

**FOUNDATION INVESTIGATION AND DESIGN REPORT
MUSKOKA ROAD OVERPASS NBL
HIGHWAY 11 BURK'S FALLS TO SOUTH RIVER
G.W.P. 759-93-00, W.P. 758-93-02, SITE: 44-420/1**

Geocres Number: 31E - 257

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 NBL 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 additional boreholes were 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 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 and a swampy lowland area with occasional mature trees exists on the east side of the highway where a commercial building structure previously existed.

The east edge of the south abutment foundation encroaches into the existing Highway 11 embankment fill.

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 were carried out between the period of November 15 to 16, 2005 for the present investigation and the periods of April 28 to May 03 and August 16 to 25, 2004 for the previous investigation. Six boreholes numbered 420-01, 420-09, 420-12, 420-15, 420-19 and 420-26 pertaining to the single-span structure and approach embankments were drilled to depths ranging from 3.0 m to 10.9 m. Twelve additional boreholes numbered 420-02, 420-06, 420-08, 420-13, 420-16, 420-18 and 420-20 to 25 were drilled in the previous investigation for the three-span structure design to depths ranging from 2.8 m to 11.6 m. The approximate locations of all of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

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. Thurber obtained utility clearances prior to drilling.

A combination of hollow-stem auger drilling techniques and casing and washboring methods were used to advance the boreholes. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. In some boreholes auger refusal was observed and diamond coring was required to extend some of these boreholes through cobbles and boulders and into bedrock. The boreholes at each abutment were advanced 2.2 m to 5.1 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 25 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 generally caved upon removal of the augers or were grouted with bentonite.

Table 3.1 – Piezometer Installation Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
420-12 South Abutment	8.5/355.6	Piezometer with 1.5 m slotted screen installed with sand filter to 6.7 m, bentonite seal from 6.7 m to 5.8 m, grout from 5.8 m to 0.6 m and bentonite seal from 0.6 m to ground surface
420-15 North Abutment	7.7/357.5	Piezometer with 1.5 m slotted screen installed with sand filter to 5.9 m and bentonite seal from 5.9 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 recovered 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

The recovered 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" drawing in Appendix G. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general, the site is underlain by 2.7 m to 8.5 m of overburden soils overlying Pre-Cambrian bedrock. The overburden soils generally consist of topsoil and peat, sands and silts, gravel and sand and cobbles and boulders.

5.1 Topsoil and Peat

Across the site 0.1 m to 1.0 m of topsoil was encountered that extends to elevations ranging from 365.5 m to 363.4 m. In Borehole 420-01 a 0.3 m thick layer of peat was encountered, extending to elevation 363.6 m.

The moisture content of samples of this topsoil ranged from 37% to 88% and a sample of the peat had a moisture content of 210%.

5.2 Granular Fill

Boreholes 420-02 and 420-08 were drilled from the granular shoulder of the existing Highway 11. These two boreholes encountered a layer of granular fill that extends to depths ranging from 2.9 m to 3.0 m or to elevations ranging from 363.8 m to 363.7 m.

This fill can be divided into two zones. The upper zone consists of a 0.8 m to 2.4 m thick layer of gravelly sand and the lower zone consists of a 0.6 m to 2.1 m thick layer of sand with trace silt and trace gravel. Cobbles and boulders are inferred to exist within this deposit based on the resistance to augering during drilling and the blow count information obtained from SPT tests.

A sample from this deposit was subjected to a grain size distribution test and the results are presented in Figure B1.

Standard penetration tests conducted in this fill gave 'N' values ranging from 17 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 was approximately 2%.

Borehole 420-9 was drilled on the side slope of the existing Highway 11 embankment. This borehole encountered granular fill that extends to a depth of 1.5 m, or to elevation 364.0 m. The upper 0.9 m of this fill consisted of silt with some sand and topsoil. The lower 0.6 m of this fill consisted of sand and gravel with occasional cobbles.

A standard penetration test conducted in the sand and gravel fill gave an 'N' value of 17 blows per 0.3 m penetration indicating a compact relative density.

The moisture content of a sample from the sand and gravel fill was approximately 63%.

5.3 Silt

At the north abutment, a layer of silt with trace sand and occasional rootlets underlies the topsoil. This material extends to depths ranging from 0.6 m to 1.2 m or an elevation of 363.9 m.

Standard penetration tests in this silt gave 'N' values from 4 to 9 blows per 0.3 m penetration indicating a loose relative density.

The moisture content of samples from this material ranged from 42% to 59%.

5.4 Sand

In the vicinity of the north abutment, the topsoil and silt layers are further underlain by a deposit of well graded sand containing trace to some silt, trace to some gravel and occasional cobbles. This layer extends to depths ranging from 1.4 m to 2.3 m or to elevations varying between 363.5 m and 362.2 m.

Two selected samples from this deposit were subjected to grain size distribution tests and the results are presented in Figure B2.

SPT 'N' values ranged from 11 to 70 blows for 0.3 m penetration but generally, most 'N' values ranged between 11 and 47 blows for 0.3 m penetration. Based on these results this layer is considered to have a generally compact to dense relative density with occasional very dense zones.

The moisture content of samples from this deposit ranged from 15% to 24%.

At the south abutment, the topsoil, granular fill and silt are underlain by a zone of sand and gravel to gravelly sand containing trace to some silt and occasional cobbles. This zone extends to depths ranging from 2.3 m to 4.6 m or to elevations of 361.8 m to 360.9 m.

One sample from this deposit was subjected to grain size distribution testing and the results are presented in Figure B2.

SPT 'N' values in this material ranged from 4 to 43 blows per 0.3 m penetration, but were generally between 39 and 43 blows per 0.3 m penetration, indicating a dense relative density with a loose zone.

The moisture content of this material ranged from 15% to 18%.

5.5 Silt and Sand

Below the sand described above, a layer of soil was encountered that ranged from a fine, uniform sand to silty sand. This soil contained trace gravel and trace clay. The layer was encountered across the site but was discontinuous at the north end of the site.

The sand and silt layer was encountered at depths ranging from 0.9 m to 2.3 m below ground surface. This deposit extended to depths ranging from 2.2 m (Elev. 363.3 m) to 7.6 m (Elev. 356.5 m).

Samples from this deposit were subjected to grain size distribution tests and the results are illustrated in Figures B3a, B3b, B4a and B4b. The results show a soil consisting of 0 to 3% gravel, 24 to 59% sand, 33 to 72% silt and 4 to 9% clay sized particles.

SPT 'N' values in this deposit ranged from 6 to more than 50 blows for 0.3 m penetration, but generally, most values ranged between 13 and more than 50 blows for 0.3 m

penetration indicating a compact to very dense relative density. Auger resistance and grinding during drilling indicated the presence of occasional cobbles and boulders.

The moisture content of samples from this deposit ranged from 10% to 23%.

5.6 Gravel and Sand

A deposit of gravel and sand was encountered at depths ranging from 1.4 m to 7.6 m below ground surface. Cobbles and boulders were encountered throughout this deposit.

Most of the boreholes were advanced through this deposit and into bedrock at depths ranging from 2.7 to 8.5 m, corresponding to Elevation 362.8 to Elevation 356.6. The remaining boreholes, that were not advanced into bedrock, encountered auger refusal on assumed bedrock at similar elevations.

Selected samples from the gravel and sand matrix in this deposit were subjected to grain size distribution tests and the results are shown in Figure B5. The results show that the soil gradation is variable but generally consists of 42 to 48% gravel, 29 to 39% sand and 14 to 29% silt.

Standard Penetration tests in this deposit gave 'N' values ranging from 37 to more than 50 blows per 0.3 m penetration. Based on these results the deposit is considered to have a dense to very dense relative density.

The moisture content of samples from this stratum generally varies between 14% and 20%.

5.7 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	420-9	7.6	357.9
	420-12	7.8	356.3
North Abutment	420-15	4.9	360.3
	420-19	5.2	359.3

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 pink with black bands and occasional black blotches visible in most cores.

Core recovery in the bedrock was generally between 87% and 100%. The RQD values generally ranged from 63% to 100% indicating fair to excellent 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 and they were mostly tight with little to no infilling or secondary weathering material.

The unconfined compressive strength of most of the rock cores is estimated to range between 63 and 142 MPa indicating a strong to very strong intact rock. Outside of the foundation elements, at Borehole 420-18 the estimated unconfined compressive strength of the rock cores ranged from 39 to 48 MPa and in run 3 of Borehole 420-06 an estimated unconfined compressive strength of 49 MPa was recorded indicating a moderately strong rock. 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.

5.8 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 at the foundation elements are presented in Table 5.2.

Table 5.2: Water Level Measurements

Date	BH 420-12		BH 420-15	
	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)
November 17, 2005	0.3	363.8	0.6	364.6
November 18, 2005	0.3	363.8	0.5	364.7
November 21, 2005	0.1	364.0	0.4	364.8
November 23, 2005	0.1	364.0	0.4	364.8
November 28, 2005	0.1	364.0	0.6	364.6
November 29, 2005	0.1	364.0	0.5	364.7

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 363.8 m to 364.8 m. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

6 MISCELLANEOUS

All-Terrain Drilling of Waterloo, Ontario supplied a track mounted CME 75 drill rig 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 operations and prepared the report.

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

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 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 NBL 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 NBL at Sta. 18 + 032.

At the south abutment, the finished grade of Highway 11 will be at Elevation 371.9 and the existing ground surface at the east edge of the footing, remote from the existing highway embankment fill, lies at Elevation 364.1. The resulting embankment height above original ground level will, therefore, be in the order of 7.8 m at the south abutment.

At the north abutment, the finished grade of Highway 11 will be at Elevation 372.6 and the existing ground surface averages Elevation 364.9 at the abutment, resulting in an approach embankment in the order of 7.7 m high.

The grade of Muskoka 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 three 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 7.6 to 7.8 m of compact to very dense, cohesionless overburden soils overlying bedrock. The overburden consists of granular fill (topsoil at the east edge) underlain by sands silts, sand and gravel and cobbles and boulders.

At the north abutment, the stratigraphy consists of 4.0 to 5.2 m of cohesionless soils overlying bedrock.

The groundwater level exists at approximate Elevation 364.0 at the south abutment and 364.7 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 founded at or below the elevation given in Table 8.1 may be designed for the following values:

- Factored geotechnical resistance of 500 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

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

Table 8.1 – Founding Elevations on Native Soil

Location	Relevant Boreholes	Founding Elevation
South Abutment	BH 420-9, 420-12	363.0
North Abutment	BH 420-15, 420-16, 420-19	363.0

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 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

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 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 dense, cohesionless soils and it is recommended that the footings bearing on engineered fill be designed for the same values of geotechnical resistance as the native soil, i.e:

- Factored geotechnical resistance of 500 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

Using these values, there is no minimum thickness requirement for the engineered fill pad at these foundation locations.

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.

Table 8.2 - Maximum Elevation for Engineered Fill

Location	Relevant Boreholes	Founding Elevation
South Abutment	BH 420-9, 420-12	363.5
North Abutment	BH 420-15, 420-16, 420-19	364.0

Temporary excavations required to construct the engineered fill pads will extend in granular soils below the water table. Dewatering prior to excavation will 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 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 10 m below the underside of the footing at the south abutment, rising to approximately 8 m below the underside of the north abutment.

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 the same class of concrete and the footing 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 5 to 7 m below existing grade and 3 to 5 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 Driven to Bedrock (Abutments)

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

The stratigraphy encountered at the site consists of relatively thin overburden deposits overlying gently sloping bedrock. The most pronounced slope in the bedrock surface is at the north abutment, where the rock dips north to south. Table 8.3 below gives details on the natural bedrock elevations and the estimated pile lengths.

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. The minimum length of 6.8 m shown in Table 8.3 is considered to be acceptable.

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	420-9	7.6	357.9	367.0**	9.1
	420-12	7.8	356.3		10.7
North Abutment	420-15	4.9	360.3	367.6**	7.3
	420-16	4.1	360.8		6.8
	420-19	5.2	359.3		8.3

* From ground surface existing at the time of investigation

** From the General Arrangement Drawing

The recommended minimum pile length below the abutment is 6 m, consisting of 3 m in loose sand and a minimum of 3 m driven into resisting material below. In the case of short piles, it is also recommended that the piles all be of similar length to provide similar performance across the width of the abutment.

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 driven to 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 361.4	3,000	3.0	11	Sand and gravel, loose to dense
	361.4 to 356.3	10,000	3.3	10	Sand, trace gravel and cobbles, dense to very dense.
North Abutment	Granular B-I Fill	15,000	3.3	22	Compacted fill.
	OGL to 361.0	3,000	2.8	11	Sand, loose to very dense
	361.0 to 359.3 (or BDR)	10,000	3.3	10	Gravel and sand with cobbles

*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 Prunyn 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

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 the 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 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 will 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 just below 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 7 to 8 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	1.7	F1
Earth Fill	With Seismic	1.4	F2
Rock Fill	Normal	1.6	F3
Rock Fill	With Seismic	1.3	F4
North Approach			
Earth Fill	Normal	2.1	F5
Earth Fill	With Seismic	1.7	F6
Rock Fill	Normal	1.8	F7
Rock Fill	With Seismic	1.5	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. The sampled boreholes indicate that there is up to 1 m of topsoil or peat at the site. The contract drawing must indicate the requirement to strip this material prior to constructing the approach fills.

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 500 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 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 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.

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.

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.

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.

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.

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.

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.

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

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.

