788.1 Muskoka Road Overpass (SBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 15.04.04 - 15.04.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
365.9													
0.0	GRAVEL and SAND Compact Dark Brown Dry (FILL)		1	SS	24								
365.2													
0.7	SAND, trace gravel Compact Brown Moist to Wet (FILL)		2	SS	12								
364.5													
1.4	Sandy SILT, trace clay, trace rootlets and organics to 2.2m Loose to Compact Dark Brown to Brown Moist to Wet		3	SS	7								
			4	SS	17								
			5	SS	21								
361.9													
4.0	AUGER REFUSAL AT 3.96m. set casing to 4.6m and then cored. GNEISS BEDROCK Fresh to slightly weathered, massive, grey- white with black blotches, strong to very strong. Planar to subplanar joints at 4.7m, 4.8m, 4.9m, 5.1m, 5.5m, 6.6m, 6.7m and 7.1m. Fractured zone with sand infilling from 5.7m to 6.1m.		1	RUN									
			2	RUN									
358.3													
7.6	END OF BOREHOLE AT 7.62m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEVATION(m) 16/04/04 1.3 364.6												

ONTMT4S 420MUSKOKA-1.GPJ 04/08/06

RECORD OF BOREHOLE No 420-34

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 536.5 E 310 796.8 Muskoka Road Overpass (SBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 15.04.04 - 15.04.04 CHECKED BY AEG

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		
366.3														
0.0	GRAVEL and SAND Compact, Dark Brown, Moist (FILL)		1	SS	22		366							
365.6														
0.7	SAND, trace gravel, trace silt Compact to Loose Brown Moist to Wet (FILL)		2	SS	19		365							5 91 5 (SI+CL)
			3	SS	6									
364.1														
2.2	Silty SAND Compact Brown Wet		4	SS	21		364							
			5	SS	27		363							0 78 22 (SI+CL)
362.4														
3.9	SAND cobbles and boulders inferred													
362.0														
4.3	AUGER REFUSAL AT 4.26m AND CASING ADVANCED INTO BEDROCK. commence coring from 4.57m. GNEISS BEDROCK Fresh, massive, grey- white with black blotches, strong to very strong. Subplanar joints from 4.8m to 5.1m.		1	RUN			361							RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=107MPa
			2	RUN			360							RUN 2# TCR=97%, SCR=97%, RQD=97%, UCS=95MPa
358.8							359							
7.5	END OF BOREHOLE AT 7.49m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEVATION (m) 16/04/04 1.4 364.9													

ONTMT4S 420MUSKOKA-1.GPJ 04/08/06

RECORD OF BOREHOLE No 420-38

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 527.3 E 310 806.5 Muskoka Road Overpass (SBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 13.04.04 - 13.04.04 CHECKED BY AEG

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		
366.9														
0.0	ASPHALT (230mm)						367							
0.2	GRAVEL and SAND Compact, Brown, Dry (FILL)		1	SS	28									
366.2														
0.7	SAND, trace gravel Compact Brown Dry (FILL)		2	SS	30		366							
365.5														
1.4	SAND and SILT, trace gravel, trace clay Loose to Compact Brown Moist to Wet		3	SS	8		365							4 53 40 3
			4	SS	26		364							
	occasional cobbles below 2.2m													
			5	SS	17		363							
			6	SS	14		362							0 42 55 3
361.4														
5.5	END OF BOREHOLE AT 5.48m. AUGER REFUSAL AT 5.48m ON PROBABLE BEDROCK OR BOULDERS. WATER LEVEL AT 3.0m ON COMPLETION.													

ONTMT4S 420MUSKOKA-1.GPJ 26/06/06

RECORD OF BOREHOLE No 420-39

2 OF 2

METRIC

W.P. 758-93-01 LOCATION N 5 066 531.0 E 310 810.0 Muskoka Road Overpass (SBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 13.04.04 - 13.04.04 CHECKED BY AEG

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			20	40	60	80	100	w _p	w	w _L		
356.6							357								1		
10.5	END OF BOREHOLE AT 10.49m. BOREHOLE FILLED WITH DRILL WATER UPON COMPLETION OF CORING.														2		

ONTM74S 420MUSKOKA-1.GPJ 26/06/06

RECORD OF BOREHOLE No 420-40

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 557.7 E 310 816.5 Muskoka Road Overpass (SBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 16.04.04 - 16.04.04 CHECKED BY AEG

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		
366.9														
0.0	ASPHALT (250mm)						367							
0.2	GRAVEL and SAND Dense, Brown, Dry (FILL)		1	SS	35									
366.2														
0.7	SAND, trace gravel Dense Brown Dry (FILL)		2	SS	38		366							
365.5														
1.4	SAND, trace gravel, trace silt, occasional cobbles and/or boulders from 1.7m to 2.1m Very Dense to Dense Brown Moist to Wet		3	SS	50/ .076		365							
			4	SS	38		364							
			5	SS	38		363							
363.0														
3.9	SAND with cobbles and boulders Very Dense Brown Moist		6	SS	50/ .100		362							
361.3														
5.6	AUGER REFUSAL AT 5.56m. GNEISS BEDROCK Fresh to slightly weathered, massive, pinkish- white with black bands, strong Vertical joint at 7.2m to 7.5m.		1	RUN			361							
			2	RUN			360							
							359							
358.1			3	RUN										
8.9	END OF BOREHOLE AT 8.86m. BOREHOLE FILLED WITH DRILL WATER UPON COMPLETION OF DRILLING.													

ONTMT4S 420MUSKOKA-1.GPJ 04/08/06

RECORD OF BOREHOLE No 420-41

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 564.3 E 310 821.3 Muskoka Road Overpass SBL ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 14.11.05 - 14.11.05 CHECKED BY AEG

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		
366.9														
0.0	ASPHALT (250mm)						367							
0.2	SAND and GRAVEL Very Dense Brown		1	SS	50/ .150									
366.2	Dry (FILL)													
0.8	SAND, trace gravel, trace silt Dense Brown		2	SS	40		366							4 91 5 (SI+CL)
365.4	Dry (FILL)													
1.5	Sandy SILT, trace gravel, occasional organics and black staining Compact		3	SS	11		365							
364.8	Brown Moist to Wet (FILL)													
364.6														
364.4	SAND, some gravel, occasional cobbles and boulders Very Dense Brown		4	SS	50/ .000		364							
2.5	BOULDER													
363.6			5	SS	50/ .150		363							
3.4	BOULDER													
363.0														
4.0														
362.1														
4.9	GNEISS BEDROCK Fresh to slightly weathered, thinly bedded, pink, with black banding, strong, vertical joint at 7.29m to 7.37m		1	RUN			362							RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=82MPa
			2	RUN			361							RUN 2# TCR=100%, SCR=100%, RQD=95%, UCS=81MPa
							360							
359.4	END OF BOREHOLE AT 7.52m. BOREHOLE OPEN TO BOTTOM UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND PATCHED WITH ASPHALT AND GRAVEL AT SURFACE.												3	
7.5														

ONTMT4S 420MUSKOKA-LGPJ 26/06/06

RECORD OF BOREHOLE No 420-45

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 552.4 E 310 829.4 Muskoka Road Overpass (SBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 27.04.04 - 27.04.04 CHECKED BY AEG

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	
365.7								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	W P	W	W L	
0.0	GRAVEL and SAND Compact Dark Brown to Brown Moist (FILL)							20 40 60 80 100				GR SA SI CL
364.3			SS 1		18							
1.4	Silty SAND, trace gravel Very Dense Brown Wet		SS 2		56							
			SS 3		49							2 66 32 (SI+CL)
	occasional cobbles below 2.9m.		SS 4		53							
361.7												
4.0	AUGER REFUSAL AT 4.01m. GNEISS BEDROCK Fresh, massive, pinkish to grey-white with subhorizontal black banding, strong. Vertical joint at 6.7m to 6.8m. Rubble zone from 4.0m to 4.1m.		RUN1									RUN 1# TCR=96%, SCR=89%, RQD=85%, UCS=75MPa
			RUN2									RUN 2# TCR=98%, SCR=95%, RQD=78%, UCS=60MPa
			RUN3									RUN 3# TCR=100%, SCR=100%, RQD=53%, UCS=84MPa
358.5												
7.2	END OF BOREHOLE AT 7.18m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEVATION(m) 28/04/04 1.1 364.6 18/06/04 1.4 364.3 14/10/04 2.0 363.7 29/11/05 1.2 364.5											

ONTMT4S 420MUSKOKA-1.GPJ 10/08/06

RECORD OF BOREHOLE No 420-46

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 569.5 E 310 827.4 Muskoka Road Overpass (SBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 14.04.04 - 14.04.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
367.0							20	40	60	80	100	W _P	W	W _L		GR	SA	SI	CL
0.0	GRAVEL and SAND Compact, Brown, Dry (FILL)			SS 1	15														
366.3																			
0.7	SAND, trace gravel, trace silt Compact Brown Dry (FILL)			SS 2	14												4	87	9 (SI+CL)
365.6																			
1.4	Silty SAND Compact Brown Moist to Wet			SS 3	12														
				SS 4	25												0	81	19 (SI+CL)
				SS 5	19												0	88	12 (SI+CL)
362.9																			
4.1	SAND, some gravel, occasional cobbles and/or boulders Very Dense Brown Moist			SS 6	50/ .152														
362.0																			
5.0	AUGER REFUSAL AT 5.02m. GNEISS BEDROCK Slightly weathered to fresh, massive, pinkish- white with black blotches, moderately strong to very strong. Biotite schist layer from 5.0m to 5.1m.			RUN1													FI		
																	2		
																	<5		
																	1		
																	1		
																	0		
																	0		
359.9				RUN2A													0		
																	0		
7.0	Very coarse grained, black BIOTITE layer No recovery from 7.5m to 7.9m			SS2B													0		
																	0		
																	0		
358.7																	0		
8.2	BIOTITE SCHIST			RUN3													1		
358.4																			
8.6	END OF BOREHOLE AT 8.55m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEVATION(m) 15/04/04 1.6 365.4 16/04/04 1.5 365.5																		
</																			

ONTM4S 420MUSKOKA-I.GPJ 10/08/06

RECORD OF BOREHOLE No 420-51

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 563.9 E 310 840.5 Muskoka Road Overpass (SBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 28.04.04 - 28.04.04 CHECKED BY AEG

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		
366.5														
0.0	GRAVEL and SAND Compact Dark Brown Moist (FILL)		1	SS	32		366							
365.2														
1.3	Silty SAND Dense Brown Moist to Wet		2	SS	34		365							
364.3														
2.2	SAND, some gravel, trace silt, occasional cobbles and/or boulders Very Dense Brown Wet		3	SS	50/ .100		364							17 80 4 (SI+CL)
362.6							363							
3.9	GNEISS BEDROCK Fresh, massive, pinkish- white with black blotches, strong to very strong. Subvertical joint from 6.1m to 6.2m.		1	RUN			362							RUN 1# TCR=100%, SCR=87%, RQD=73%, UCS=76MPa
			2	RUN			361							RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=98MPa
			3	RUN			360							RUN 3# TCR=100%, SCR=91%, RQD=61%, UCS=113MPa
359.3														
7.2	END OF BOREHOLE AT 7.2m. BOREHOLE FILLED WITH DRILL WATER UPON COMPLETION OF DRILLING.													

