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

Geocres Number: 31E-215

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 Muskoka Road Overpass SBL structure on the proposed four-laning of Highway 11 in the Township of Strong, Ontario. A previous, preliminary foundation investigation was carried out by Golder Associates Ltd. (Golder) and the factual data from that investigation has been used as a reference in preparing 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 investigation.

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

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

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between April 13 and 28, 2004 and consisted of drilling and sampling twelve boreholes to depths ranging from 3.7 m to 10.5 m. The boreholes were numbered 420-27, 420-29, 420-31, 420-34, 420-38, 420-39, 420-40, 420-42, 420-45, 420-46, 420-51 and 420-52 and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F. Some of these boreholes were drilled from the shoulders and travelled lanes of existing Highway 11.

As described later in the report, the investigation encountered shallow bedrock. Under the Terms of Reference, a total of six sampled boreholes are required at each foundation element. At this site, it was not possible to drill that number of boreholes within or close to the foundation footprints due to the combination of existing highway embankment slopes and the presence of existing utility lines.

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

A combination of hollow-stem auger drilling techniques and casing and washboring methods were used to advance the boreholes and samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. The boreholes at the foundation elements were also advanced 2.9 m to 3.6 m into bedrock by NQ size diamond coring techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. At each foundation element a standpipe piezometer consisting of 19 mm PVC pipe with a slotted screen was installed and enclosed in filter sand to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1.

Table 3.1 – Piezometer Installation Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
420-31 South Abutment	7.6/358.3	Piezometer with 1.52 m slotted screen installed with sand filter to 5.8 m, bentonite seal from 5.8 m to 0.6 m and drill cuttings from 0.6 m to ground surface.
420-34 South Pier	7.5/358.81	Piezometer with 1.52 m slotted screen installed with sand filter to 5.7 m, bentonite seal from 5.7 m to 0.6 m and drill cuttings from 0.6 m to ground surface.
420-45 North Pier	7.1/358.6	Piezometer with 1.52 m slotted screen installed with sand filter to 5.5 m, bentonite seal from 5.5 m to 0.15 m and drill

		cuttings from 0.15 m to ground surface.
420-46 North Abutment	8.5/358.5	Piezometer with 1.52 m slotted screen installed with sand filter to 6.7 m, bentonite seal from 6.7 m to 0.3 m and concrete grout from 0.3 m to ground surface.

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 Oakville laboratory for further examination and testing.

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

4 LABORATORY TESTING

All the 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. The boreholes drilled by Golder as part of the preliminary investigation are included in Appendix C. Details of the encountered soil and rock stratigraphy are presented in these appendices and on the Borehole Locations and Soil Strata drawings in Appendix F. 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 3.9 m to 7.6 m of overburden soils overlying Pre-Cambrian bedrock. The overburden soils generally consist of sand and gravel fill, sands and silts, sand some gravel and occasional cobbles and boulders.

5.1 Pavement and Granular Fill

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

west, east
I thought
north &
south

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

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

SPT 'N' values ranged from 4 to 55 blows for 0.3 m penetration but generally, most recorded 'N" values ranged from 12 to 38. Based on these results the fill is considered to have a compact to dense relative density with occasional loose and very dense zones. The moisture content of samples from this deposit ranged from 1% to 16%.

5.2 Sandy Silt

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

Four samples from this deposit were subjected to grain size distribution tests and the results are presented in Figure B2. These results show a soil consisting of 0 to 4% gravel, 19 to 53% sand, 40 to 75% silt and 3 to 7% clay sized particles. Based on these results the deposit is essentially a cohesionless soil ranging from a sandy silt to sand and silt. The upper part of this layer contains trace rootlets and organics. Occasional cobbles were noted in this layer below a depth of 2.2 m.

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

5.3 Sand

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

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

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

5.4 Sand and Gravel with Cobbles and Boulders

In some boreholes, the sand and silt were underlain by bedrock and in the remainder by a discontinuous layer of sand and gravel with occasional cobbles and boulders at depths ranging from 2.2 m to 5.6 m below ground surface. At some of the boreholes this layer was fully penetrated and the deposit was found to extend to depths ranging from 3.9 m to 7.6 m below ground surface or from elevations ranging from Elev. 362.6 m to Elev. 359.5 m. Further west of the site at Golder's Borehole 7-4 this deposit was encountered at a depth of 0.7 m (Elev. 366.3 m) below ground surface. It should be noted that cobbles and boulders are inferred to exist in this deposit based on the resistance to augering that was observed while drilling through this deposit.

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

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

5.5 Bedrock

The overburden soils described above are underlain by gneiss bedrock. Bedrock was proved by coring at the abutment and pier locations. Table 5.1 summarizes the bedrock depth and the elevations to the top of bedrock.