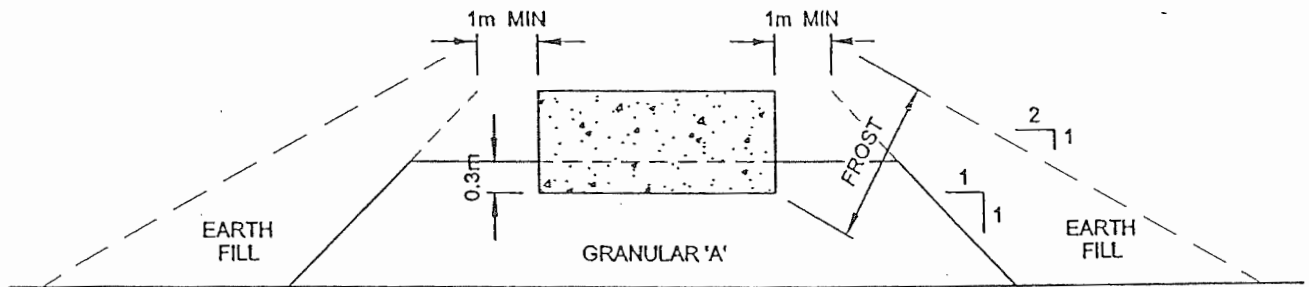
The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

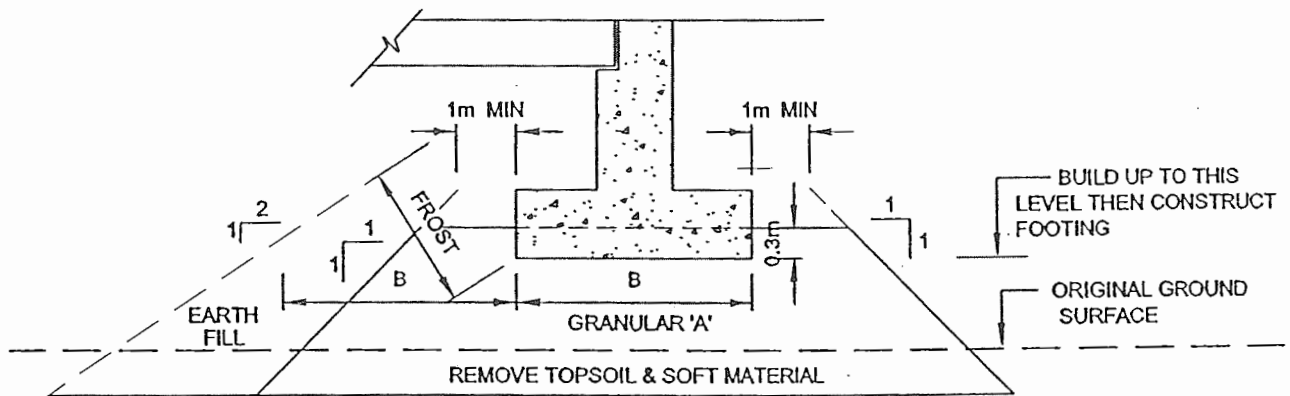
Alastair E. Gorman, P.Eng.,
Senior Foundations Engineer

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





CROSS-SECTION



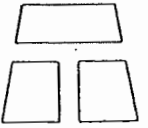
LONGITUDINAL SECTION

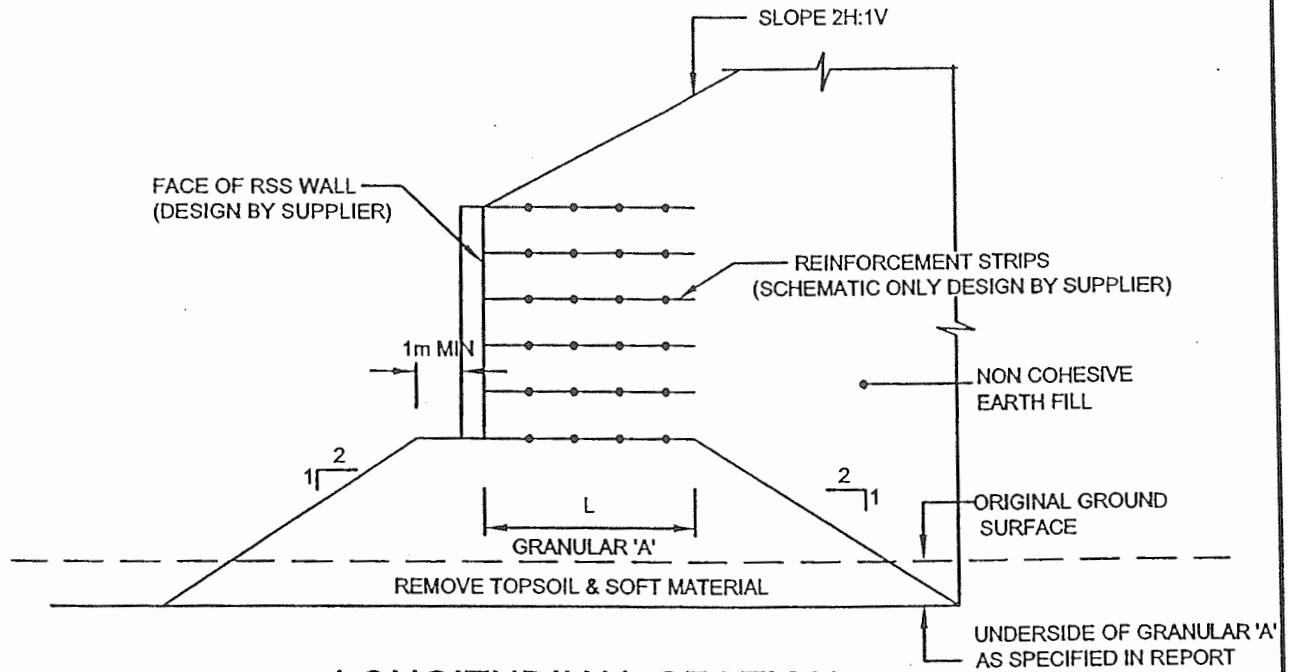
NOT TO SCALE

NOTES:

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

TED35146.DWG

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



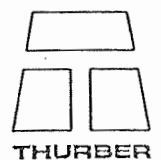
LONGITUDINAL SECTION
NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' BELOW PLAN AREA OF RSS MASS.
3. CONSTRUCT RSS MASS
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. MODIFIED FROM M.T.C 1982.

ENGINEER	RA
DRAWN	HS
DATE	Sept, 2004
APPROVED	PKC
SCALE	NTS

**RSS MASS ON COMPACTED FILL SHOWING
GRANULAR A**



DWG. NO.

FIGURE 2

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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



Water Level

C_{pen}

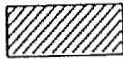

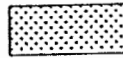


Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

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

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

RECORD OF BOREHOLE No 420-01

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 066 459.0 E 310 795.0 Muskoka Road Overpass (NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM/SS
 DATUM Geodetic DATE 17.08.04 - 17.08.04 CHECKED BY AEG



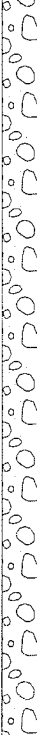

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
363.9														
0.0	PEAT, some rootlets													
363.6	Dark Brown		1	SS	4								210	
0.3	SAND and SILT, trace gravel													
	Dense to Very Dense													
	Brown		2	SS	41		363							
	Wet													
			3	SS	87/ .250		362							
			4	SS	90									
			5	SS	87/ .250		361							
359.9														
4.0	END OF BOREHOLE AT 3.96 m.													
	AUGER REFUSAL AT 3.96 m.													
	BOREHOLE OPEN TO 3.96 m.													
	WATER ENTERING BOREHOLE													
	FROM SURFACE AND WATER													
	LEVEL AT 3.96 m UPON													
	COMPLETION.													
	BOREHOLE GROUTED TO													
	SURFACE.													

RECORD OF BOREHOLE No 420-02

1 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 478.0 E 310 787.0 Muskoka Road Overpass (NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM/SS
 DATUM Geodetic DATE 25.08.04 - 25.08.04 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		W _P
366.7							20	40	60	80	100	20	40	60	GR	SA	SI	CL
0.0	Gravelly SAND, some silt Compact Brown Dry (FILL)		1	SS	25													
			2	SS	21													
	Possible cobbles between 1.52 and 2.13 m		3	SS	21													
364.3			4	SS	50/ .125													
2.4	SAND, trace silt, trace gravel Very Dense Grey (FILL)																	
363.7																		
3.0	SAND and SILT, trace gravel Compact Brown Damp		5	SS	22													
363.1																		
3.6	GRAVEL and SAND, trace silt, with cobbles and boulders Very Dense Brown Wet occasional cobbles or boulders between 3.66 and 4.27 m																	
			6	SS	71													
	possible boulders or cobbles between 5.18 and 6.10 m																	
	becoming sandy		7	SS	66													
	cobbles or boulders between 7.62 and 7.92 m	8	SS	50/ .125														
358.2																		
8.5	GNEISS BEDROCK, fresh, massive, pink with subhorizontal black banding, very strong																	
	Horizontal joints at 8.6 m, 9.0 m, 9.2 m, 9.8 m, 10.4 m and 10.7 m		1	RUN														

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

ONTM14S 420MUSKOKA-1.GPJ 26/06/06

RECORD OF BOREHOLE No 420-02

2 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 478.0 E 310 787.0 Muskoka Road Overpass (NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM/SS
 DATUM Geodetic DATE 25.08.04 - 25.08.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE																	
							20 40 60 80 100	20 40 60 80 100	20 40 60								
355.1			2	RUN			356								0	RUN 2#	
11.6	END OF BOREHOLE AT 11.58 m. BOREHOLE FILLED WITH DRILL WATER AND OPEN TO 11.58m. BOREHOLE GROUTED TO SURFACE.														0	TCR=100%, SCR=100%, RQD=100%, UCS=100MPa	

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

RECORD OF BOREHOLE No 420-06

1 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 466.8 E 310 810.5 Muskoka Road Overpass (NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM/SS
 DATUM Geodetic DATE 16.08.04 - 17.08.04 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		
364.2	TOPSOIL Black						364						
363.7													
0.5	SAND and SILT, trace gravel, occasional cobbles Compact to Very Dense Grey Wet		1	SS	28		363						0 51 44 6
			2	SS	50		362						
361.6			3	SS	52		361						
2.6	GRAVEL and SAND, trace silt, with cobbles and boulders Very Dense Brown Wet		4	SS	37		360						
	BOULDER		5	SS	50/ .125		359						
	Auger refusal at 4.8 m, borehole extended by coring and wash boring		6	SS	50/ .125		358						
357.8							357						
6.4	GNEISS BEDROCK, fresh, massive pink with horizontal black banding, very strong to strong, moderately strong from 7.9 m to 9.0 m		1	RUN			356						
			2	RUN			355						
			3	RUN									
	Horizontal joints at 9.1 m, 9.9 m, 10 m,												

Continued Next Page

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

ONTM/T4S 420MUSKOKA-1.GPJ 26/06/06

RECORD OF BOREHOLE No 420-06

2 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 466.8 E 310 810.5 Muskoka Road Overpass (NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM/SS
 DATUM Geodetic DATE 16.08.04 - 17.08.04 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		
	10.1 m, 10.6 m, 10.8 m and 11.2 m												1	
			4	RUN			354						0	
352.7							353						1	RUN 4# TCR=100%, SCR=95%, RQD=81%, UCS=90MPa
11.5	END OF BOREHOLE AT 11.46 m. BOREHOLE OPEN TO 11.46 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen. WATER LEVEL READINGS: DATE DEPTH Elevation (m) (m) 17/08/04 0.6 363.6 18/08/04 0.4 363.8													

ONTM14S 420MUSKOKA-1.GPJ 26/06/06

METRIC

SOIL PROFILE			SAMPLES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER		TYPE	"N" VALUES	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%) 20 40 60			GR SA SI CL
366.7 0.0	Gravelly SAND, some silt Compact Brown Dry (FILL)	[Cross-hatch pattern]	1	SS	29						
365.9 0.8	SAND, trace silt, trace gravel! Dense to Compact Brown Dry (FILL) Sampler refusal and resistance to augering at 2.4 m. Possible cobbles and/or boulders	[Cross-hatch pattern]	2	SS	37						
			3	SS	17						
			4	SS	50/ .150						
363.8 2.9	SILT and SAND, trace clay, occasional iron oxide staining Compact to Dense Brown Wet	[Vertical hatch pattern]	5	SS	21						0 29 61 9
			6	SS	30						
360.6 6.1	GRAVEL and SAND, trace silt, with cobbles and boulders Very Dense Brown Wet boulders and cobbles between 6.55 and 7.32 m	[Stippled pattern]	7	SS	96						
359.4 7.3	GNEISS BEDROCK, fresh to slightly weathered, massive, strong to very strong, pink with black banding Subhorizontal to horizontal joints at 7.4m, 7.8 m and 8.7 m	[Diagonal hatch pattern]	1	RUN						Ft 1 >5	RUN 1# TCR=100%, SCR=90%, RQD=90%, UCS=113MPa
			2	RUN						0 0 0 0 0	RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=98MPa

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 420-08

2 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 491.0 E 310 795.0 Muskoka Road Overpass (NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM/SS
 DATUM Geodetic DATE 25.08.04 - 25.08.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
356.3										0	
10.4	END OF BOREHOLE AT 10.36 m. BOREHOLE FILLED WITH DRILL WATER AND OPEN TO 10.36 m. BOREHOLE GROUTED TO SURFACE.						356				

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

RECORD OF BOREHOLE No 420-09

1 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 487.6 E 310 800.3 Muskoka Road Overpass NBL ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 15.11.05 - 15.11.05 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			SHEAR STRENGTH kPa				
								○ UNCONFINED + FIELD VANE				
								● QUICK TRIAXIAL × LAB VANE				
						WATER CONTENT (%)						
365.5												
0.0	SILT, some sand, some topsoil, trace rootlets and wood fragment Brown (FILL)											
364.6												
0.9	SAND and GRAVEL, occasional cobbles Brown (FILL)		1	SS	17							
364.0												
1.5	Sandy SILT, trace clay Loose to Compact Grey Moist to Wet		2	SS	6							
			3	SS	24							
362.6												
2.9	SAND and GRAVEL, occasional cobbles Dense Grey to Brown Wet		4	SS	43							
360.9												
4.6	Silty SAND, trace clay, trace gravel, occasional cobbles Very Dense Brown Wet		5	SS	74							
			6	SS	74/ 200							
357.9												
7.6	GNEISS BEDROCK Fresh to slightly weathered, thinly bedded, pink, with black banding, very strong to strong, rubble zone from 7.8m to 8.2m, subvertical joint from 9.3m to 9.6m		1	RUN								
			2	RUN								