ONTM14S 420MUSKOKA-1.GPJ 04/08/06

RECORD OF BOREHOLE No 420-52

1 OF 1

METRIC

W.P. 758-93-01 LOCATION N 5 066 581.2 E 310 847.4 Muskoka Road Overpass (SBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 28.04.04 - 28.04.04 CHECKED BY AEG

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	20 40 60 80 100		
366.9														
0.0	GRAVEL and SAND Compact Brown Moist (FILL)													
365.5			1	SS	15		366							
1.4	SAND, trace gravel, trace silt Dense Brown Moist to Wet		2	SS	33		365							
364.0			3	SS	40									8 89 3 (SI+CL)
2.9	GRAVEL and SAND, trace silt, occasional cobbles and bouldes Dense Brown Wet		4	SS	50/ .152		364							40 54 6 (SI+CL)
363.2														
3.7	END OF BOREHOLE AT 3.7m. AUGER REFUSAL AT 3.7m ON PROBABLE BEDROCK OR BOULDERS. WET CAVE AT 0.86m UPON COMPLETION.													

Appendix B

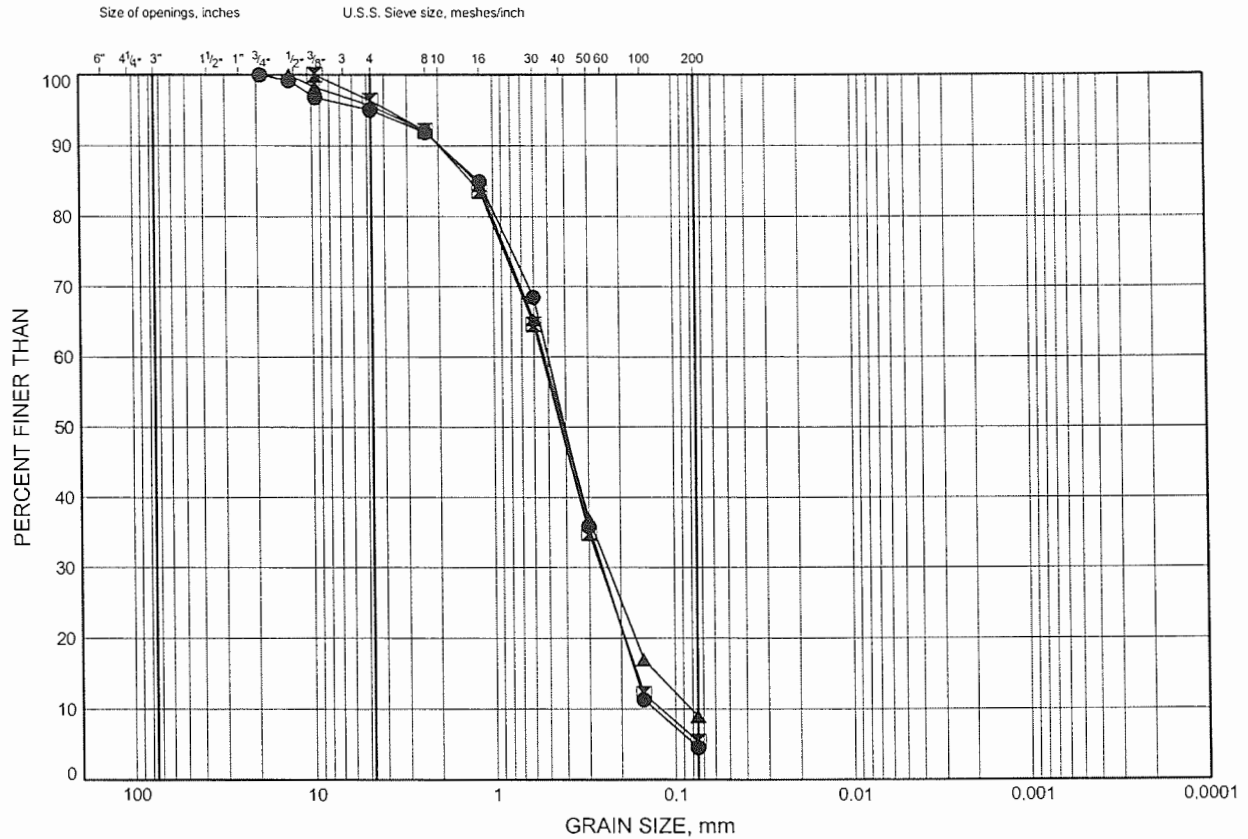
Laboratory Test Results

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B1

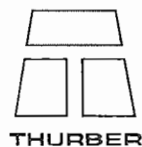
SAND (FILL)



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-34	1.07	365.18
⊠	420-41	1.07	365.86
▲	420-46	1.07	365.88

Date June 2006
Project 759-93-00

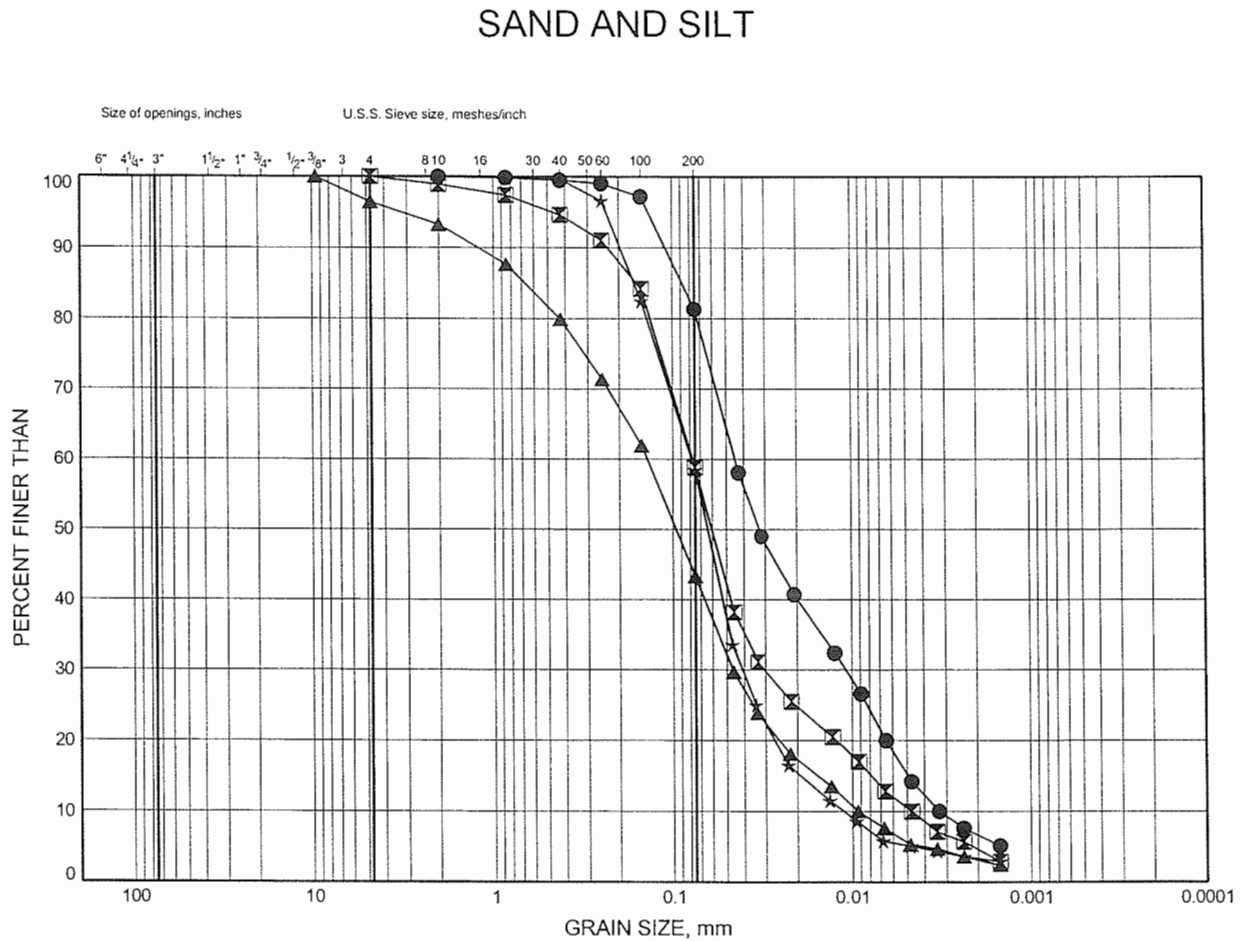


Prep'd JHL
Chkd. MEF

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

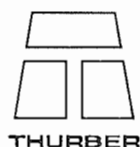
FIGURE B2



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-27	2.59	362.39
⊠	420-31	1.83	364.06
▲	420-38	1.83	365.12
★	420-38	4.88	362.07