TABLE 5.1 – Depth to Bedrock

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
South Abutment	420-29	7.0	359.7
	420-31	4.0	361.9
South Pier	420-34	4.3	362.0
	420-39	7.6	359.5
North Pier	420-40	5.6	361.3
	420-42	6.1	360.9
	420-45	4.0	361.7

North Abutment	420-46	5.0	362.0
	420-51	3.9	362.6

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

In Borehole 420-46 biotite rich inclusions were found in the structure of the gneiss bedrock. A biotite schist layer was encountered from 5.0 m to 5.1 m above the surface of the gneiss. In run 2 a layer of biotite was encountered from 7.0 m to 10.4 m with no recovery from 7.5 m to 7.9 m in this layer. Underlying this layer of biotite, biotite schist was encountered from 8.2 m to 8.6 m.

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

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

The unconfined compressive strength of the rock cores is estimated to range between 59 and 148 MPa indicating strong to very strong intact rock. In Borehole 420-46 the gneiss bedrock immediately above the biotite layer is considered to be moderately strong based on an estimated unconfined compressive strength value of 23 MPa. These estimated rock strength values are based on point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Table B1 in Appendix B.

5.6 Water Levels

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

Table 5.2: Water Level Measurements

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

* Piezometers Destroyed

Based on these observations, local groundwater levels exist at Elevations 363.7 to 365.5. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

6 MISCELLANEOUS

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

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

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

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

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach embankments for the proposed structure.

It is understood that Highway 11 SBL will cross over Muskoka Road via a three span structure with a 29 m central span and two 16 m spans between the abutments and piers. Muskoka Road will be realigned south of its existing alignment to pass under the new Highway 11 SBL at Sta. 18 + 063.

At the south abutment, the finished grade of Highway 11 will be at Elevation 370.8 and the existing ground surface averages Elevation 366.3, resulting in an approach embankment approximately 4.5 m high. At the north abutment, the finished grade of Highway 11 will be at Elevation 372.0 and the existing ground surface averages Elevation 366.8, resulting in an approach embankment approximately 5.2 m high.

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.

8 STRUCTURE FOUNDATIONS

The proposed bridge is a multi-span overpass structure with two piers and two abutments as foundation elements.

The stratigraphy encountered at the foundation elements consist of 3.9 m to 7.6 m of overburden soils overlying bedrock. The overburden consists of sand and gravel fill of the existing highway underlain by native sands and silts and cobbles and boulders. Bedrock surface is uneven and sloping at the south abutment and south pier locations and more uniform at the north pier and abutment. The groundwater level ranges from Elevation 364.3 m to Elevation 365.5 m.

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 - General

This site presents the following particular problems with respect to the use of spread footings:

- Variable soil stratigraphy and zones of loose to compact soil
- High groundwater table
- Shallow bedrock but variable depth to the top of bedrock.

While the soils encountered are, in some locations and in the undisturbed condition, suitable to support spread footings, there are practical difficulties for both design and construction.

In terms of design, the variable bedrock levels can result in a footing having one end virtually resting on bedrock and the other end underlain by 4 to 5 m of soil over the bedrock. This situation will result in virtually no settlement where the footing rests on bedrock and possibly 15 to 20 mm of settlement under the end resting on the deeper overburden.

At the piers and possibly at the abutments, excavation to form either a conventional footing or to place an engineered fill pad must penetrate below the groundwater table. The base of such an excavation will lie within 1 m of the top of bedrock, which may create difficulties in dewatering and maintaining a stable base in the cohesionless soils encountered in the investigation.

As a consequence of the risks to the design and to the construction process, it is recommended that spread footings at the piers bear directly on bedrock or on mass concrete fill placed directly on the bedrock.

Spread footings on engineered fill pads may be possible for perched, non-integral abutment design.

Spread footings bearing on native soil are not recommended at this site.

8.2 Spread Footings on Engineered Fill

At the abutments, spread footings may be founded on engineered fill pads. If an engineered fill pad is used at this site, all fill, topsoil, organics and loose/soft soils should be stripped from below the footing area as shown in Figure 1 and the native soil should be stripped at least to the elevations in Table 8.1, and deeper if required to achieve the minimum thickness of engineered fill and desired founding grade. However, excavation should not penetrate into bedrock except to prepare a bench to facilitate compaction of the engineered fill.

Table 8.1 – Maximum Elevation for Engineered Fill

Foundation Element	Borehole	Depth below existing ground surface(m)	Elevation (m)
South Abutment	420-29 and 420-31	2.4 to 3.2	363.5
North Abutment	420-46 and 420-51	1.5 to 2.0	365

364.