Continued Next Page

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

ONTNT4S 420MUSKOKA-1.GPJ 26/06/06

RECORD OF BOREHOLE No 420-09

2 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 487.6 E 310 800.3 Muskoka Road Overpass NBL ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 15.11.05 - 15.11.05 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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
354.8							355						0	
10.7	END OF BOREHOLE AT 10.67m. BOREHOLE OPEN TO BOTTOM AND WATER LEVEL AT 1.52m UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.												1	

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

METRIC

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. PLT.	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)
								<div style="display: flex; justify-content: space-between;"> <div>○ UNCONFINED + FIELD VANE</div> <div>● QUICK TRIAXIAL × LAB VANE</div> </div>			<div style="display: flex; align-items: center;"> PLASTIC LIMIT w_p NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L </div>
								<div style="display: flex; justify-content: space-around;"> <div>20 40 60 80 100</div> <div>20 40 60 80 100</div> </div>			<div style="display: flex; justify-content: space-around;"> <div>20 40 60</div> <div>20 40 60</div> </div>
364.1 -0.0 0.1	TOPSOIL (75mm) SAND and GRAVEL, some silt Very Loose Brown	[Pattern]	1	SS	4						
363.5 -0.6	Wet SAND, some gravel to gravelly, some silt Dense Brown Wet	[Pattern]	2	SS	39						
		[Pattern]	3	SS	43					19 66 16 (SI+CL)	
361.8 -2.3	Silty SAND, trace clay, trace gravel, occasional cobbles, trace iron oxide staining Dense to Very Dense Brown Wet	[Pattern]	4	SS	31						
		[Pattern]	5	SS	61					3 58 33 5	
		[Pattern]	6	SS	76/ .150						
		[Pattern]	7	SS	83/ 100					1 56 35 8	
356.5 -7.8	BOULDER GRAVEL and SAND GNEISS BEDROCK Fresh to slightly weathered, thinly bedded, pink with black banding, strong, some clay, trace sand at joint surfaces	[Pattern]	1	RUN					FI	RUN 1# TCR=100%, SCR=100%, RQD=88%, UCS=68MPa	
		[Pattern]								RUN 2# TCR=100%, SCR=100%, RQD=100%	

$\div 3, \times 3$: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 420-12

2 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 487.9 E 310 811.4 Muskoka Road Overpass NBL ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 16.11.05 - 16.11.05 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20
353.2			2	RUN			354											
10.9	END OF BOREHOLE AT 10.87m. BOREHOLE OPEN TO BOTTOM AND WATER LEVEL AT 0.76m UPON COMPLETION. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) Nov. 17, 05 0.28 Nov. 18, 05 0.25 Nov. 21, 05 0.10 Nov. 23, 05 0.10 Nov. 28, 05 0.10 Nov. 29, 05 0.10																	

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

RECORD OF BOREHOLE No 420-13

1 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 482.5 E 310 815.4 Muskoka Road Overpass (NBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 03.05.04 - 03.05.04 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		
364.3	0.0	TOPSOIL Black					364							
363.4	0.9	SAND and SILT, trace gravel, trace clay, occasional cobbles	1	SS	46		363							
			2	SS	44		362							
		Dense to Very Dense Grey Wet	3	SS	35		361							1 58 36 5
360.7	3.6	GRAVEL and SAND, silty, with cobbles and boulders Very Dense inferred Grey Wet	4	SS	56		360							
							359							
			5	SS	60/ .150		358							
							357							
356.6	7.7	auger refusal at 7.6m. GNEISS, BEDROCK Fresh to slightly weathered, massive, strong to very strong, pink with black blotches.	1	RUN			356							RUN 1# TCR=100%, SCR=94%, RQD=94%, UCS=126MPa RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=142MPa RUN 3# TCR=100%, SCR=100%, RQD=95%,
			2	RUN			355							

Continued Next Page

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

RECORD OF BOREHOLE No 420-13

2 OF 2

METRIC

W.P. 758-93-02 LOCATION N 5 066 482.5 E 310 815.4 Muskoka Road Overpass (NBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 03.05.04 - 03.05.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
353.5			3	RUN			354					1	
10.7	END OF BOREHOLE AT 10.74m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH ELEVATION (m) (m) 04/05/04 0.3 364.0 18/06/04 0.5 363.8											3	

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

RECORD OF BOREHOLE No 420-15

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 066 516.4 E 310 829.8 Muskoka Road Overpass NBL ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 16.11.05 - 17.11.05 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		
365.2														
0.0	TOPSOIL (75mm)													
0.2	SILT, trace sand, occasional rootlets Loose Brown Wet		1	SS	9		365							
363.9			2	SS	9									
1.2	SAND, some gravel, trace silt Brown Wet						364							
363.4														
1.7	Silty SAND to SAND and SILT, trace clay, trace gravel, occasional cobbles Dense to Very Dense Brown Wet		3	SS	38		363							2 72 25 (SI+CL)
			4	SS	51									
							362							1 50 44 5
			5	SS	76									
360.6							361							
360.6	BOULDER													
360.3	GRAVEL and SAND		6	SS	50/									
4.9	GNEISS BEDROCK Fresh to slightly weathered, thinly bedded, pink, with black banding, very strong to strong				.000		360							RUN 1# TCR=100%, SCR=97%, RQD=87%, UCS=123MPa
			1	RUN										
							359							
			2	RUN										RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=95MPa
357.4							358							
7.7	END OF BOREHOLE AT 7.72m. BOREHOLE OPEN TO BOTTOM AND WATER LEVEL AT 0.46m UPON COMPLETION. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) Nov. 17, 05 0.56 Nov. 18, 05 0.52 Nov. 21, 05 0.42 Nov. 23, 05 0.41 Nov. 28, 05 0.58 Nov. 29, 05 0.53													







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

RECORD OF BOREHOLE No 420-16

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 065 509.8 E 310 828.6 Muskoka Road Overpass (NBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 29.04.04 - 29.04.04 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					
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL			× LAB VANE
365.0							20	40	60	80	100		
0.0	TOPSOIL (250mm)						365						
364.7													
0.3	SAND, trace to some silt Dense Brown Moist to Wet		1	SS	32		364						
363.5													
1.4	SILT and SAND, trace clay Compact Brown Wet		2	SS	22		363						0 38 57 5
362.7													
2.2	GRAVEL and SAND, silty, occasional cobbles and boulders below 3.7 m Very Dense Brown Wet		3	SS	66		362						42 29 29 (SI+CL)
360.8	auger refusal at 3.73m.		4	SS	60		361						
4.1	GNEISS (BEDROCK) Fresh to slightly weathered, massive, pink with black banding, strong to very strong. Subvertical joint at 4.9m Vertical joint at 5.0m Rubble zone from 5.7m to 5.9m		1	RUN			360					FI 0 1 2	RUN 1# TCR=100%, SCR=82%, RQD=74%, UCS=115MPa
			2	RUN			359					1 0 >5	RUN 2# TCR=96%, SCR=87%, RQD=75%, UCS=114MPa
			3	RUN			358					0 1 1	RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=63MPa
357.6													
7.4	END OF BOREHOLE AT 7.37m. BOREHOLE CAVED AT 3.3m ON COMPLETION OF CORING.												

ONTM174S 420MUSKOKA-1.GPJ 26/06/06

RECORD OF BOREHOLE No 420-18

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 066 499.8 E 310 831.9 Muskoka Road Overpass (NBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 29.04.04 - 29.04.04 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		
364.4														
0.0	TOPSOIL													
363.6														
0.8	SAND, trace to some silt Compact Brown Wet		1	SS	29									
			2	SS	20									
362.2														
2.2	SILT and SAND, trace gravel, trace clay Compact Brown Wet		3	SS	17									
361.4			4	SS	50/									
3.0	GRAVEL and SAND, silty, with cobbles and boulders		1	RUN	.051									
			2	RUN										
359.6														
4.9	GNEISS, BEDROCK Fresh, massive, pink with subhorizontal black banding, moderately strong Subvertical and vertical joints at 5.7m and 5.8m.		3	RUN										
			4	RUN										
357.3														
7.1	END OF BOREHOLE AT 7.13 m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH ELEVATION (m) (m) 30/04/04 0 364.4 18/06/04 0 364.4													

ONTMT4S 420MUSKOKA-1.GPJ 26/05/06

RECORD OF BOREHOLE No 420-20

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 066 520.5 E 310 833.2 Muskoka Road Overpass (NBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 30.04.04 - 30.04.04 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		
365.2														
0.0	TOPSOIL (250mm)													
365.0														
0.3	SAND, some silt, some gravel Compact Brown Wet		1	SS	19		365							20 65 14 (SI+CL)
							364							
	Dense		2	SS	38									
363.0														
2.2	GRAVEL and SAND, some silt, occasional cobbles Dense Brown Wet		3	SS	47		363							48 39 14 (SI+CL)
362.3														
2.9	END OF BOREHOLE AT 2.95m. AUGER REFUSAL AT 2.95m ON PROBABLE BEDROCK OR BOULDERS. WATER LEVEL AT 1.0m AND HOLE CAVED AT 1.1m ON COMPLETION.													

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

METRIC

[illegible]

RECORD OF BOREHOLE No 420-22

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 066 521.3 E 310 839.7 Muskoka Road Overpass (NBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 30.04.04 - 30.04.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
365.5														
0.0	TOPSOIL													
365.0														
0.5	SAND, trace to some silt, occasional cobbles Very Dense Brown Moist		1	SS	50/ .076		365							
364.1														
1.4	SAND and SILT Dense Grey/ Brown Wet		2	SS	34		364							
363.3														
2.2	GRAVEL and SAND, silty, occasional cobbles and boulders Very Dense Grey Wet		3	SS	66		363							
362.7														
2.8	END OF BOREHOLE AT 2.82m. AUGER REFUSAL AT 2.82m ON PROBABLE BEDROCK OR BOULDERS. WATER LEVEL AT 1.7m AND HOLE CAVED AT 2.3m ON COMPLETION.													

ONTMT-4S 420MUSKOKA-1.GPJ 26/06/06

RECORD OF BOREHOLE No 420-23

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 066 514.9 E 310 839.3 Muskoka Road Overpass (NBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 30.04.04 - 30.04.04 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						WATER CONTENT (%) w _p w w _L		
365.4							20	40	60	80	100					
0.0	TOPSOIL															
364.5																
0.9	SAND, trace to some silt Compact Brown Wet		1	SS	11											
364.0																
1.4	SILT and SAND, trace gravel, trace clay Very Dense Grey Moist		2	SS	52											3 38 54 6
363.2																
2.2	Inferred GRAVEL and SAND, silty, occasional cobbles and boulders Very Dense		3	SS	50/ .152											
362.2																
			4	SS	50/ .127											
3.2	END OF BOREHOLE AT 3.20m. AUGER REFUSAL AT 3.20m ON PROBABLE BEDROCK OR BOULDERS. WATER LEVEL AT 1.8m AND HOLE CAVED AT 2.2m ON COMPLETION.															

RECORD OF BOREHOLE No 420-24

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 066 511.3 E 310 843.1 Muskoka Road Overpass (NBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 29.04.04 - 29.04.04 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		
365.2														
0.0	TOPSOIL													
364.8														
0.4	SAND, trace to some silt, trace gravel Dense Brown Moist to Wet		1	SS	31									
363.5			2	SS	50/ .100									
1.7	GRAVEL and SAND, silty, with cobbles and boulders Very Dense inferred Brown Wet		1	RUN										
			2A	RUN										
361.5														
3.7	GNEISS (BEDROCK) Fresh, massive, strong to very strong, pink with subhorizontal black banding		2B	RUN										
			3	RUN										
359.1														
6.1	END OF BOREHOLE AT 6.12m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEVATION (m) 30/04/04 0.6 364.6 18/06/04 0.9 364.3													

RECORD OF BOREHOLE No 420-25

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 066 515.2 E 310 846.2 Muskoka Road Overpass (NBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 30.04.04 - 30.04.04 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		
365.6 0.0	TOPSOIL													
365.2 0.4	SAND, trace to some silt Dense Brown Moist		1	SS	42		365							
364.1 1.4	GRAVEL and SAND, silty, with cobbles and boulders Very Dense Brown Wet		2	SS	50/ .076		364							
			3	SS	50/ .051		363							
362.6 3.0	END OF BOREHOLE AT 3.0m. AUGER REFUSAL AT 3.0m ON PROBABLE BEDROCK OR BOULDERS. WATER LEVEL AT 2.1m AND HOLE CAVED AT 2.5m ON COMPLETION.													

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

RECORD OF BOREHOLE No 420-26

1 OF 1

METRIC

W.P. 758-93-02 LOCATION N 5 066 529.2 E 310 849.5 Muskoka Road Overpass (NBL) 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		
365.8														
0.0	TOPSOIL													
365.5														
0.3	SAND, trace to some silt Loose to Compact Brown Wet		1	SS	9		365							
			2	SS	11		364							
363.6														
2.2	GRAVEL and SAND Compact Brown Wet		3	SS	13									11 58 31 (SI+CL)
362.8							363							
3.0	END OF BOREHOLE AT 3.0m. AUGER REFUSAL AT 3.0m ON PROBABLE BEDROCK OR BOULDERS. WATER LEVEL AT 1.1m AND HOLE CAVED AT 1.4m ON COMPLETION.													