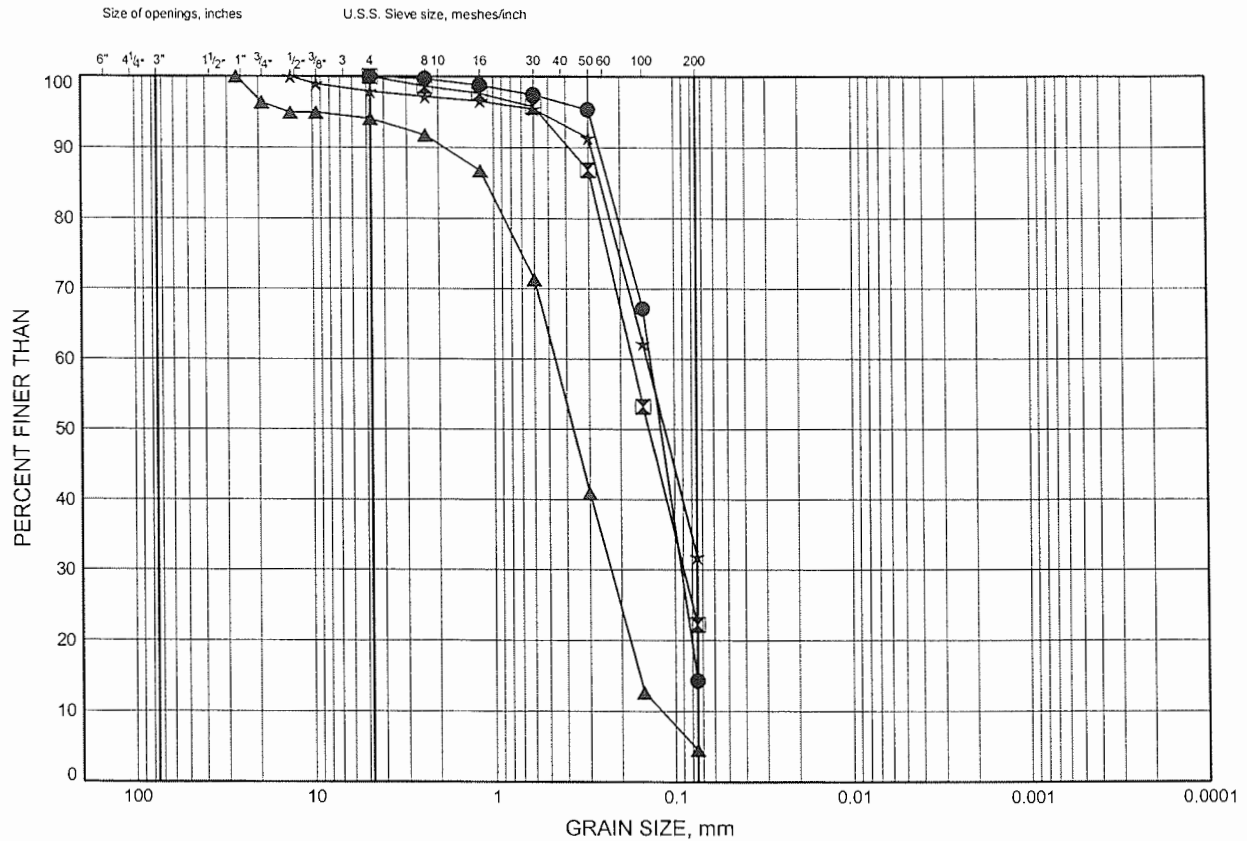
Date June 2006
Project 759-93-00



Prep'd JHL
Chkd. MEF

FIGURE B3a

SAND AND SILTY SAND

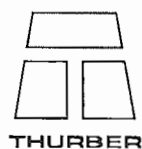


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-29	3.35	363.34
⊗	420-34	3.35	362.90
▲	420-40	3.35	363.58
★	420-45	2.59	363.14

Date June 2006

Project 759-93-00.....



Prep'dJHL.....

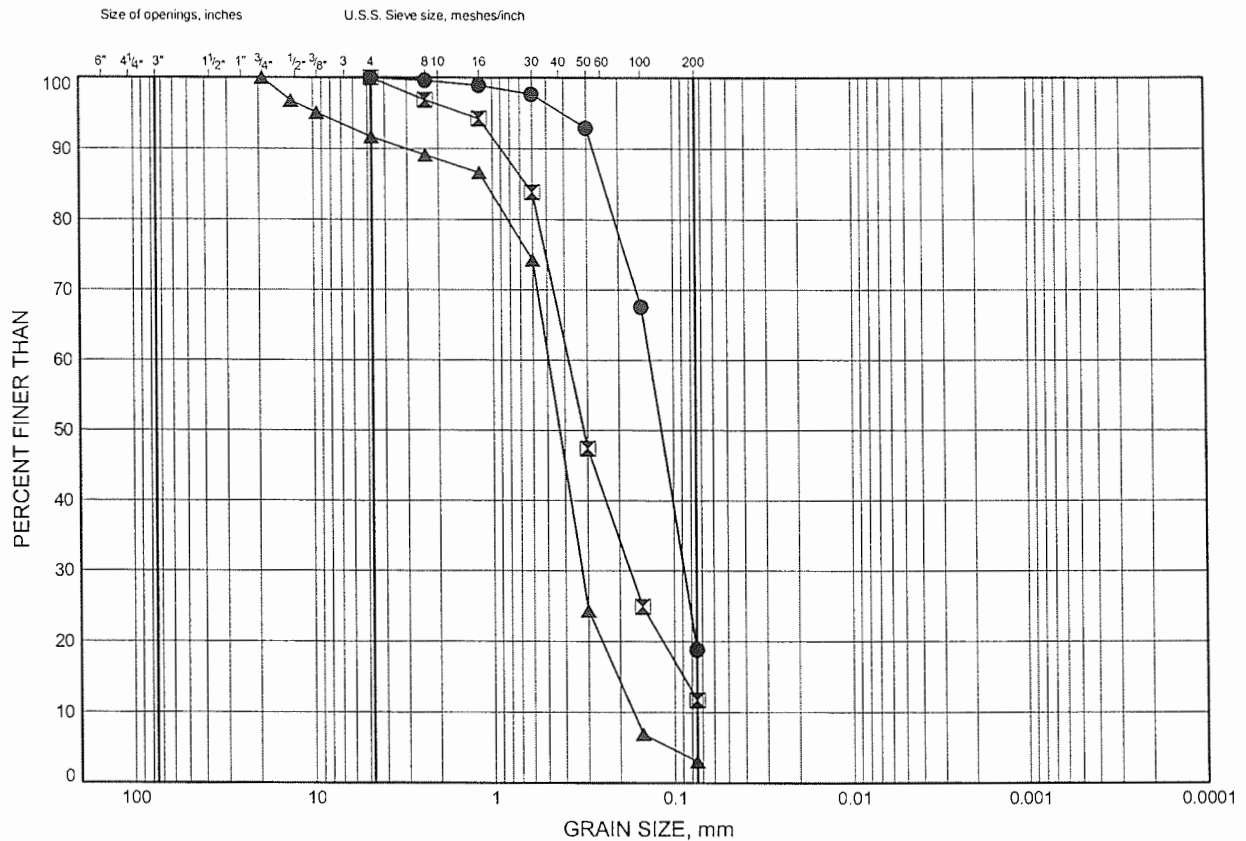
Chkd. MEF

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B3b

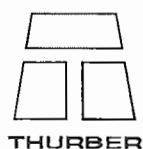
SAND AND SILTY SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-46	2.59	364.36
⊠	420-46	3.35	363.60
▲	420-52	2.59	364.29

Date June 2006
Project 759-93-00

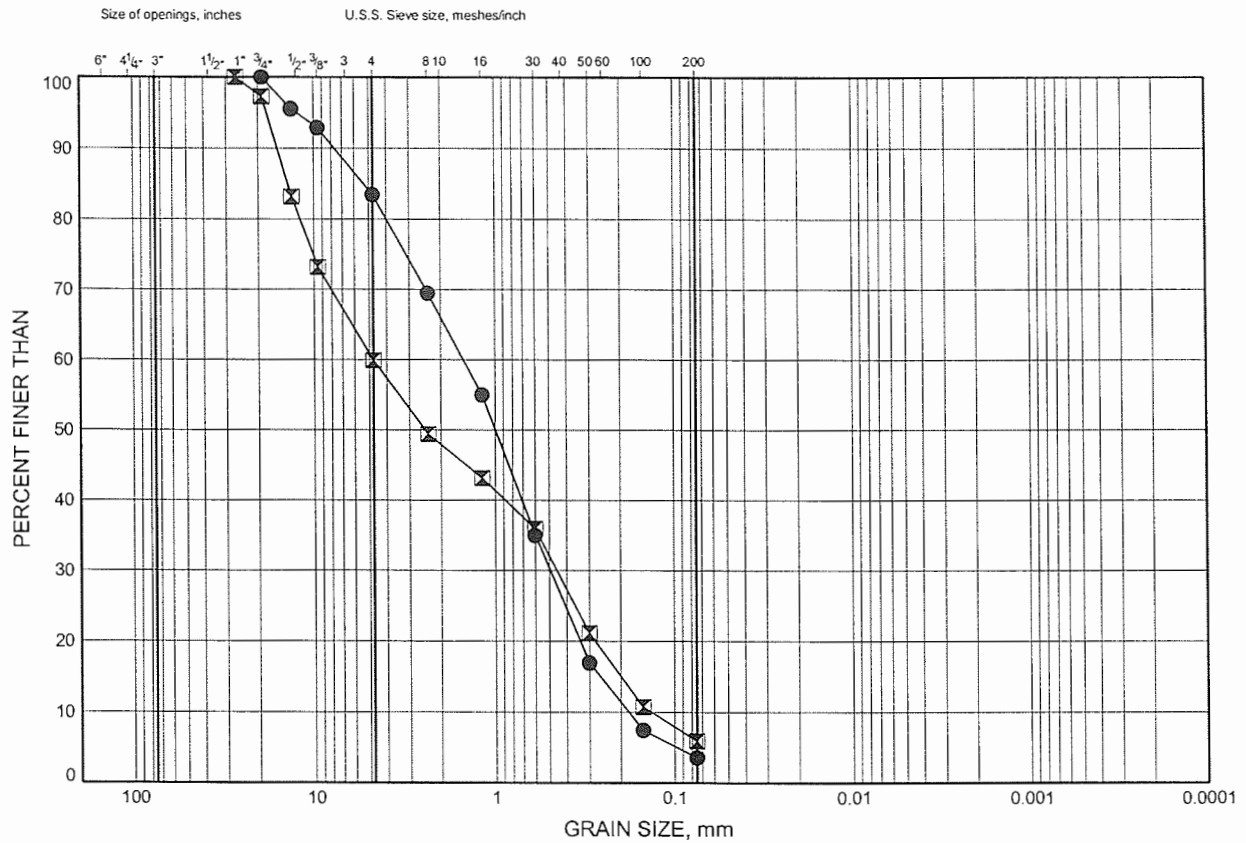


Prep'd JHL
Chkd. MEF

Hwy 11 Four Laning GRAIN SIZE DISTRIBUTION

FIGURE B4

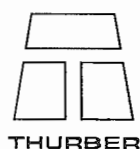
SAND, SOME GRAVEL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-51	2.36	364.17
⊠	420-52	3.20	363.68