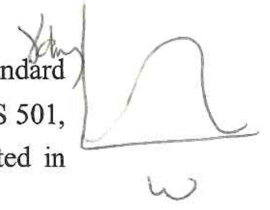
Do we need to subexcavate this deep

N=17/18

$\gamma_{wet} = \gamma_d(1+w)$

The thickness of the fill pad must be at least 2 m both to provide sufficient stress distribution and to facilitate compaction where the fill is placed over bedrock.. The founding surface for the engineered fill placed on soil should be recompacted. 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.

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



Provided a minimum footing width of 2 m is maintained, a footing bearing on a compacted Granular 'A' pad may be designed for the concentric, vertical geotechnical resistances given in Table 8.2.

Are we going to subexcavate at the pier?

Table 8.2 – Bearing Resistances on Engineered Fill

	Perched Abutment (assumed to be 1 footing width above the groundwater table)	Pier (assumed to be below the groundwater level)
Factored ULS	900 kPa	600 kPa
SLS	350 kPa	250 kPa

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 maximum total settlements of up to 20 mm. This settlement is expected to be substantially complete by the end of construction.

At the north pier and north abutment, the differential settlement across the width of the structure is not expected to exceed 12 mm. However, at the south pier in particular, the

incomplete

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.

The groundwater level is relatively close to the ground surface and excavations to achieve the elevations given in Table 8.1 will penetrate the water table. Excavation and dewatering are dealt with elsewhere in this report.

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 1.5 to 2.3 m below the underside of the footing at the north pier. At the south pier where sloping bedrock was encountered the depth to bedrock will vary from 1.0 to about 3.5 m below the underside of the footing.

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 5,000 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 5,000 kPa, provided the concrete fill will safely support this loading. It is recommended that the fill consist of 30 MPa concrete and that the plan dimensions of the fill be at least 0.6 m larger than the footing dimensions in all directions to mitigate stress concentrations in the unreinforced concrete. The SLS condition will not govern for a footing bearing on mass concrete as described herein.

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

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

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

8.4 Caissons

Caissons are not considered to be a viable alternative at this site due to the shallow cohesionless soil and the high groundwater table.

8.5 Steel Piles Driven to Bedrock

The foundations may be supported on steel H-piles driven to bedrock. However, at the piers, the underside of foundation is close to bedrock and piles are not considered feasible in these locations.

The stratigraphy encountered at the site consists of relatively thin overburden deposits overlying bedrock and the top of bedrock is sloping especially at the south abutment location. Table 8.3 below gives details on the natural bedrock elevations and the estimated pile lengths.

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	420-29	7.0	359.7	365.7*	6.0
	420-31	4.0	361.9		3.8
North Abutment	420-46	5.0	362.0	366.8*	4.8
	420-51	3.9	362.6		4.2

* From the General Arrangement Drawing

The pile lengths in Table 8.3 are considered acceptable for piles supporting a conventional or semi-integral abutment.

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 lengths of 3.8 and 4.2 m shown in Table 8.3 are considered too short and there is a recognized risk that the variability in the bedrock surface may result in some shorter piles.

The recommended minimum pile length below the abutment is 5 m, consisting of 3 m in loose sand and a minimum of 2 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.

An integral abutment structure may be designed at this site if the foundation area is prepared as follows:

1. Excavate bedrock to form a trench 2 m wide, centred on the bridge bearings and extending 1 m beyond the edge of the structure to either side. The base of the trench must be at least 5 m below the underside of the abutment.
2. Backfill the trench up to a level 3 m below the underside of the abutment using OPSS Granular "A" compacted in accordance with OPSS 501.

Proceed with normal integral abutment construction, driving the piles through the granular backfill to seat on bedrock..

✓

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 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 provided the site is prepared according to the recommendations in Section 8.4 of this report.

The integral abutment design requires that the piles posses flexibility in the upper 3 m of the pile length. At this site, the upper 3 m of the pile length will lie in very loose sandy silt which, in its original state, would provide sufficient flexibility. However, if the upper 3 m of the piles lies in compacted fill or if the native soil became compacted by the construction processes, the required flexibility may be compromised. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by one of the following systems:

- For a "true 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%

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:

$$k_s = n_h \, . \, z / D \qquad (\text{kN/m}^3)$$

$$p_{ult} = 3 \, . \, \gamma \, . \, z \, . \, K_p \qquad (\text{kPa})$$

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

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

Table 8.5 – Recommended Soil Parameters

Area Reference Borehole No	Applicable Elevation	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (kN/m^3)
North Abutment BH 420-46	367.0 -365.6	Gran. Fill	21.2	30	4000
	365.6-362.9	Silty Sand	20	30	4000
	362.9-362.0	Sand some Gravel	20	32	10000
North Abutment BH 420-51	367.0-365.2	Gran. Fill	21.2	30	4000
	365.2-364.3	Silty Sand	20	30	4000
	364.3-362.6	Sand some Gravel	20	32	10000
South Abutment BH 420-29	366.0-364.5	Gran. Fill	21.2	30	4000
	364.5-361.2	Silt and Sand	19	30	4000
	361.2-359.7	Sand	19	32	10000
South Abutment BH 420-31	366.0-364.5	Gran. Fill	21.2	30	4000
	364.5-361.9	Silt and Sand	19	32	4000

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, 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 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 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

Berg points

All piles must be reinforced with driving shoes in accordance with SS 103-12.