ONT.MT4S 420MUSKOKA-1.GPJ 26/06/06

Appendix B

Laboratory Test Results

FIGURE B1

Size of openings, inches

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

PERCENT FINER THAN

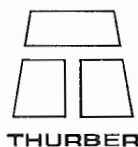
GRAIN SIZE, mm

Grain Size (mm)	U.S.S. Sieve Size (meshes/inch)	Size of Opening (inches)	Percent Finer (%)
100	20	2.0	100
75	25	1.5	100
60	30	1.25	100
50	35	1.0	100
40	40	0.85	100
30	50	0.75	100
25	60	0.6	100
20	75	0.5	100
15	100	0.425	100
12.5	120	0.375	100
10	150	0.3	100
7.5	200	0.25	100
6.0	250	0.2	100
5.0	300	0.165	100
4.0	40	0.425	100
3.0	60	0.25	100
2.0	100	0.15	100
1.5	120	0.125	100
1.18	150	0.106	100
1.0	20	2.0	100
0.85	25	1.5	100
0.75	30	1.25	100
0.6	40	0.85	100
0.5	50	0.75	100
0.425	60	0.6	100
0.375	75	0.5	100
0.3	100	0.425	100
0.25	120	0.375	100
0.2	150	0.3	100
0.165	200	0.25	100
0.15	250	0.2	100
0.125	300	0.165	100
0.106	40	0.425	100
0.10	20	2.0	100
0.085	25	1.5	100
0.075	30	1.25	100
0.06	40	0.85	100
0.05	50	0.75	100
0.0425	60	0.6	100
0.0375	75	0.5	100
0.03	100	0.425	100
0.025	120	0.375	100
0.02	150	0.3	100
0.0165	200	0.25	100
0.015	250	0.2	100
0.0125	300	0.165	100
0.0106	40	0.425	100
0.01	20	2.0	100
0.0085	25	1.5	100
0.0075	30	1.25	100
0.006	40	0.85	100
0.005	50	0.75	100
0.00425	60	0.6	100
0.00375	75	0.5	100
0.003	100	0.425	100
0.0025	120	0.375	100
0.002	150	0.3	100
0.00165	200	0.25	100
0.0015	250	0.2	100
0.00125	300	0.165	100
0.00106	40	0.425	100
0.001	20	2.0	100
0.00085	25	1.5	100
0.00075	30	1.25	100
0.0006	40	0.85	100
0.0005	50	0.75	100
0.000425	60	0.6	100
0.000375	75	0.5	100
0.0003	100	0.425	100
0.00025	120	0.375	100
0.0002	150	0.3	100
0.000165	200	0.25	100
0.00015	250	0.2	100
0.000125	300	0.165	100
0.000106	40	0.425	100
0.0001	20	2.0	100
0.000085	25	1.5	100
0.000075	30	1.25	100
0.00006	40	0.85	100
0.00005	50	0.75	100
0.0000425	60	0.6	100
0.0000375	75	0.5	100
0.00003	100	0.425	100
0.000025	120	0.375	100
0.00002	150	0.3	

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-02	1.07	365.63

Date June 2006
Project 759-93-00

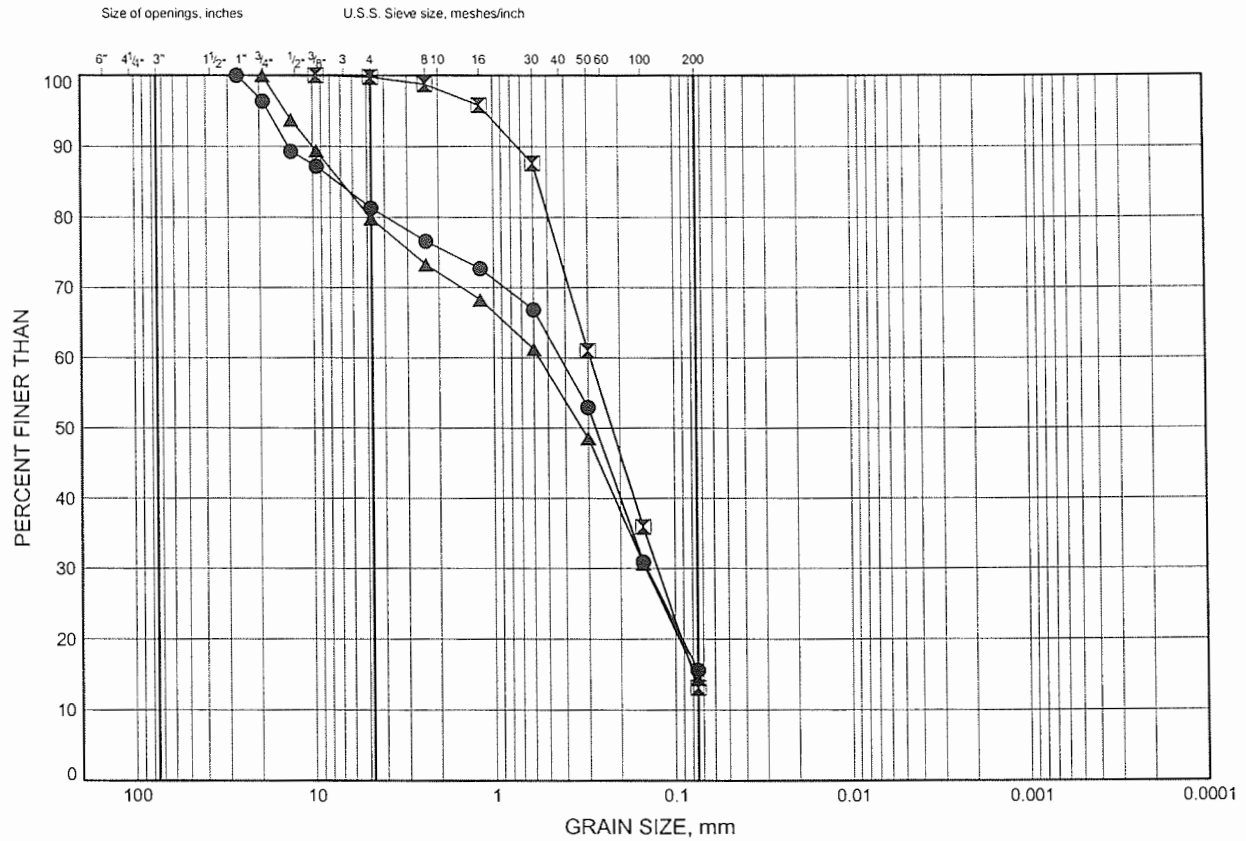


Prep'd JHL
Chkd. MEF

Hwy 11 Four Laning GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND

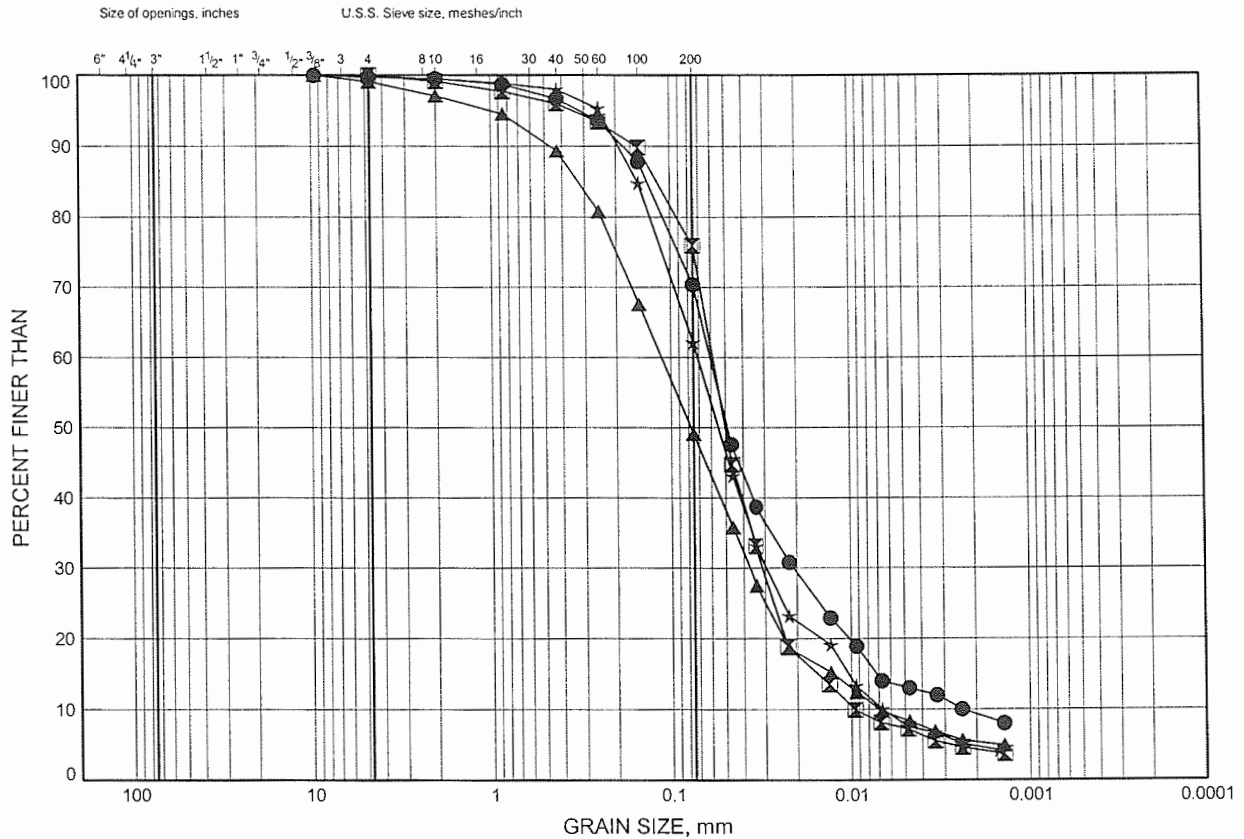


Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B3a

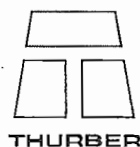
SILT AND SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-08	3.35	363.35
⊠	420-9	2.59	362.90
▲	420-15	3.23	361.92
★	420-16	1.83	363.12

Date June 2006
Project 759-93-00



Prep'd JHL
Chkd. MEF

FIGURE B3b

Size of openings, inches

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

PERCENT FINER THAN

GRAIN SIZE, mm

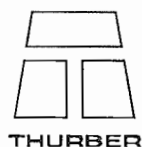
Grain Size (mm)	Percent Finer (Triangles)	Percent Finer (Circles)	Percent Finer (Crosses)
100	100	100	100
10	100	100	100
1	95	98	100
0.1	78	92	98
0.075	60	78	88
0.06	45	65	72
0.05	42	58	62
0.04	38	52	55
0.03	32	45	48
0.025	28	40	42
0.02	25	35	38
0.015	22	30	32
0.01	18	25	28
0.0075	15	20	22
0.006	12	15	18
0.005	10	12	15
0.004	8	10	12
0.003	6	8	10
0.0025	5	7	9
0.002	4	6	8
0.0015	3	5	7
0.001	2	4	6

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-18	2.59	361.83
☒	420-19	2.59	361.93
▲	420-23	1.83	363.59

Date June 2006
Project 759-93-00

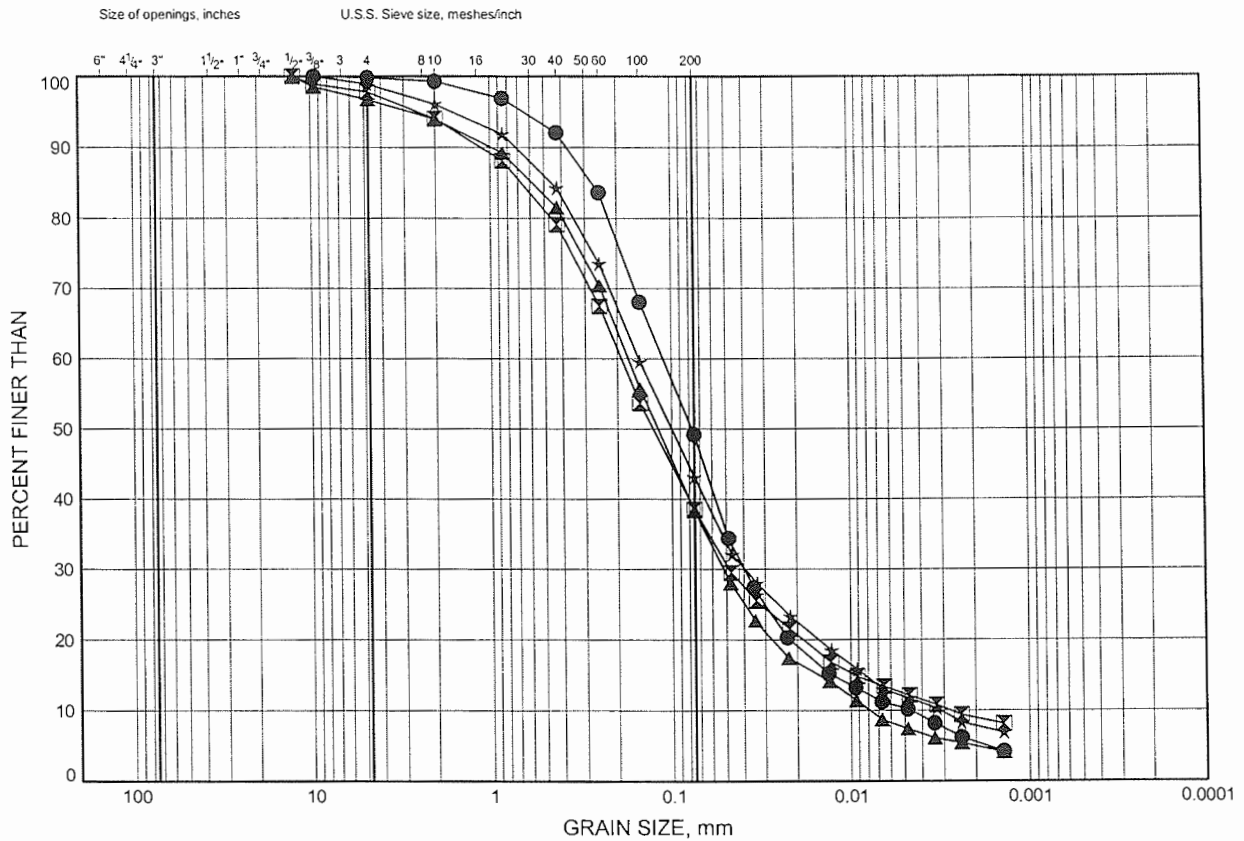
Prep'd JHL
Chkd. MEF



Hwy 11 Four Laning GRAIN SIZE DISTRIBUTION

FIGURE B4a

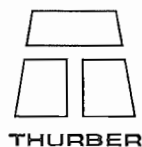
SAND AND SILT



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-06	1.07	363.13
⊠	420-9	4.88	360.62
▲	420-12	3.35	360.73
★	420-12	6.25	357.84