Date June 2006
Project 759-93-00



Prep'd JHL
Chkd. MEF

**TABLE B1 - Point Load Test Results
Muskoka Road Overpass SBL**

Depth			Is50	UCS (MPa)
feet	Inches	m		
420-29				
23	8	7.21	3.86	92.70
24	5	7.44	5.14	123.24
25	4	7.72	3.20	76.90
26	0	7.92	3.20	76.90
27	0	8.23	4.52	108.50
28	2	8.59	3.91	93.75
29	1	8.86	2.77	66.36
30	0	9.14	2.59	62.15
31	0	9.45	0.97	23.17
31	9	9.68	5.22	125.35
32	7	9.93	2.98	71.63

Total Rock Core			
Average	Minimum	Maximum	
84	23	125	MPa
Run #	Average		
1	95.65		
2	78.70		

Depth			Is50	UCS (MPa)
feet	Inches	m		
420-31				
15	3	4.65	8.16	195.93
17	0	5.18	6.14	147.47
17	9	5.41	7.90	189.61
18	3	5.56	4.26	102.18
19	4	5.89	4.48	107.44
20	6	6.25	2.94	70.58
21	0	6.40	3.47	83.22
22	2	6.76	2.19	52.67
23	2	7.06	4.56	109.55
24	0	7.32	3.99	95.86

Total Rock Core			
Average	Minimum	Maximum	
115	53	196	MPa
Run #	Average		
1	148.52		
2	82.37		

Depth			Is50	UCS (MPa)
feet	Inches	m		
420-34				
15	4	4.67	4.35	104.28
16	0	4.88	5.31	127.46
17	0	5.18	3.56	85.32
18	0	5.49	4.04	96.91
19	0	5.79	5.18	124.30
20	0	6.10	3.82	91.64
21	2	6.45	4.48	107.44
22	0	6.71	4.87	116.92
22	9	6.93	3.91	93.75
23	2	7.06	3.34	80.06
24	0	7.32	3.47	83.22

Total Rock Core			
Average	Minimum	Maximum	
101	80	127	MPa
Run #	Average		
1	107.65		
2	95.51		

**TABLE B1 - Point Load Test Results
Muskoka Road Overpass SBL**

Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-39								
25	6	7.77	4.30	103.23	<div><div>Total Rock Core</div><div>Average Minimum Maximum</div><div>60 0 103 MPa</div><div>Run # Average</div><div>1 73.74</div><div>2 54.47</div></div>			
26	1	7.95	2.90	69.52				
27	8	8.43	2.02	48.45				
29	10	9.09	1.93	46.35				
30	4	9.25	2.72	65.31				
31	1	9.47	2.19	52.67				
32	2	9.80	0.00	0.00				
32	11	10.03	3.12	74.79				
33	6	10.21	2.59	62.15				
34	1	10.39	3.34	80.06				

Note: Point load test at 9.80 m was performed at hidden joint

Depth		Is50	UCS (MPa)					
feet	Inches							m
420-40								
18	7	5.66	4.83	115.87	<div>Total Rock Core</div> <div>Average Minimum Maximum</div> <div>70 28 116 MPa</div> <div>Run # Average</div> <div>1 64.68</div> <div>2 76.05</div>			
19	6	5.94	1.19	28.44				
20	3	6.17	2.85	68.47				
21	4	6.50	3.16	75.84				
22	6	6.86	1.45	34.76				
23	7	7.19	3.03	72.68				
25	0	7.62	3.25	77.95				
26	0	7.92	2.85	68.47				
27	0	8.23	4.26	102.18				
28	10	8.79	2.46	58.99				

Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-41					Total Rock Core			
16	9	5.11	3.04	72.85	Average	Minimum	Maximum	MPa
17	10	5.44	3.47	83.26	81	71	88	
19	8	5.99	3.69	88.46	Run #	Average		
20	3	6.17	3.47	83.26	1	81.52		
22	0	6.71	3.69	88.46	2	80.83		
24	2	7.37	2.95	70.77				

**TABLE B1 - Point Load Test Results
Muskoka Road Overpass SBL**

Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-42								
20	3	6.17	3.42	82.16	Total Rock Core Average Minimum Maximum 65 45 90 MPa Run # Average 1 70.79 2 58.78			
21	2	6.45	3.73	89.54				
22	1	6.73	2.28	54.78				
23	3	7.09	2.77	66.36				
24	4	7.42	2.55	61.10				
25	1	7.65	1.98	47.40				
25	8	7.82	2.28	54.78				
27	6	8.38	3.03	72.68				
28	6	8.69	3.07	73.74				
29	0	8.84	1.89	45.29				
420-45								
13	6	4.11	3.42	82.16	Total Rock Core Average Minimum Maximum 72 45 121 MPa Run # Average 1 75.49 2 60.46 3 84.10			
14	4	4.37	3.73	89.54				
15	0	4.57	2.28	54.78				
15	10	4.83	2.77	66.36				
16	8	5.08	2.55	61.10				
17	6	5.33	1.98	47.40				
18	6	5.64	2.28	54.78				
19	8	5.99	3.03	72.68				
20	8	6.30	4.03	96.68				
21	8	6.60	5.03	120.68				
22	8	6.91	3.07	73.74				
23	6	7.16	1.89	45.29				
420-46								
17	3	5.26	7.86	188.55	Total Rock Core Average Minimum Maximum 89 0 189 MPa Run # Average 1 138.41 2A 23.17 2B 0.00			
18	1	5.51	4.35	104.28				
19	3	5.87	6.10	146.42				
20	0	6.10	5.71	136.94				
21	0	6.40	4.83	115.87				
21	10	6.65	0.97	23.17				
23	0	7.01	0.00	0.00				
23	10	7.26	0.00	0.00				

Note: Point load test at 7.01 and 7.26 m was performed on Biotite layer

**TABLE B1 - Point Load Test Results
Muskoka Road Overpass SBL**

feet	Depth		Is50	UCS (MPa)				
	Inches	m						
420-51								
13	6	4.11	2.51	60.13				
14	5	4.39	3.89	93.31				
15	8	4.78	4.32	103.68				
17	0	5.18	4.23	101.60				
18	6	5.64	3.89	93.31				
19	11	6.07	3.89	93.31				
21	0	6.40	4.67	111.97				
22	0	6.71	4.97	119.23				
23	5	7.14	4.58	109.90				
					Total Rock Core			
					Average	Minimum	Maximum	
					98	60	119	MPa
					Run #	Average		
					1	76.72		
					2	97.98		
					3	113.70		

Appendix C

Factual Information from Golder's Report

PROJECT 991-1193		RECORD OF BOREHOLE No 7-1		1 OF 1		METRIC	
W.P. 335-98-00		LOCATION N 5066458.88; E 310835		ORIGINATED BY SB			
DIST 54 HWY 11		BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS		COMPILED BY DKB			
DATUM GEODETIC		DATE Feb 29/00		CHECKED BY ASP			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE 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					WATER CONTENT (%)				
								<div><div></div><div>20406080100</div></div>					<div><div></div><div>102030</div></div>				

364.77	GROUND SURFACE																
0.00 364.27	Topsoil		1	SS	2		364										
0.30 364.08	Sandy Silt, trace clay and organics Very loose Brown Wet organic content = 5.5%		2	SS	22												
0.69	Sand, trace to some silt Compact Brown Wet		3	SS	22												
362.56			4	SS	15												
2.21	Silt and Sand, trace clay Compact Brown Wet		5	SS	55/05												
361.80	Sand, some gravel, trace silt, occ. cobbles and/or boulders Very dense Brown Wet END OF BOREHOLE Refusal to further auger penetration; probable bedrock Note: Water level measured in open borehole at 0.8m depth (El. 364.0m) upon completion of drilling. Easting co-ordinate accurate to nearest metre.																
3.08																	

PROJECT 991-1193			RECORD OF BOREHOLE No 7-2			1 OF 1			METRIC				
W.P. 335-98-00			LOCATION N 5066487.78; E 310829.15			ORIGINATED BY SB							
DIST 54 HWY 11			BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS			COMPILED BY DKE							
DATUM GEODETIC			DATE Feb 29/00			CHECKED BY ASP							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GRAIN SIZE DISTRIBUTION (%)
364.42	GROUND SURFACE												GR SA SI CL
0.00	Topsoil		1	SS	2		364						
0.15	Sandy Silt, trace clay and organics Very loose												
363.73	Brown												
0.69	Wet												
362.97	Sand, trace silt Compact		2	SS	25		363						
1.45	Brown												
	Wet												
361.45	Silt and Sand, trace clay Compact to loose		3	SS	19		362						0 44 49 7
	Brown												
	Wet		4	SS	9		361						
361.45	Sand, some gravel to gravelly sand, trace silt, occ. cobbles and/or boulders		5	SS	21		360						16 74 10 0
2.97	Compact to very dense												
	Brown		6	SS	44		359						
	Wet												
359.19			7	SS	53		358						
5.23	Slightly weathered to fresh, grey-white with dark grey blotches, moderately jointed, coarse-grained, strong BIOTITE GNEISS.						357						
							356						
	Bedrock cored from 5.23m to 9.32m depth.												
	For bedrock coring details refer to Record of Drillhole 7-2												
355.10													
9.32	END OF HOLE												
	Note: 1. Water level measured in piezometer at ground surface upon completion of installation. 2. Water level measured in piezometer at 0.2m depth (El. 364.2m) on March 8, 2000. 3. Water level measured in piezometer at 0.1m depth (El. 364.3m) on March 26, 2000.												

ON MOT 991-1193 GPJ ON MOT.GDT 24/4/00

PROJECT: 991-1193

RECORD OF DRILLHOLE: 7-2

SHEET 1 OF 1

LOCATION: N 5066487.78; E 310829.15

DRILLING DATE: Feb.29/00

DATUM: Geodetic

INCLINATION: -90°

AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	COLOUR FLUSH RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	BC-BROKEN CORE MB-MECH BREAK B-BEDDING	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE		359.19											
		Slightly weathered, grey-white with dark grey blotches, moderately jointed, coarse grained, strong BIOTITE GNEISS.		5.23											
6		Becomes dark brown biotite rich between 7.55m-7.75m and 7.8m-8.0m.			1	0.5	100								
7					2	0.6	100								
8					3	0.6	100								
9															
10		END OF HOLE		355.10											
11				9.32											
12															
13															
14															
15															

DRILLHOLE 1193 ROCK GPJ GLDR CAN GDT 24/400 PS

DEPTH SCALE

1:50



LOGGED: SB

CHECKED: PD

PROJECT 991-1193			RECORD OF BOREHOLE No 7-3			1 OF 1			METRIC																				
W.P. 335-98-00			LOCATION N 5066559.56; E 310803.80			ORIGINATED BY SB																							
DIST 54 HWY 11			BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS			COMPILED BY DKB																							
DATUM GEODETIC			DATE Feb.29/00			CHECKED BY ASP																							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			WATER CONTENT (%)			SHEAR STRENGTH kPa			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																								
366.14	GROUND SURFACE																												
0.00	Sand and Gravel, trace silt Compact to dense Brown Moist (Fill)		1	SS	22																								
364.69			2	SS	36																								
1.45	Silt and Sand, trace gravel, trace clay and organics																												
364.31																													
1.83	Loose Brown Wet non-plastic Atterberg limit test result for Sample 3A organic content=3.1%		3	SS	8																								
363.09	Sand, some silt, trace gravel																												
3.05	Dense Brown Moist Slightly to moderately weathered, gray and white-brown with black blotches, foliated (30°), moderately jointed, coarse to very coarse grained, medium strong GNEISS.		4	SS	34																								
360.04	Bedrock cored from 3.05m to 6.10m depth. For bedrock coring details refer to Record of Drillhole 7-3																												
6.10	END OF HOLE Note: Open borehole dry upon completion of drilling.																												

PROJECT: 991-1193

RECORD OF DRILLHOLE: 7-3

SHEET 1 OF 1

LOCATION: N 5066559.56; E 310803.80

DRILLING DATE: Feb.29/00

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: --

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH BREAK SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED										DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
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DEPTH SCALE

1:50



LOGGED: SB

CHECKED: PD

DRILLHOLE 1193 ROCK GPJ GLDR CAN GDT 24/4/00 PS

PROJECT <u>991-1193</u>			RECORD OF BOREHOLE No 7-4			1 OF 1			METRIC		
W.P. <u>335-98-00</u>			LOCATION <u>N 5066579.07, E 310794</u>			ORIGINATED BY <u>SS</u>					
DIST <u>54</u> HWY <u>11</u>			BOREHOLE TYPE <u>108mm I.D. HOLLOW STEM AUGERS</u>			COMPILED BY <u>DKB</u>					
DATUM <u>GEODETIC</u>			DATE <u>Feb.29/00</u>			CHECKED BY <u>ASP</u>					

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED					WATER CONTENT (%)								
						20	40	60	80	100	20	40	60	80	100	10	20	30			
367.00	GROUND SURFACE																				
0.00	Sandy silt, trace clay and organics/decaying wood matter Loose		1	SS	4	47															
366.31	Blackish brown Moist																				
0.69	Sand, some gravel, trace silt, occ. cobbles and/or boulders Very dense Brown Wet		2	SS	59																
365.26			3	SS	100/03																
1.74	END OF BOREHOLE Refusal to further auger penetration; probable bedrock Note: Water level measured in open borehole at 0.6m depth (El. 366.4m) upon completion of drilling. Easting co-ordinate accurate to nearest metre.																				

ON_MOT_991-1193.GPJ ON_MOT_GDT_26/4/00

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Piles	Caissons	Footing on Native Soil	Footing on Engineered Fill	Footings on Bedrock
Abutments	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by seating piles on bedrock. ii. Comparatively short abutment stem. iii. Relatively short pile lengths required since bedrock is at relatively shallow depth. iv. Will allow for the construction of an integral abutment structure. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to footings. <p>RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High bearing resistances available on bedrock. <p>Disadvantages</p> <ul style="list-style-type: none"> i. Difficulties in obtaining a seal below the liner to pour concrete in dry conditions. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> a. Lower unit cost compared to pile foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower bearing resistance ii. An integral abutment design is not an available option iii. Comparatively longer abutment stem. iv. Possible dewatering requirements 	<p>Advantages</p> <ul style="list-style-type: none"> i. Lower unit cost compared to piles ii. Shorter abutment stem possible. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower bearing resistance ii. An integral abutment design is not an available option iii. Cost of constructing engineered fill iv. Possible dewatering requirements. 	<p>Advantages</p> <ul style="list-style-type: none"> i. High geotechnical resistance available <p>Disadvantages</p> <ul style="list-style-type: none"> i. Sloping bedrock surface may be encountered. ii. Relatively long abutment stems will be required if footings are founded directly on bedrock. iii. Difficulties with excavation and groundwater control. iv. Requires mass concrete fill to raise abutment footings to desired founding elevation, especially at the south abutment. <p>NOT RECOMMENDED</p>

Muskoka Road Overpass SBL
Highway 11 Burk's Falls to South River

Appendix E

Special Provisions

Muskoka Road Overpass SBL
Highway 11 Burk's Falls to South River

The following Special Provisions are referenced in this report:

110F13

105S10

Amendment to OPSS 206, December 1993

902S01

903S01

The suggested wording for the modification of OPSS 501 is as follows:

501.08.02 Method A shall be replaced by the following:

5.0.08.02 Method a

Granular materials shall be compacted to 100% of the maximum dry density and earth materials shall be compacted to 100% of the maximum dry density.

Suggested text for a NSSP on Pile Installation should contain the following:

“The soil overlying the bedrock contains cobbles and boulders, particularly below Elevation 364. The presence of cobbles and boulders will potentially have an impact on the installation of piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- The need to provide protection to the pile tips in the form of rock points*
- The cobbles and boulders may impede the driving of the piles resulting in more arduous driving to reach bedrock*
- 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*
- 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*

Muskoka Road Overpass SBL
Highway 11 Burk's Falls to South River

Appendix F

Computer Output

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 Aug 4, 2006
 Muskoka Road SBL South Approach
 Earth Fill

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Earth Fill	20	31	1
Road Fill	22	33	1
Sand	21	31	1
Sand and Silt	21	30	1

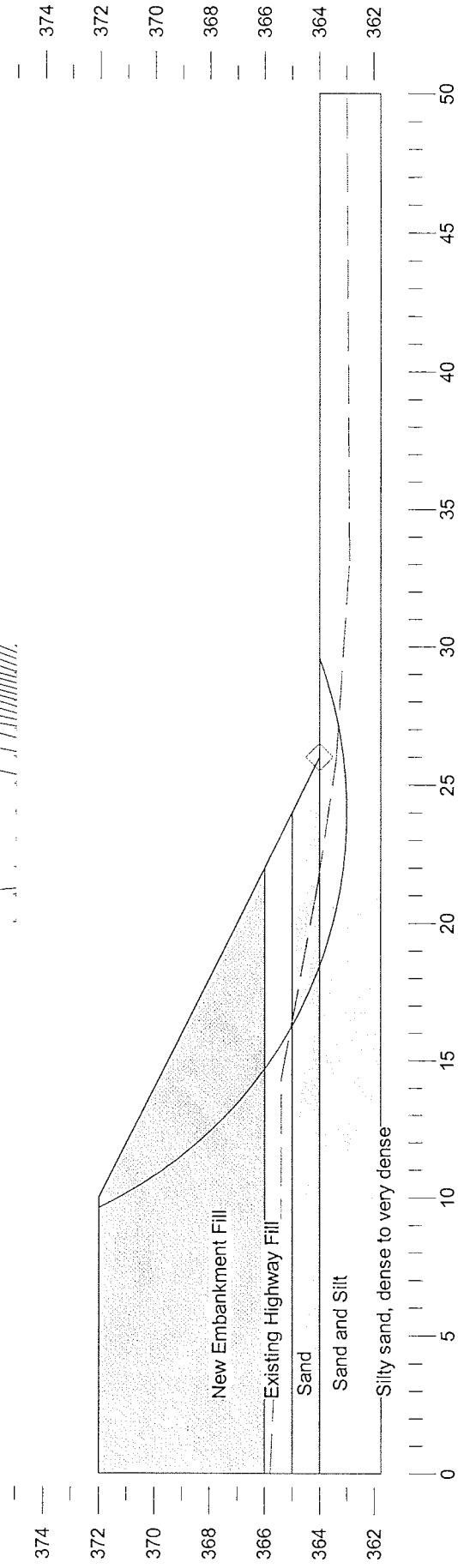
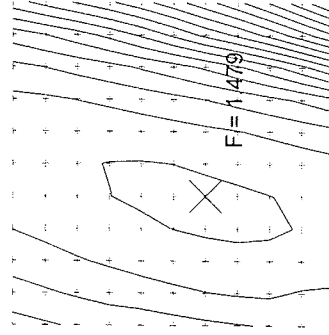


Figure F1

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 Aug 4, 2006
 Muskoka Road SBL South Approach
 Earth Fill, Seismic

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Earth Fill	20	31	1
Road Fill	22	33	1
Sand	21	31	1
Sand and Silt	21	30	1