Date June 2006
Project 759-93-00



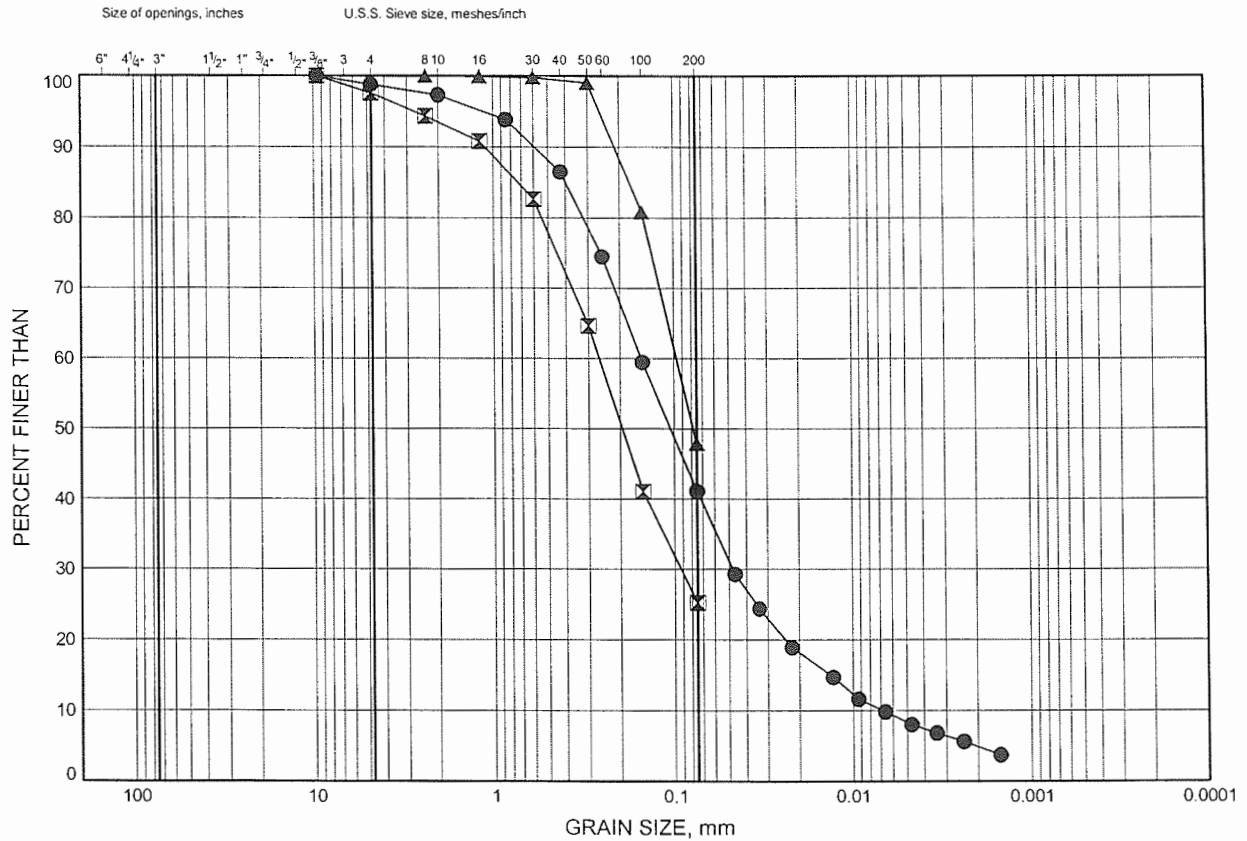
Prep'd JHL
Chkd. MEF

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B4b

SAND AND SILT

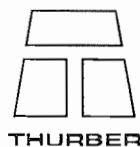


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-13	2.59	361.69
⊠	420-15	1.83	363.32
▲	420-22	1.83	363.66

Date June 2006

Project 759-93-00



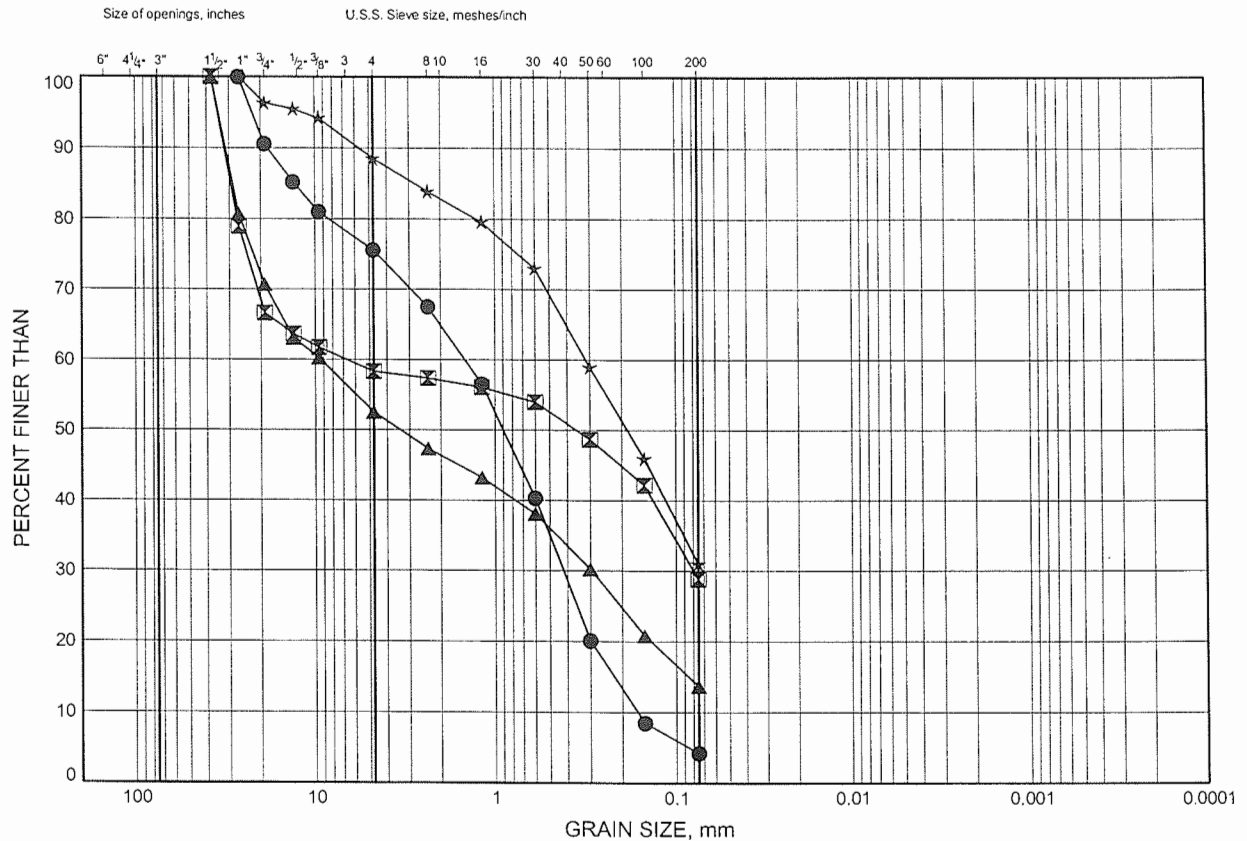
Prep'd JHL

Chkd. MEF

Hwy 11 Four Laning GRAIN SIZE DISTRIBUTION

FIGURE B5

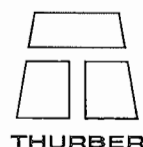
GRAVEL AND SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-02	4.88	361.82
⊠	420-16	2.59	362.36
▲	420-20	2.59	362.66
★	420-26	2.59	363.24

Date June 2006
Project 759-93-00



Prep'd JHL
Chkd. MEF

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

Depth			Is50	UCS (MPa)					
feet	Inches	m							
420-02									
28	7	8.71	4.36	104.71	}	Total Rock Core			
29	10	9.09	4.58	109.90		Average	Minimum	Maximum	
30	7	9.32	3.93	94.35		102	93	110	MPa
31	7	9.63	4.54	108.86		Run #	Average		
32	8	9.96	4.41	105.75		1	104.71		
33	8	10.26	3.93	94.35		2	100.15		
34	8	10.57	3.89	93.31					
35	7	10.85	4.19	100.57					
36	7	11.15	4.36	104.71					
37	7	11.46	4.49	107.82					

Depth			Is50	UCS (MPa)				
feet	Inches	m			Average	Minimum	Maximum	
420-06					96	25	143	MPa
21	3	6.48	5.10	122.34	}	Run #	Average	
22	5	6.83	5.62	134.78		1	125.66	
23	4	7.11	4.88	117.16		2	49.07	
24	5	7.44	5.96	143.08		3	96.68	
25	7	7.80	4.62	110.94		4	90.55	
27	0	8.23	1.99	47.69				
28	0	8.53	3.11	74.65				
29	2	8.89	1.04	24.88				
30	5	9.27	3.37	80.87				
31	6	9.60	4.28	102.64				
32	8	9.96	3.84	92.27				
33	8	10.26	4.62	110.94				
35	0	10.67	2.81	67.39				
36	2	11.02	4.67	111.97				
37	0	11.28	3.84	92.27				

Note: Point load test at 8.89 m was performed at hidden joint

Depth			Is50	UCS (MPa)					
feet	Inches	m							
420-08									
24	9	7.54	5.83	139.97	}	Total Rock Core			
25	8	7.82	3.89	93.31		Average	Minimum	Maximum	
26	6	8.08	4.10	98.49		106	83	140	MPa
27	6	8.38	5.10	122.34		Run #	Average		
28	8	8.74	4.67	111.97		1	113.22		
29	8	9.04	4.45	106.79		2	98.29		
30	10	9.40	3.46	82.94					
31	10	9.70	3.54	85.02					
32	10	10.01	4.36	104.71					
33	8	10.26	4.67	111.97					

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

Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-9					Total Rock Core			
25	3	7.70	5.33	128.01	Average	Minimum	Maximum	MPa
27	5	8.36	1.52	36.43	81	36	128	
29	9	9.07	2.82	67.65	Run #	Average		
30	3	9.22	2.82	67.65	1	77.36		
31	9	9.68	4.34	104.07	2	84.99		
33	11	10.34	3.47	83.26				
Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-12					Total Rock Core			
26	2	7.98	3.90	93.66	Average	Minimum	Maximum	MPa
27	7	8.41	0.43	10.41	116	10	168	
30	0	9.14	4.21	100.95	Run #	Average		
31	11	9.73	3.90	93.66	1	68.34		
34	3	10.44	4.77	114.48	2	100.60		
35	4	10.77	3.90	93.66				
Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-13					Total Rock Core			
25	6	7.77	5.36	128.56	Average	Minimum	Maximum	MPa
26	0	7.92	5.27	126.49	133	64	168	
26	8	8.13	5.18	124.41	Run #	Average		
27	8	8.43	7.00	167.96	1	126.49		
28	8	8.74	6.91	165.88	2	142.66		
29	8	9.04	5.01	120.27	3	121.65		
30	4	9.25	6.48	155.52				
31	4	9.55	4.32	103.68				
32	4	9.86	6.22	149.30				
33	4	10.16	2.68	64.28				
34	8	10.57	6.31	151.37				
Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-15					Total Rock Core			
16	4	4.98	4.77	114.48	Average	Minimum	Maximum	MPa
18	5	5.61	5.20	124.89	109	83	130	
20	2	6.15	5.42	130.09	Run #	Average		
21	2	6.45	4.55	109.28	1	123.15		
23	7	7.19	3.90	93.66	2	95.40		
24	10	7.57	3.47	83.26				

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

Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-16								
14	0	4.27	4.54	108.96	}	Total Rock Core		
15	4	4.67	4.84	116.16		Average	Minimum	Maximum
16	0	4.88	5.01	120.24		98	21	143 MPa
17	0	5.18	4.75	114.00		Run #	Average	
17	8	5.38	4.97	119.28		1	115.73	
18	6	5.64	4.97	119.28		2	113.36	
19	6	5.94	3.24	77.76		3	63.24	
20	2	6.15	5.96	143.04				
21	0	6.40	4.32	103.68				
21	8	6.60	3.20	76.80				
22	6	6.86	2.16	51.84				
23	4	7.11	0.86	20.64				
Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-18								
17	0	5.18	2.16	51.84	}	Total Rock Core		
17	10	5.44	1.38	33.12		Average	Minimum	Maximum
18	8	5.69	1.64	39.36		43	29	85
19	8	5.99	1.38	33.12		Run #	Average	
21	6	6.55	3.54	84.96		3	39.36	
22	2	6.76	1.30	31.20		4	48.40	
23	2	7.06	1.21	29.04				
Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-19								
17	8	5.38	2.82	67.65	}	Total Rock Core		
19	4	5.89	2.82	67.65		Average	Minimum	Maximum
21	7	6.58	4.12	98.87		88	68	104 MPa
22	10	6.96	4.34	104.07		Run #	Average	
24	1	7.34	4.01	96.27		1	78.05	
26	7	8.10	3.90	93.66		2	98.00	
Depth			Is50	UCS (MPa)				
feet	Inches	m						
420-21								
9	4	2.84	5.40	129.60	}	Total Rock Core		
10	3	3.12	4.84	116.16		Average	Minimum	Maximum
11	3	3.43	1.08	25.92		78	23	130
11	9	3.58	0.95	22.80		Run #	Average	
12	9	3.89	1.90	45.60		1	90.56	
13	7	4.14	3.67	88.08		2	69.46	
15	0	4.57	3.46	83.04		3	79.80	
16	0	4.88	4.49	107.76				
17	0	5.18	3.54	84.96				
17	5	5.31	3.11	74.64				

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

feet	Depth		Is50	UCS (MPa)			
	Inches	m					
420-24							
12	10	3.91	4.58	109.92	}	Total Rock Core	
13	8	4.17	4.58	109.92		Average	Minimum Maximum
14	6	4.42	4.92	118.08		101	42 118
15	8	4.78	4.32	103.68		Run #	Average
16	6	5.03	4.23	101.52		2B	112.64
17	4	5.28	4.84	116.16		3	93.74
18	2	5.54	1.73	41.52			
19	0	5.79	4.41	105.84			

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 5066468.88; E 310835		ORIGINATED BY SB											
DIST 54 HWY 11		BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS		COMPILED BY OKB											
DATUM GEODETIC		DATE Feb.29/00		CHECKED BY ASP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED							
364.77	GROUND SURFACE														
0.00 364.47	Topsoil		1	SS	2										
0.30 364.08	Sandy silt, trace clay and organics Very loose Brown Wet organic content = 5.5% Sand, trace to some silt Compact Brown Wet		2	SS	22										
0.69			3	SS	22										
362.56			4	SS	15										
2.21	Silt and Sand, trace clay. Compact Brown Wet		5	SS	55.05										
361.80															
3.08	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, X 3. Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

+3, X3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

PROJECT 991-1193		RECORD OF BOREHOLE No 7-3		1 OF 1 METRIC	
W.P. 335-98-00		LOCATION N 5065559.56; 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 w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED														
							20	40	60	80	100	10	20	30	GR	SA	SI	CL				
366.14	GROUND SURFACE																					
0.00	Sand and Gravel, trace silt Compact to dense Brown Moist (Fill)		1	SS	22																	
			2	SS	36																	
364.69																						
1.45	Silt and Sand, trace gravel, trace clay and organics																					
364.31	Loose Brown		3	SS	8													2	38	48	8	
1.83	Wet non-plastic Atterberg limit test result for Sample 3A organic content=3.1%																					
			4	SS	34														4	81	15	0
363.09	Sand, some silt, trace gravel																					
3.05	Dense Brown Moist Slightly to moderately weathered, grey and white-brown with black blotches, foliated (30°), moderately jointed, coarse to very coarse grained, medium strong GNEISS.																					
	Bedrock cored from 3.05m to 6.10m depth. For bedrock coring details refer to Record of Drillhole 7-3																					
360.04																						
6.10	END OF HOLE Note: Open borehole dry upon completion of drilling.																					