Seismic coefficient = 0.08

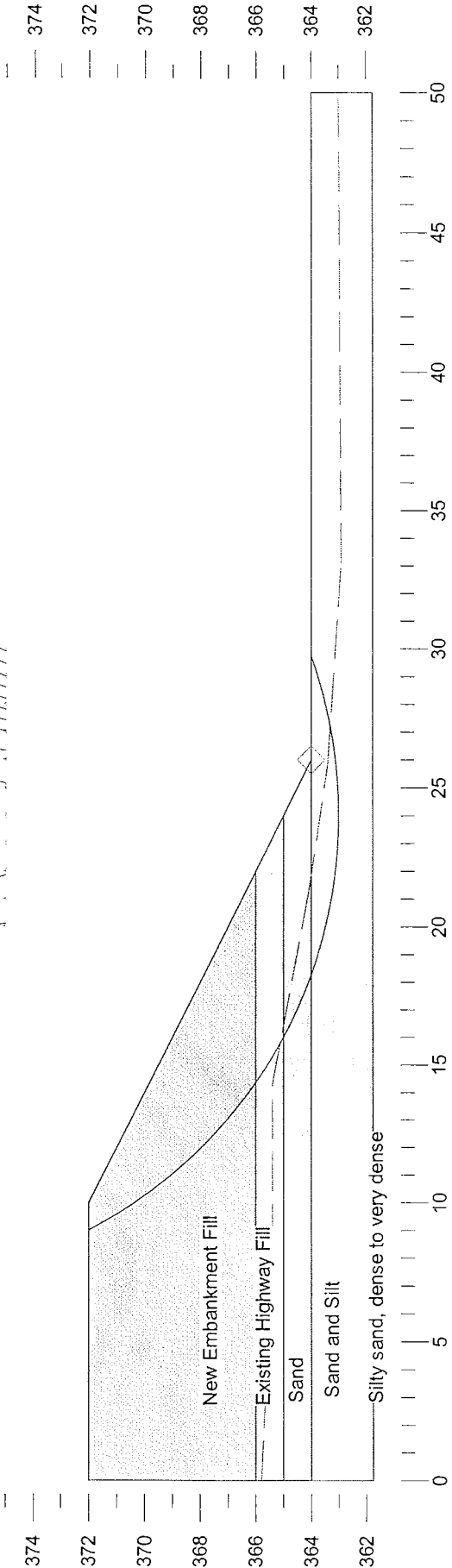
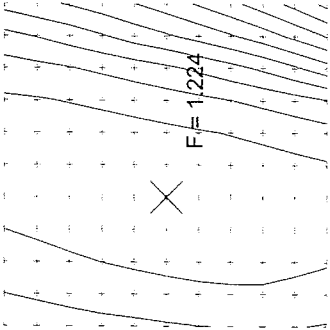


Figure F2

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 Aug 4, 2006
 Muskoka Road SBL South Approach
 Rock Fill

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Rock Fill	19	42	1
Road Fill	22	33	1
Sand	21	31	1
Sand and Silt	21	30	1

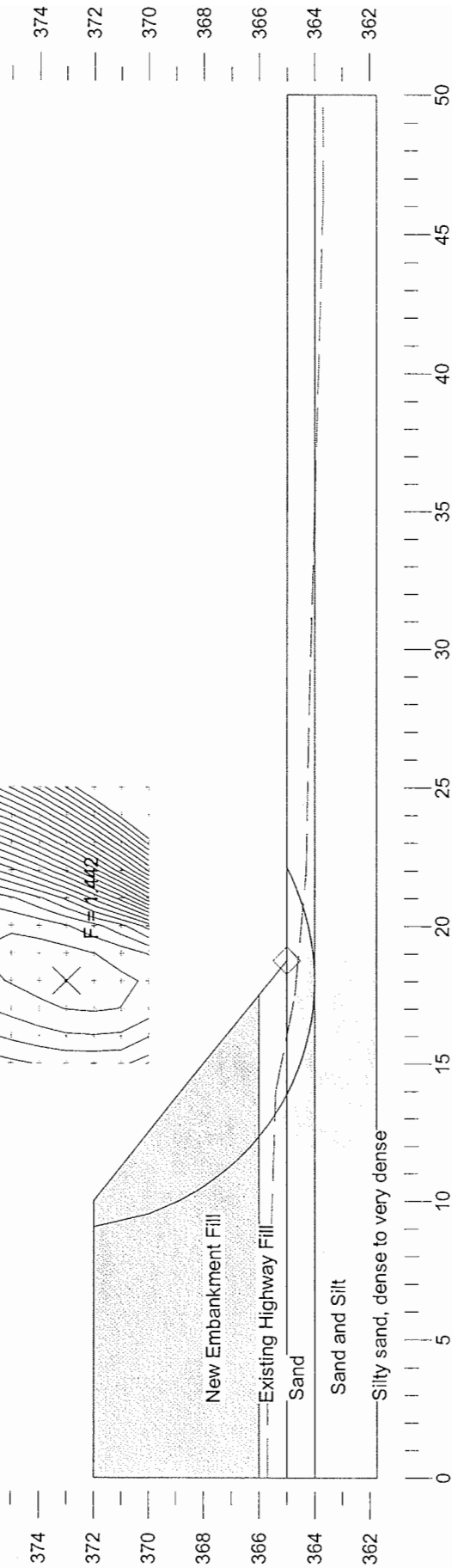
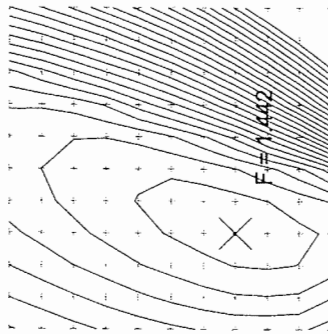


Figure F3

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 Aug 4, 2006
 Muskoka Road SBL South Approach
 Rock Fill, Seismic

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Rock Fill	19	42	1
Road Fill	22	33	1
Sand	21	31	1
Sand and Silt	21	30	1

Seismic coefficient = 0.08

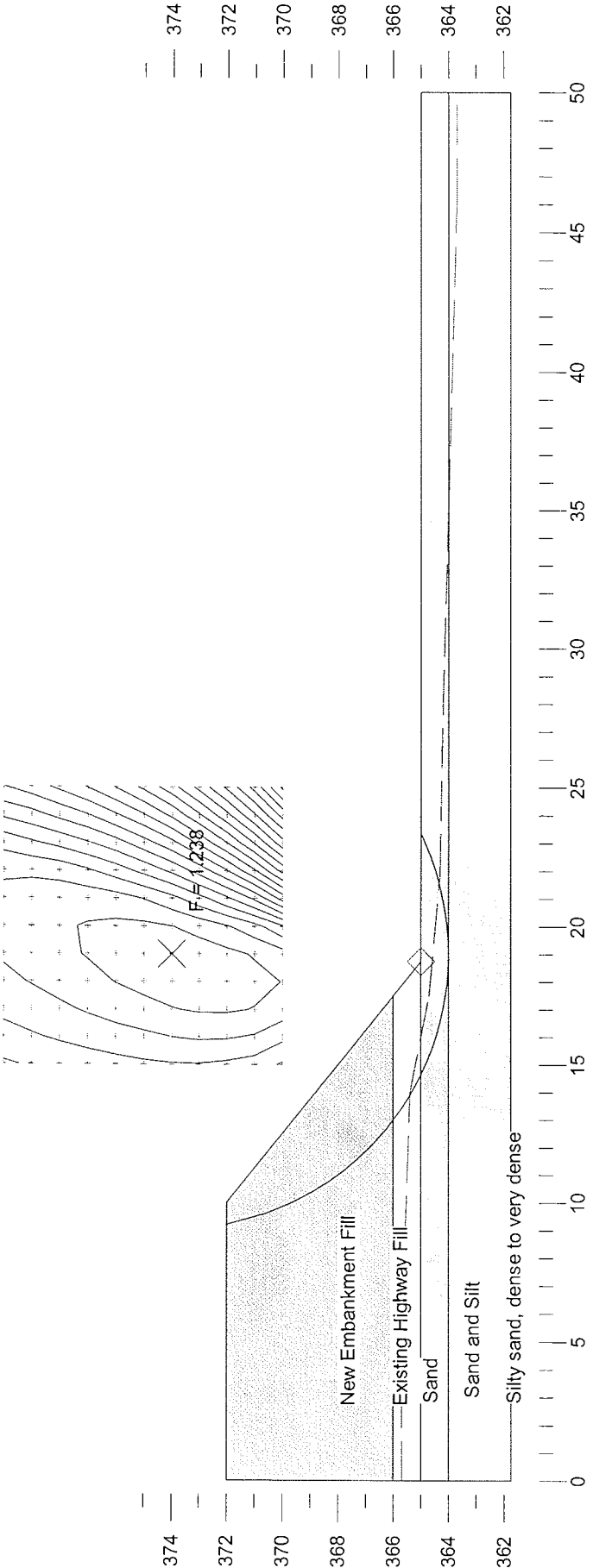


Figure F4

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 Aug 4, 2006
 Muskoka Road SBL North Approach
 Earth Fill

	Gamma	C	Phi	Piezo
	kN/m3	kPa	deg	Surf.
Earth Fill	20	0	31	1
Road Fill	22	0	33	1
Sand and Silt	21	0	30	1

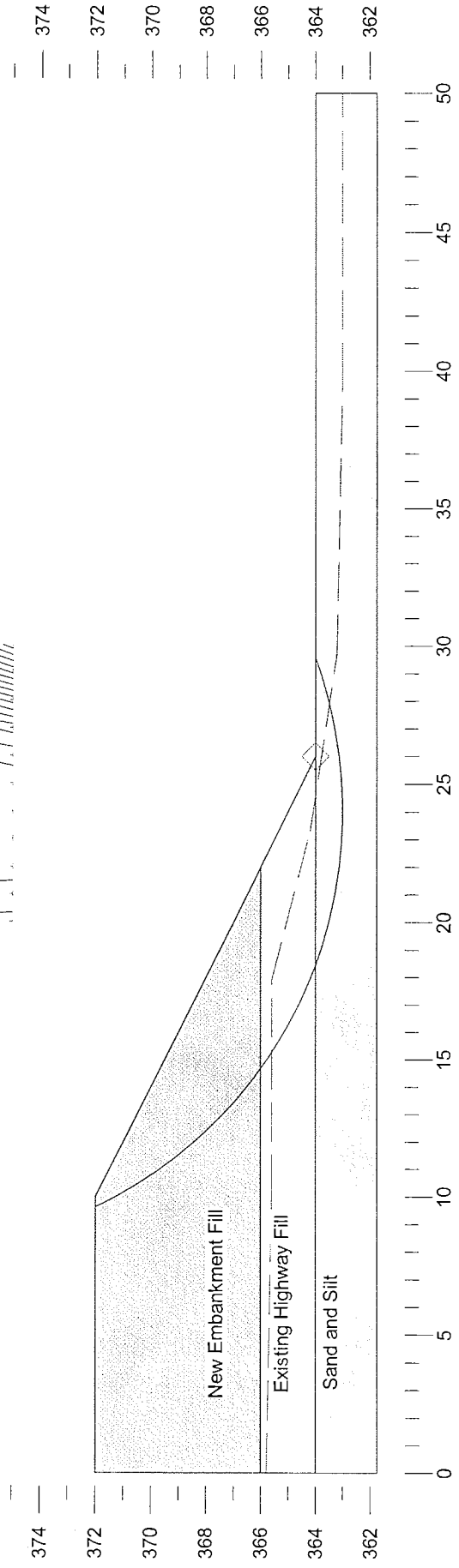
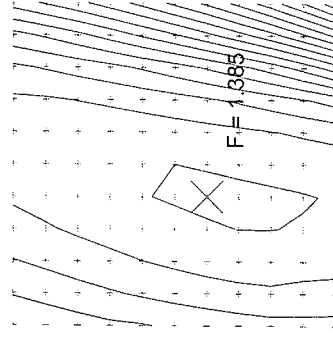


Figure F5

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 Aug 4, 2006
 Muskoka Road SBL North Approach
 Earth Fill, Seismic

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Earth Fill	20	31	1
Road Fill	22	33	1
Sand and Silt	21	30	1

Seismic coefficient = 0.08

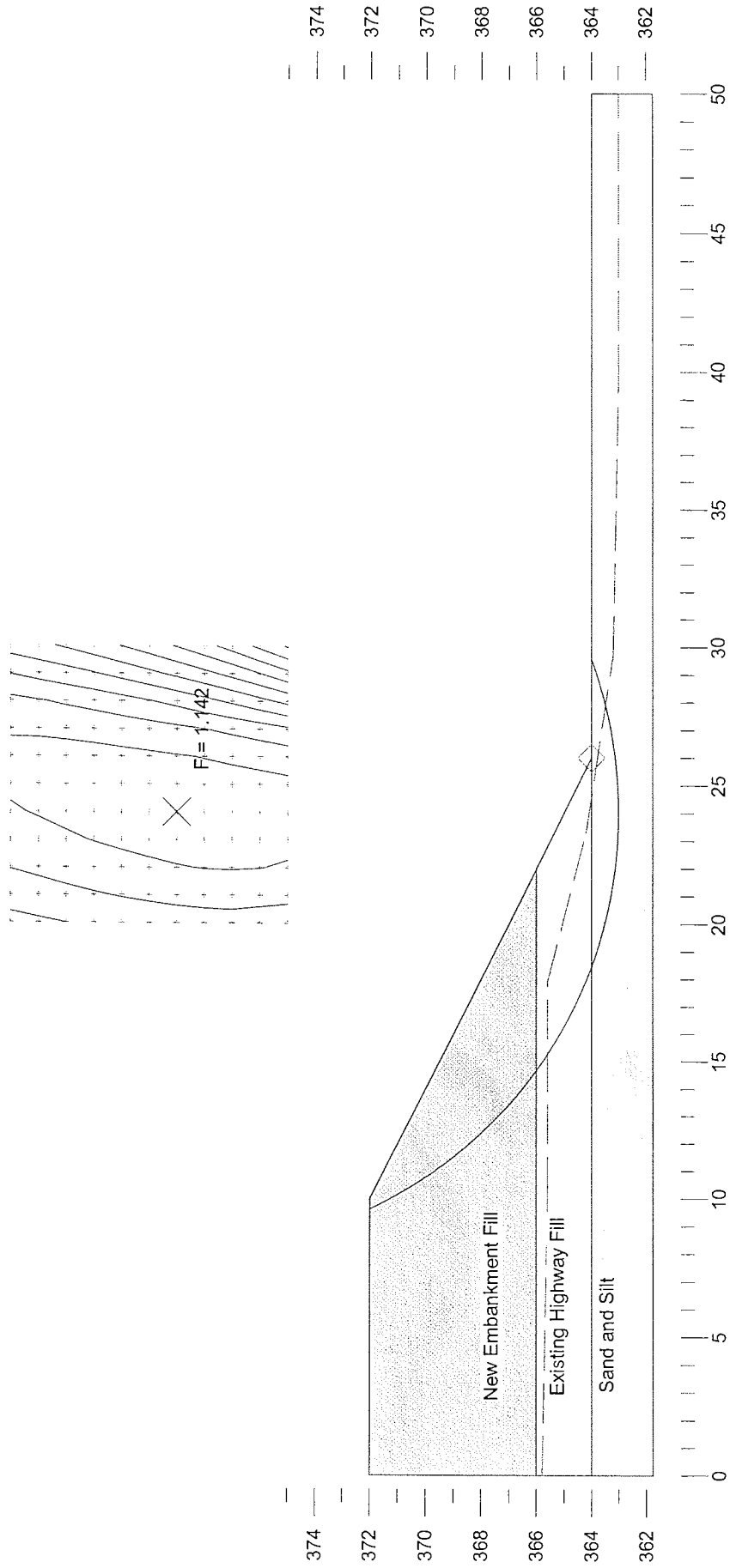


Figure F6

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 Aug 4, 2006
 Muskoka Road SBL North Approach
 Rock Fill

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Rock Fill	19	42	1
Road Fill	22	33	1
Sand and Silt	21	30	1

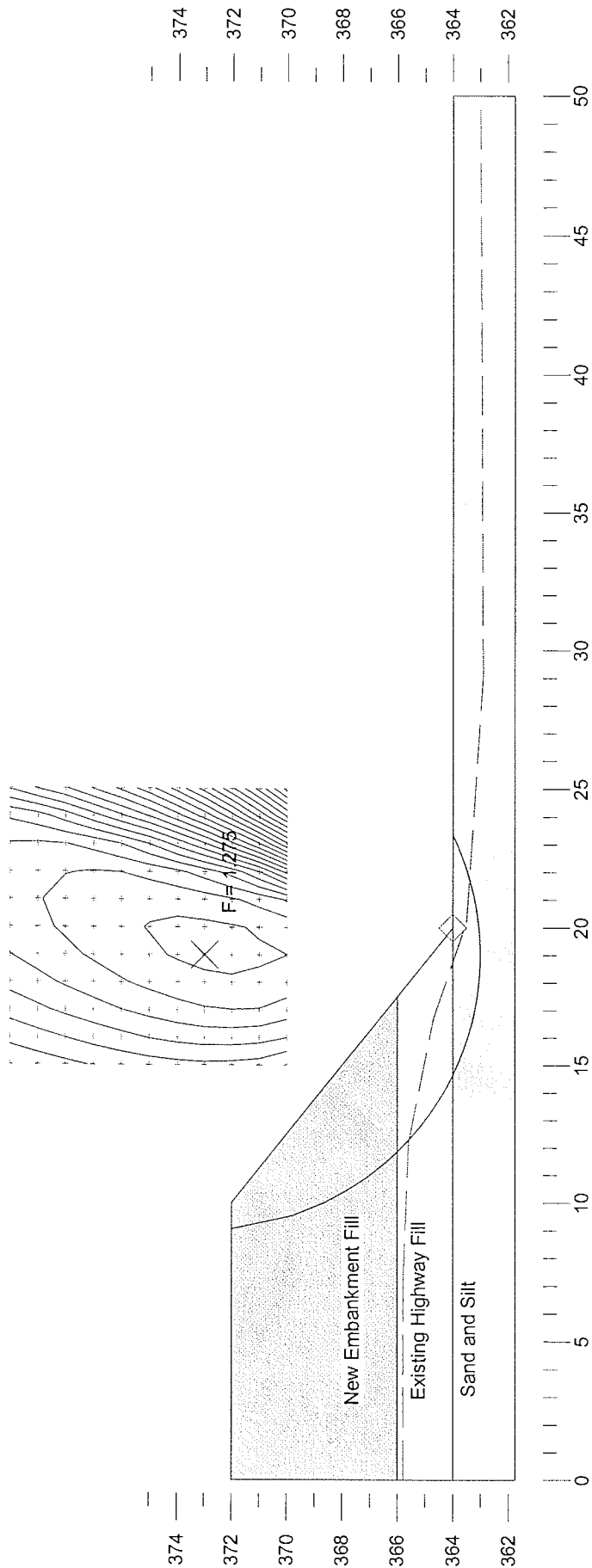


Figure F7

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 Aug 4, 2006
 Muskoka Road SBL North Approach
 Rock Fill, Srismic

	Gamma	C	Phi	Piezo
	kN/m3	kPa	deg	Surf.
Rock Fill	19	0	42	1
Road Fill	22	0	33	1
Sand and Silt	21	0	30	1