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

PROJECT 991-1193		RECORD OF BOREHOLE No 7-4		1 OF 1		METRIC	
W.P. 335-98-00		LOCATION N 5065579.07; E 310794		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 LIMIT MOISTURE CONTENT			UNIT WEIGHT Y kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20 40 60 80 100					W _p	W	W _L		
367.00	GROUND SURFACE																
0.00	Sandy Silt, trace clay and organics/decaying wood matter Loose		1	SS	4	366											
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	1007.03												
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"> i. Lower unit cost compared to pile foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. An integral abutment design is not an available option ii. Comparatively longer abutment stem. iii. 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. An integral abutment design is not an available option ii. Cost of constructing engineered fill iii. 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 NBL
Highway 11 Burk's Falls to South River

Appendix E

Special Provisions

Muskoka Road Overpass NBL
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 362. 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 NBL
Highway 11 Burk’s Falls to South River

Appendix F

Computer Output

Thurber Engineering Ltd. - Toronto
19-1423-12
Hwy 11, Burk's Falls
June 27, 2006
Muskoka Road NBL South Approach
Earth Fill

	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Earth Fill	20	0	30	0	1
Sand	21	0	31	0	1
Silty sand	22	0	32	0	1

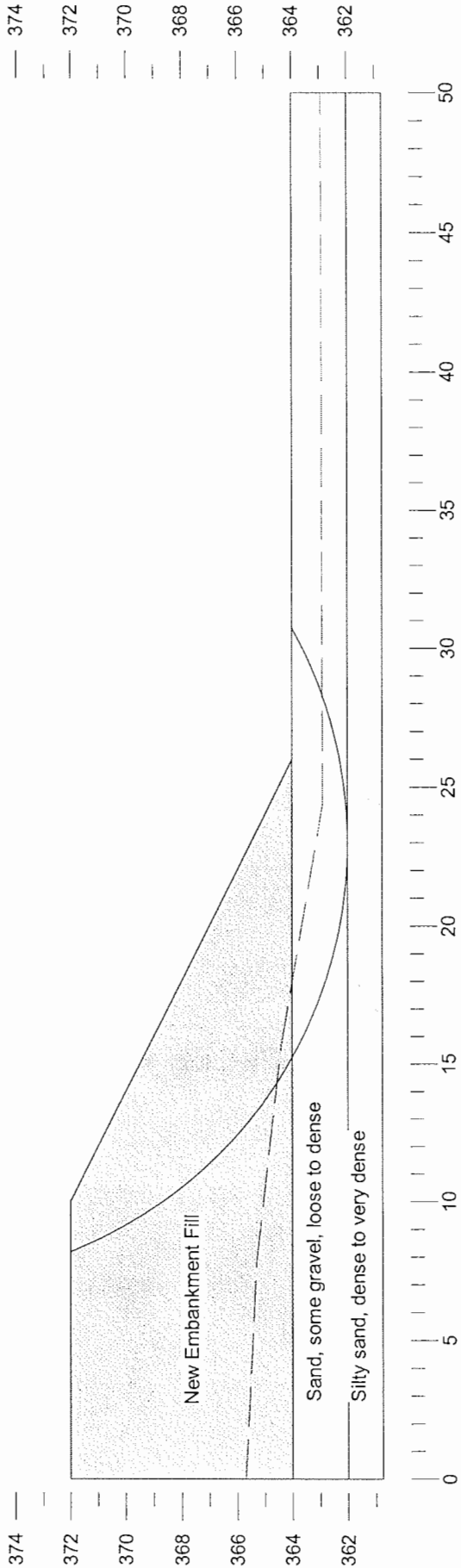
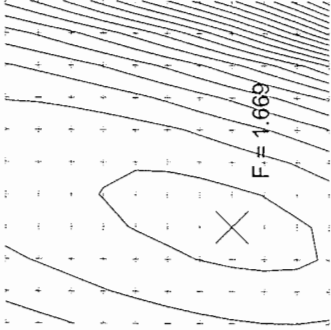


Figure F1

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 June 27, 2006
 Muskoka Road NBL South Approach
 Earth Fill, with seismic

	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Earth Fill	20	0	30	0	1
Sand	21	0	31	0	1
Silty sand	22	0	32	0	1

Seismic coefficient = 0.08

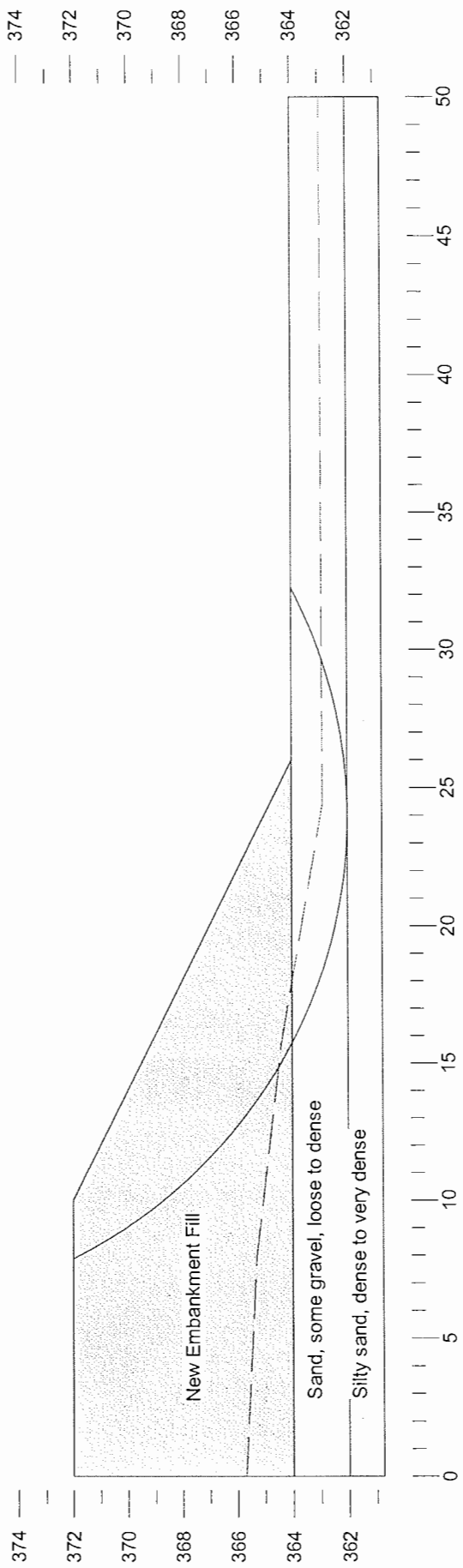
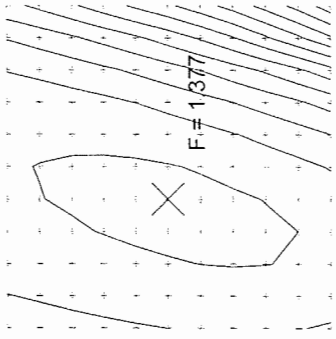


Figure F2

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 June 27, 2006
 Muskoka Road NBL South Approach
 Rock Fill

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rock Fill	19	0	42	0
Sand	21	0	31	0
Silty sand	22	0	32	0

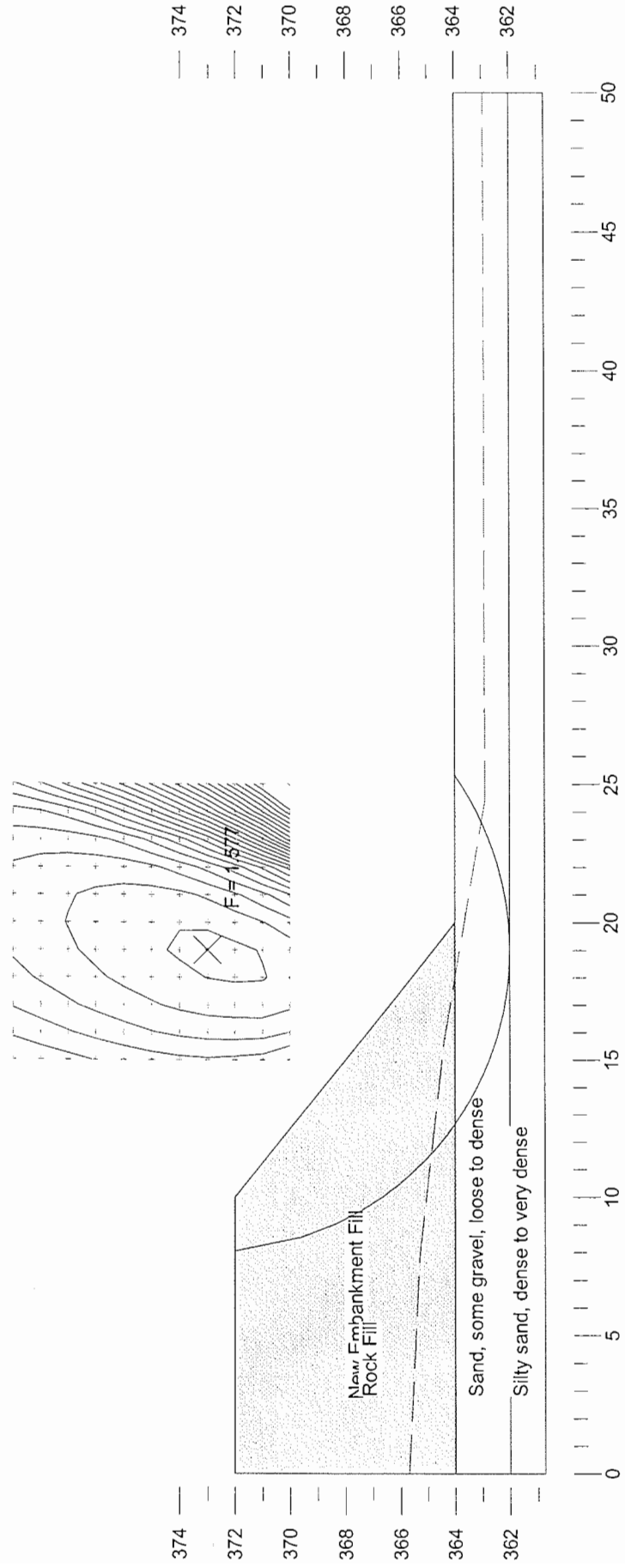


Figure F3

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 June 27, 2006
 Muskoka Road NBL South Approach
 Rock Fill, seismic

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rock Fill	19	0	42	0
Sand	21	0	31	0
Silty sand	22	0	32	0
Seismic coefficient = 0.08				

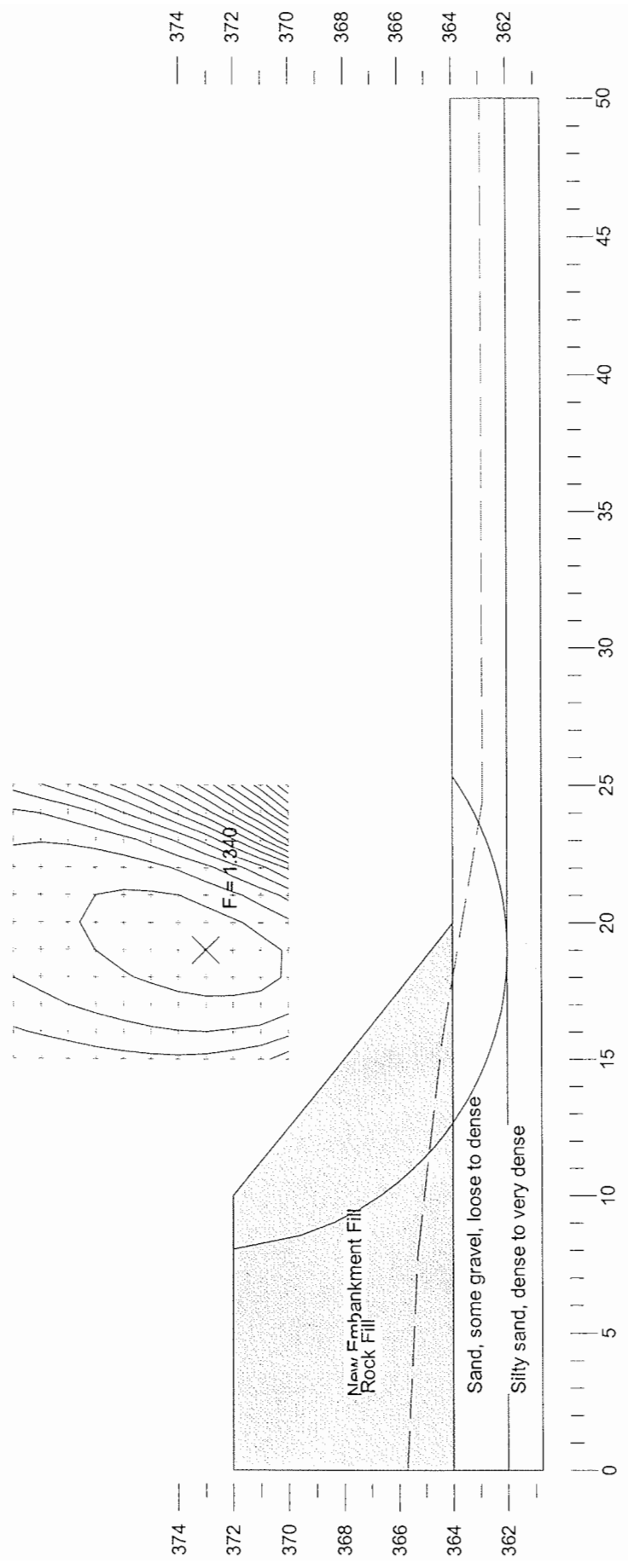


Figure F4

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 June 27, 2006
 Muskoka Road NBL North Approach
 Earth Fill