Seismic coefficient = 0.08

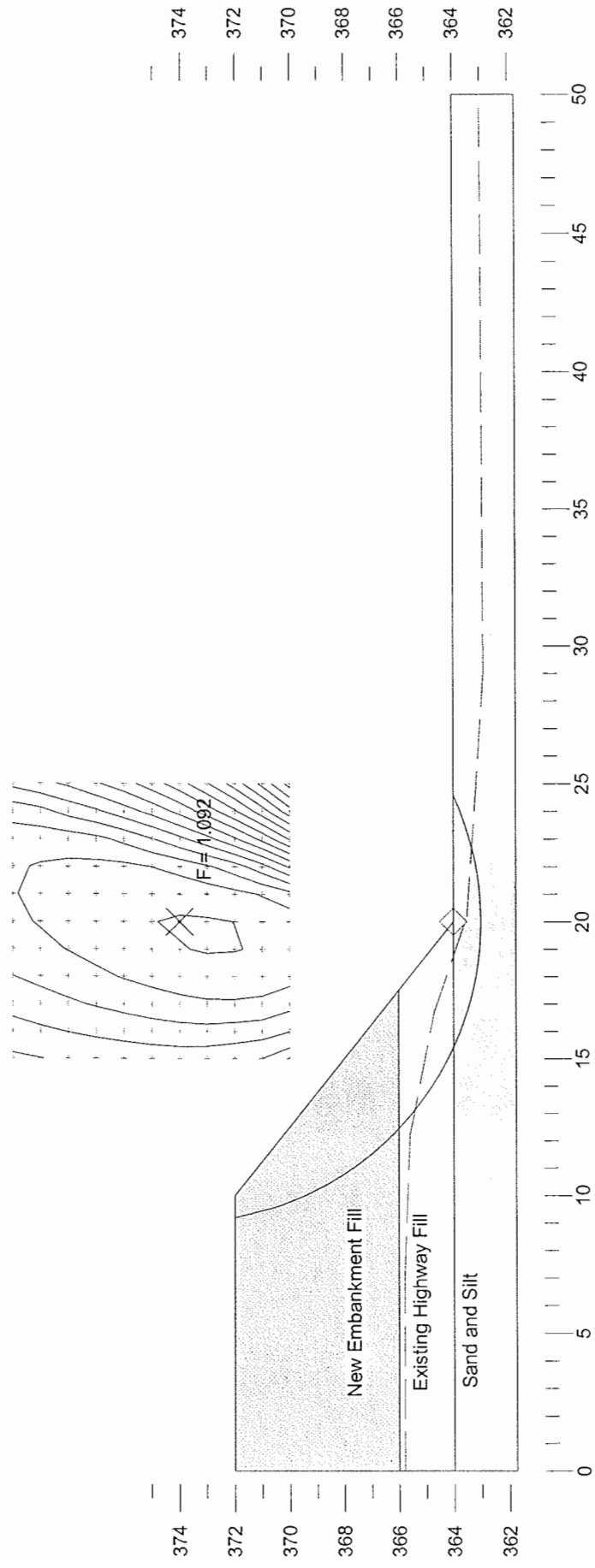
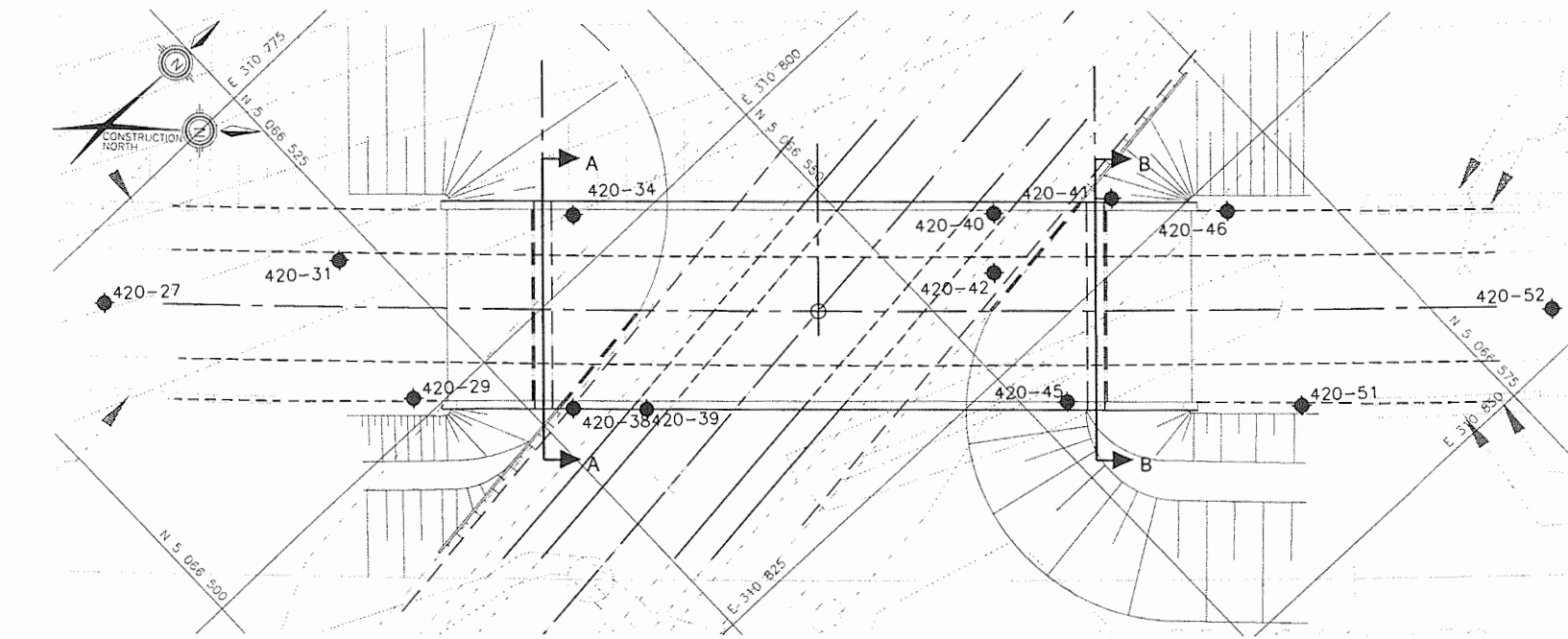


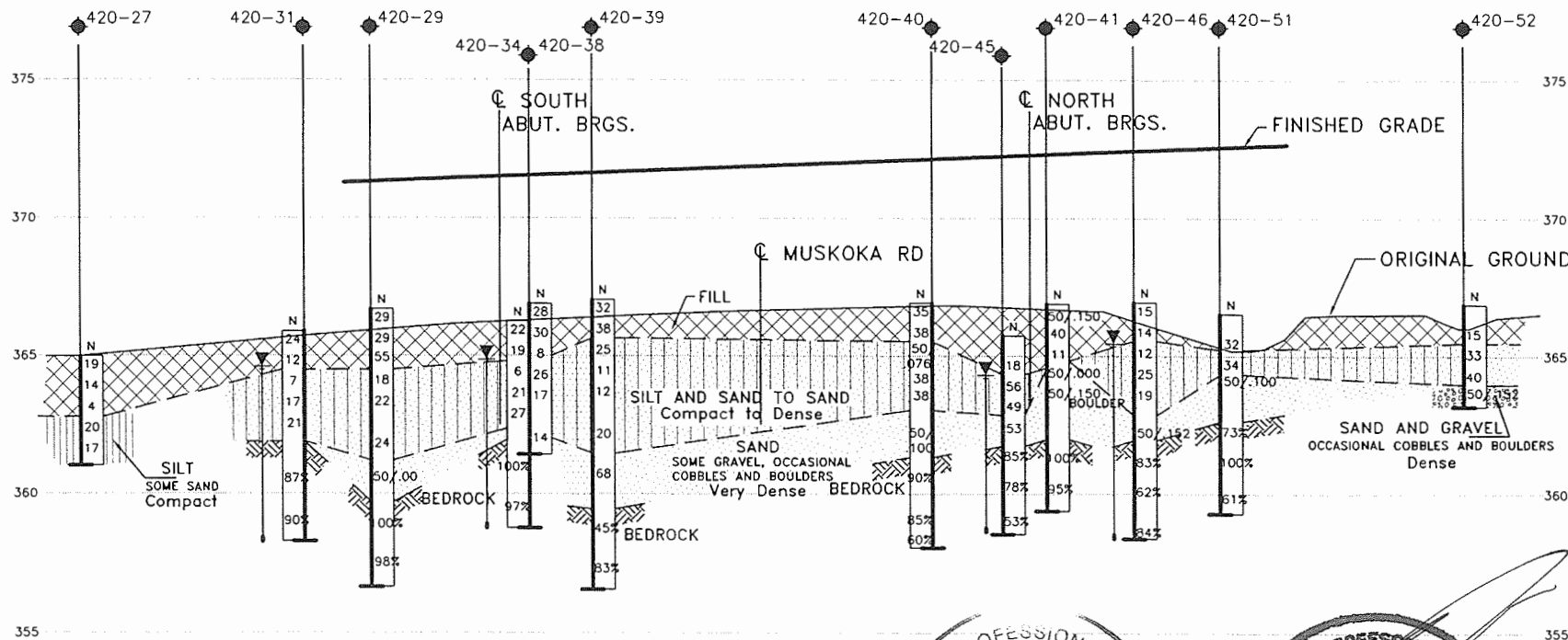
Figure F8

Appendix G

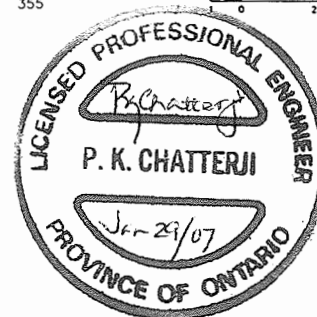
Drawings



PLAN
 0 2.5 5 10m
 HOR: 1:250
 VERT: 1:125



PROFILE @ HWY 11 SBL
 0 2.5 5 10m
 HOR: 1:250
 VERT: 1:125



DRAWING NOT TO BE SCALED
 100 mm ON ORIGINAL DRAWING

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

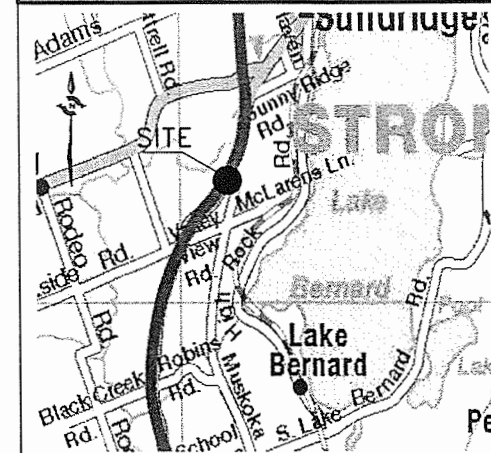
CONT No
 WP No 758-93-01

MUSKOKA ROAD
 OVERPASS SBL

BOREHOLE LOCATIONS AND SOIL STRATA



THURBER ENGINEERING LTD.
 THURBER



KEYPLAN

LEGEND

- BoreHole by THURBER
- ⊕ Dynamic Cone Penetration Test (cone)
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60' Cone, 475J/blow)
- PH Pressure, Hydraulic
- Water Level
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
420-27	365.0	5 066 508.8	310 779.0
420-29	366.7	5 066 519.8	310 798.5
420-31	365.9	5 066 522.7	310 788.1
420-34	366.3	5 066 536.5	310 796.8
420-38	366.9	5 066 527.3	310 806.5
420-39	367.1	5 066 531.0	310 810.0
420-40	366.9	5 066 557.7	310 816.5
420-41	366.9	5 066 564.3	310 821.3
420-42	367.0	5 066 554.9	310 819.5
420-45	365.7	5 066 552.4	310 829.4
420-46	367.0	5 066 569.5	310 827.4
420-51	366.5	5 066 563.9	310 840.5
420-52	366.9	5 066 581.1	310 847.4

NOTE-

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

DATE	BY	DESCRIPTION
DESIGN AEG	CHK A.H.	CODE CHBDC 2000 [LOAD CL-625-001] DATE MAY 2006
DRAWN JHL	CHK R.D.	SITE 44-420 STRUCT [SCHEME] DWG P1