	Gamma C	Phi	Min	Piezo
	kN/m ³	deg	c/p	Surf.
Earth Fill	20	0	30	0
Silt	20	0	29	0
Sand	21	0	31	0
Sandy Silty	21	0	32	0
Gravel and Sand	22	0	33	0

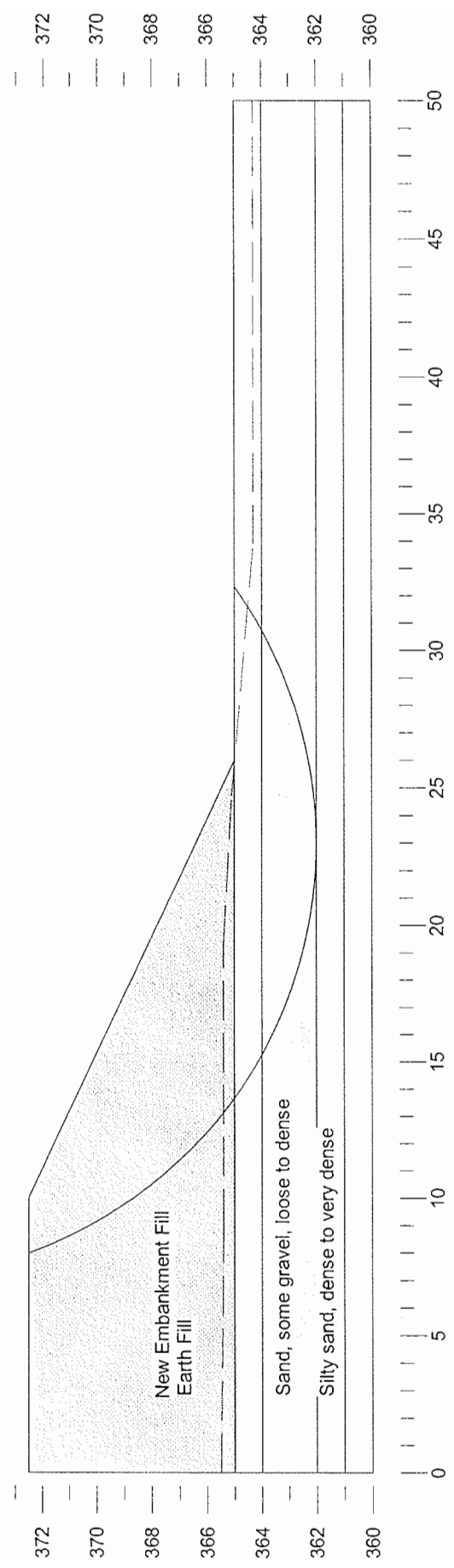
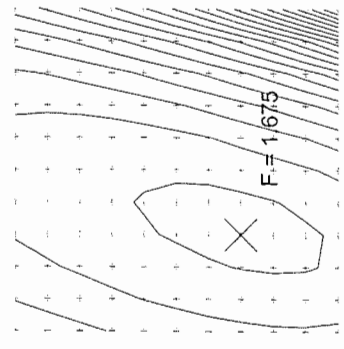


Figure F5

Thurber Engineering Ltd. - Toronto					
19-1423-12					
Hwy 11, Burk's Falls					
June 27, 2006					
Muskoka Road NBL North Approach					
Earth Fill, Seismic					
Earth Fill	Gamma C	Phi	Min	Piezo	
	kN/m3	deg	c/p	Surf.	
Silt	20	30	0	1	
Sand	20	29	0	1	
Sandy Sily	21	31	0	1	
Gravel and Sand	21	32	0	1	
	22	33	0	1	
Seismic coefficient = 0.08					

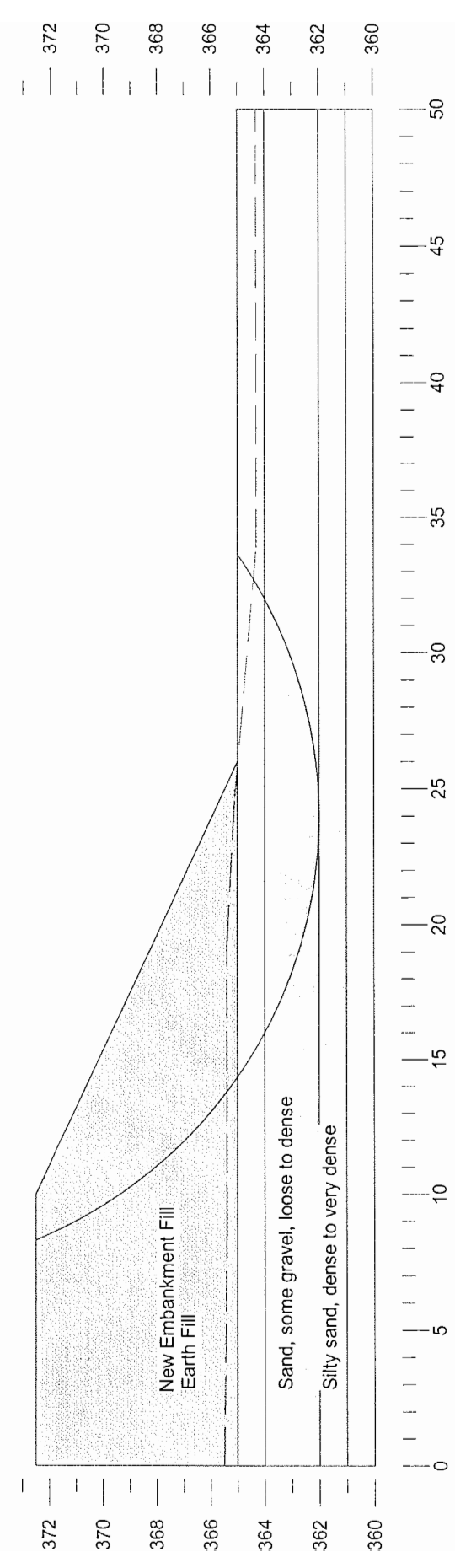
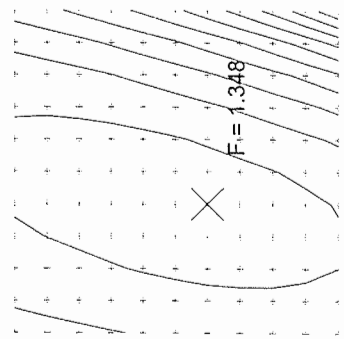


Figure F6

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 June 27, 2006
 Muskoka Road NBL North Approach
 Rock Fill

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rock Fill	19	0	42	0
Silt	20	0	29	0
Sand	21	0	31	0
Sandy Silty	21	0	32	0
Gravel and Sand	22	0	33	0

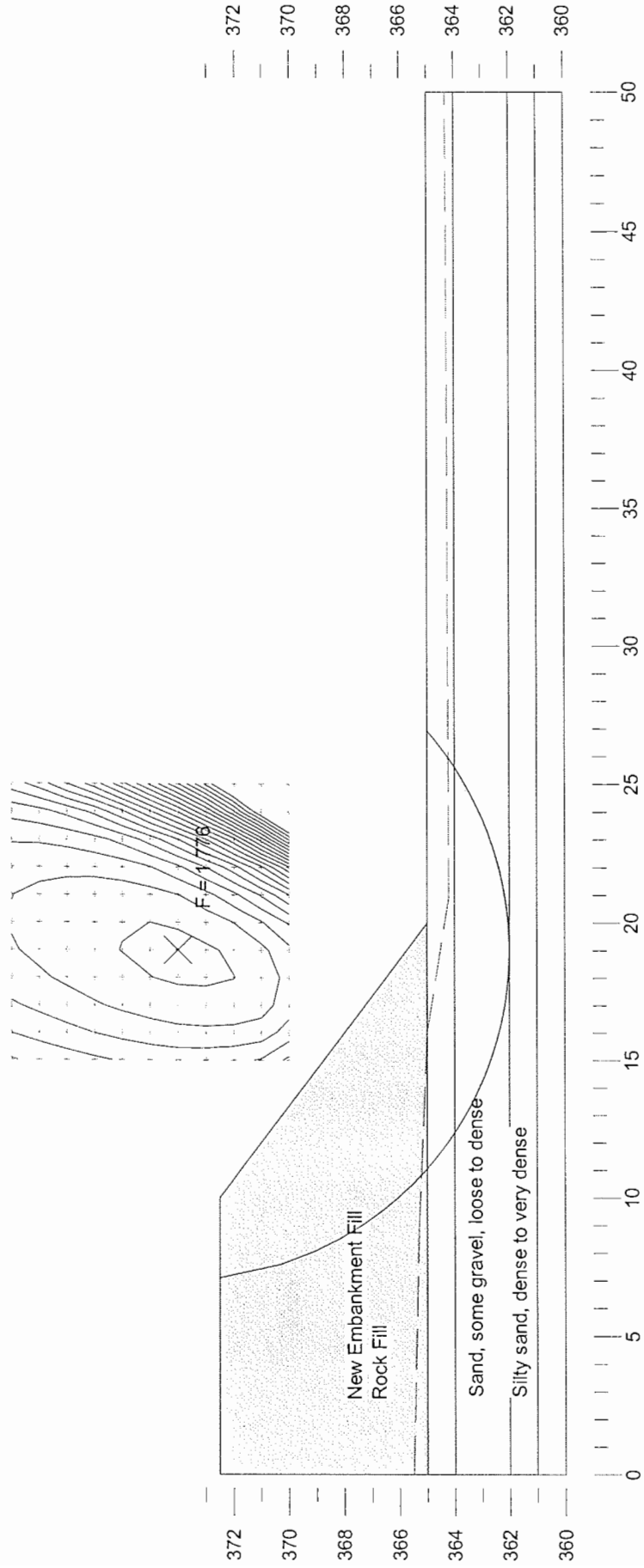


Figure F7

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy 11, Burk's Falls
 June 27, 2006
 Muskoka Road NBL North Approach
 Rock Fill, seismic

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rock Fill	19	0	42	0
Silt	20	0	29	0
Sand	21	0	31	0
Sandy Silt	21	0	32	0
Gravel and Sand	22	0	33	0

Seismic coefficient = 0.08

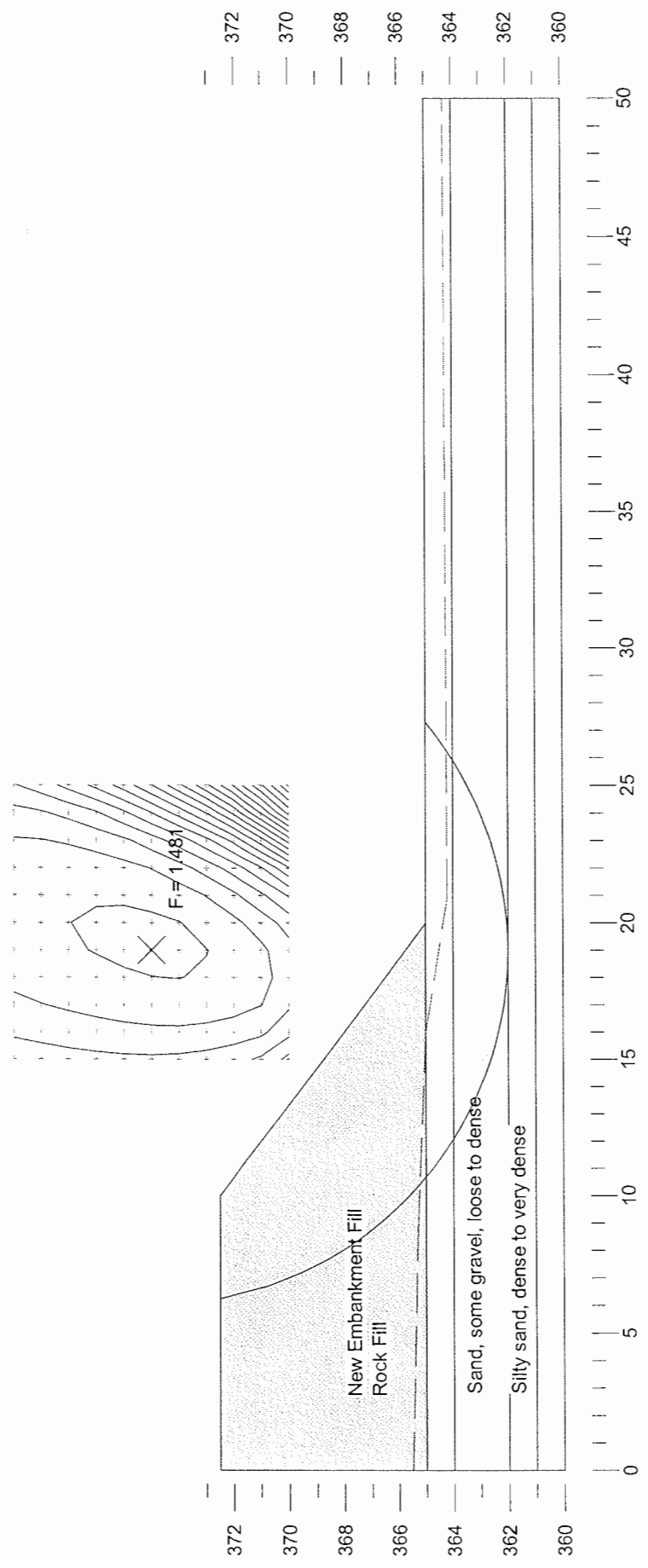
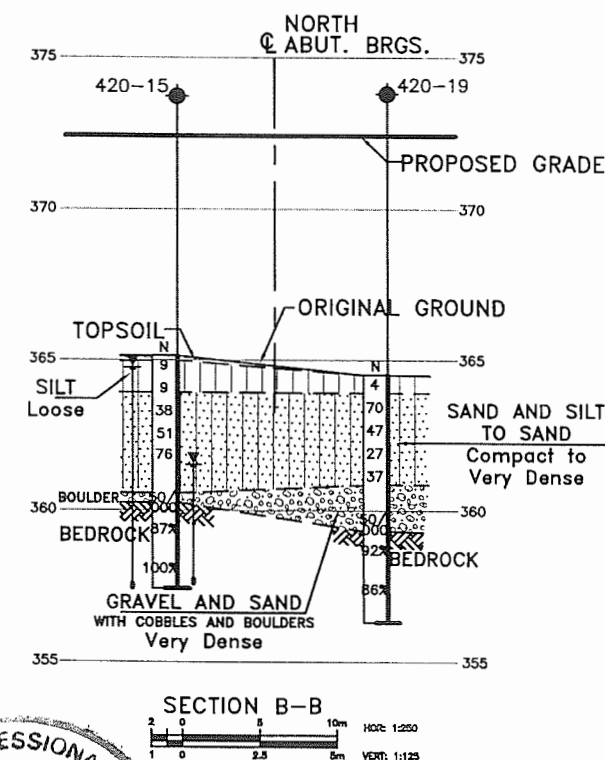
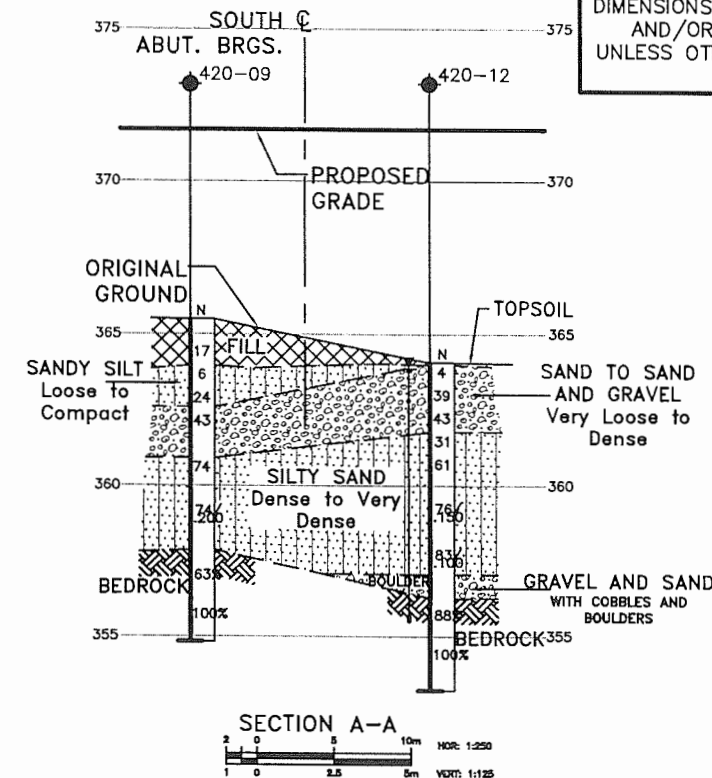
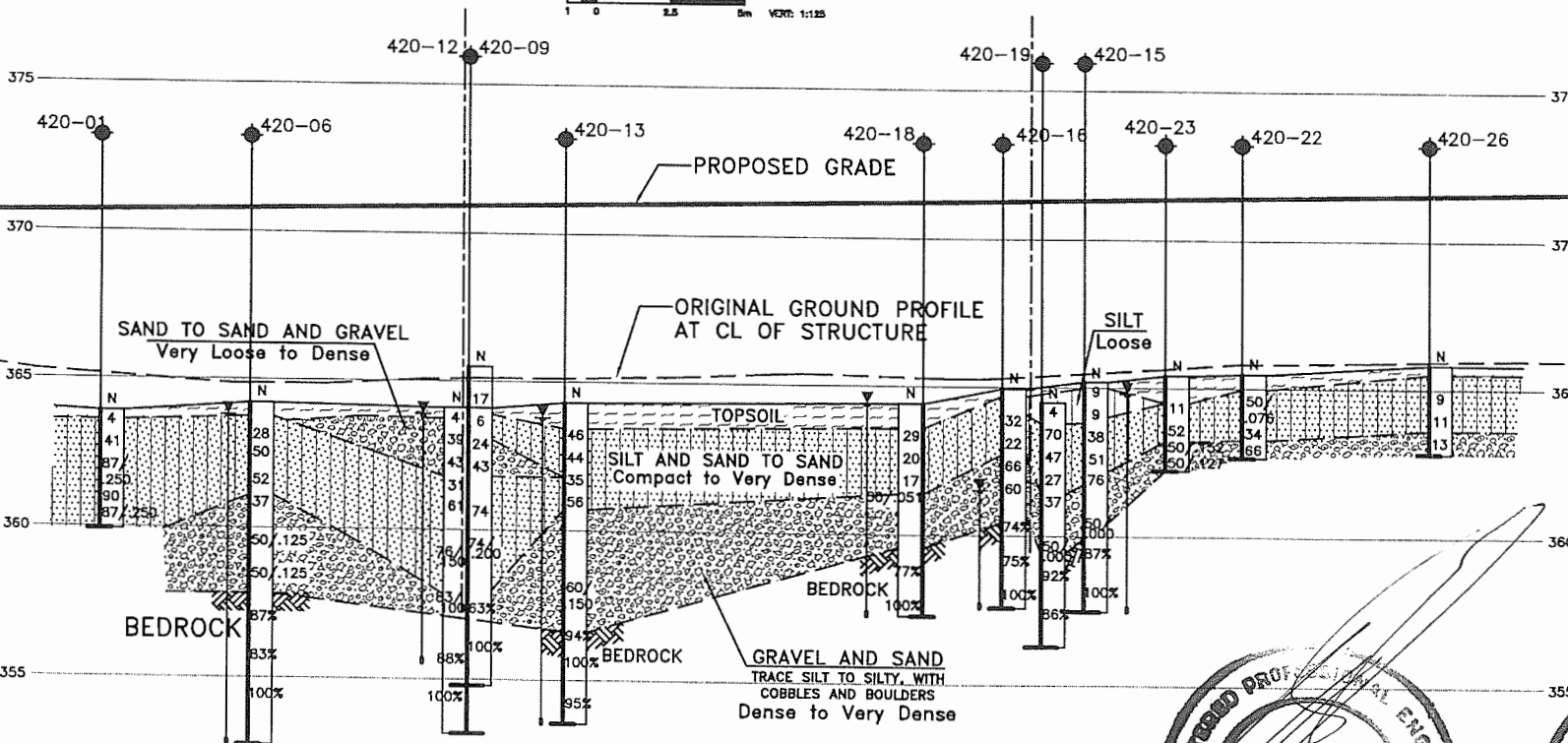
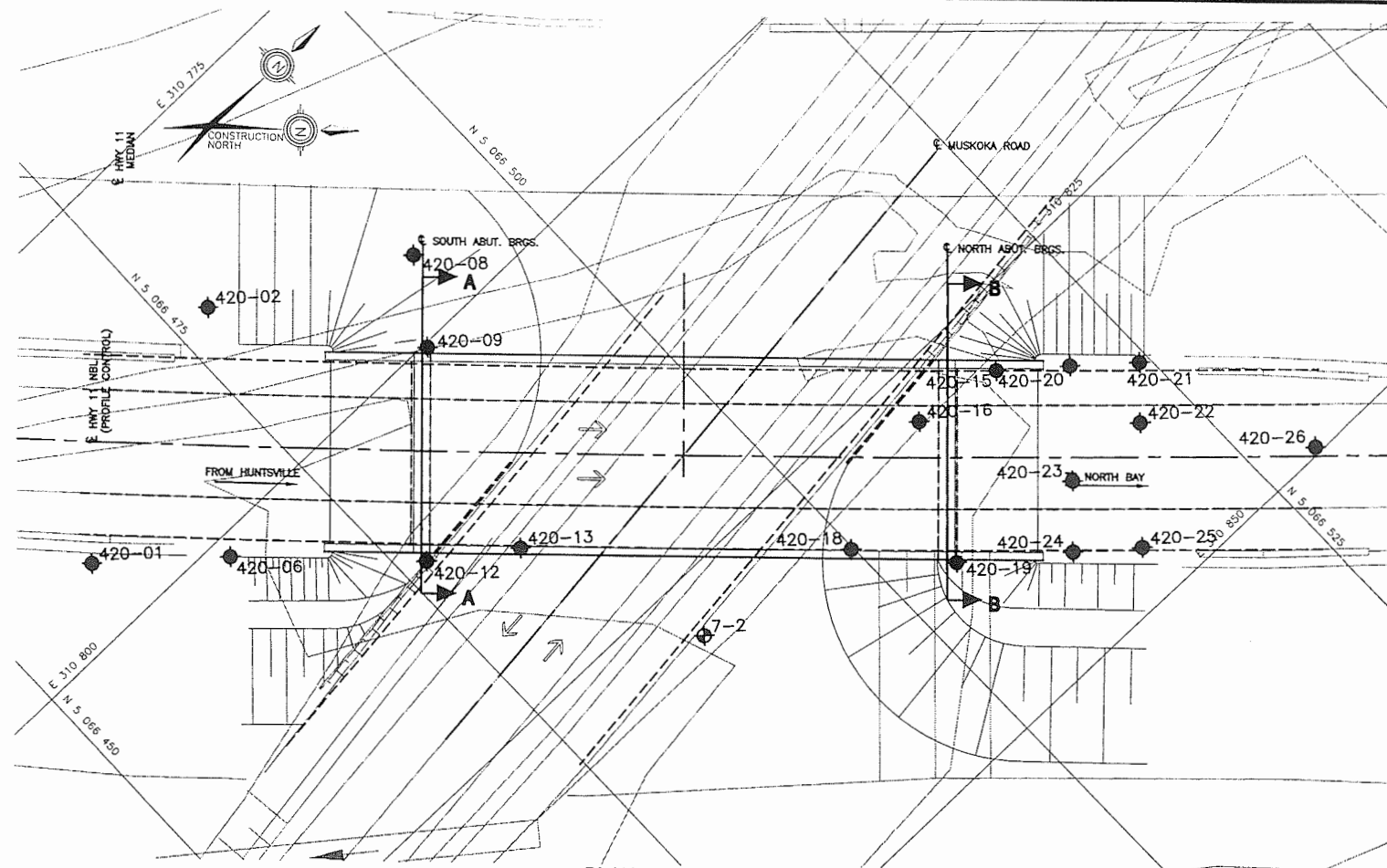


Figure F8

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

Appendix G

Drawings



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

HWY 11
CONT No
WP No 758-93-02

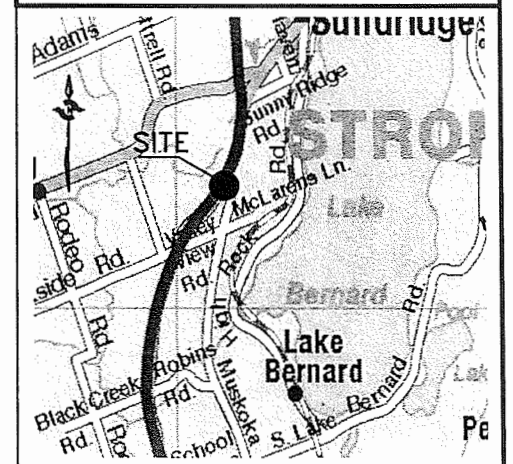
MUSKOKA ROAD
OVERPASS NBL

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Marshall Macklin Monaghan
CONSULTING ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.

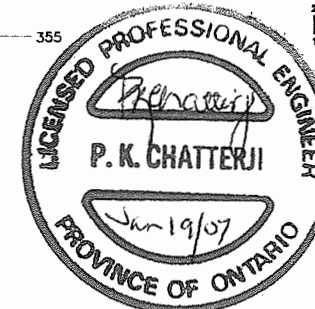
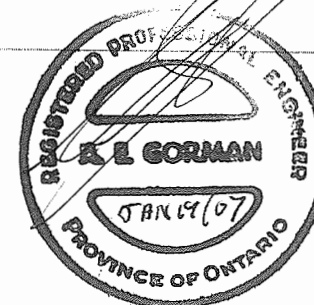


LEGEND

- Borehole by THURBER
- ⊕ Dynamic Cone Penetration Test (cone)
- ⊖ Borehole by GOLDER
- 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
420-1	363.9	5 066 459.0	310 795.0
420-2	366.7	5 066 478.0	310 787.0
420-6	364.2	5 066 466.8	310 801.5
420-8	366.7	5 066 491.0	310 795.0
420-9	365.5	5 066 487.6	310 800.3
420-12	364.1	5 066 487.9	310 811.4
420-13	364.3	5 066 482.5	310 815.4
420-15	365.2	5 066 516.4	310 829.8
420-16	365.0	5 066 504.7	310 837.8
420-18	364.5	5 066 509.8	310 828.6
420-19	364.5	5 066 509.8	310 828.6
420-20	365.2	5 066 520.5	310 833.2
420-21	365.5	5 066 524.3	310 836.5
420-22	365.5	5 066 521.3	310 839.7
420-23	365.4	5 066 514.9	310 839.3
420-24	365.2	5 066 511.3	310 843.1
420-25	365.6	5 066 515.2	310 846.2
420-26	365.8	5 066 529.2	310 849.5

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



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN AEG	CHK	CODE	LOAD
DRAWN JHL	CHK	SITE	STRUCT
			SCHEME
			DWG P1