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.

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 after site preparation as described
- Piers supported on spread footings bearing directly on bedrock or on mass concrete fill bearing on bedrock.

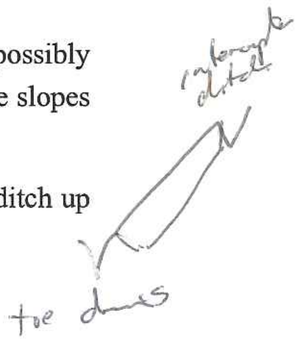
8.7 Frost Cover

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 PERMANENT CUT

The realigned Muskoka Road will lie in a shallow cut in cohesionless, sandy silt and will possibly lie below the seasonally high groundwater level. After the cut slopes have drained, the side slopes will be stable at slopes not exceeding 2H:1V.

It is recommended that gravel sheeting be applied to the cut slope from the bottom of the ditch up to Elevation 366. This treatment must extend to 20 m from the centreline of the structure.



10 EXCAVATION AND BACKFILL

10.1 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.

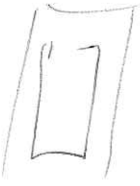
10.2 Foundations

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 under the groundwater table and terminate on an uneven bedrock surface.

NSSP

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.

11 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.

12 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 4 m to 5 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.

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 1.4.

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

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

Settlements within Fills

13 RETAINED SOIL SYSTEMS

Appearance
Performance
Geometry

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". 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.

13.1 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 must be founded at or below elevation given in Table 8.1 for engineered fill. Alternatively, the RSS may be founded on a pad of Granular "A" engineered fill founded at the elevations given in Table 8.1.

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:

- Ultimate coefficient of sliding resistance of cast in-situ concrete levelling pad on Granular A = 0.7
- 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.

13.2 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.

make assumptions and use drawing

14 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.

15 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. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

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

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

P_h = horizontal pressure on the wall at depth h (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 15.1 below.

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.

Table 15.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°; \gamma = 22.8\ kN/m^3$		OPSS Granular B Type I $\phi = 32°; \gamma = 21.2\ kN/m^3$		Rock Fill $\phi = 42°; \gamma = 19.0\ kN/m^3$	
	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.

The factors in the Table 15.1 are "ultimate" values. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 2000.

16 SEISMIC CONSIDERATIONS

16.1 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.

16.2 Liquefaction Potential

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method[1]

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 and piers. Therefore, the vertical geotechnical resistance of these footings 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.

16.3 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 0.5 ϕ. For the design of retaining walls, the coefficients of horizontal earth pressure in Table 16.1 may be used:

Table 16.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
	Granular A or Granular B Type II $\phi = 35°; \delta = 17.5°$ $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°; \delta = 16°$ $\gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42°; \delta = 21°$ $\gamma = 19.0$ kN/m^3	
Wall Condition	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	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

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[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.

17 CONSTRUCTION CONCERNS

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

- Stability of temporary excavations
- Control of groundwater and prevent of loss of fines around sheeted excavations

18 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

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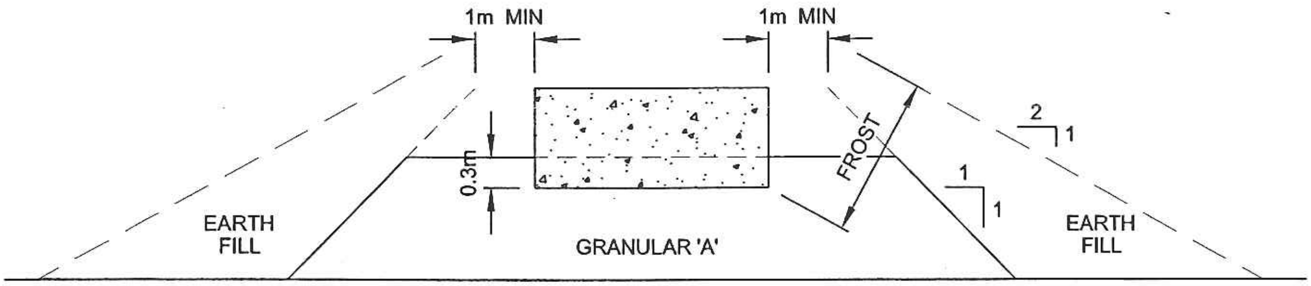
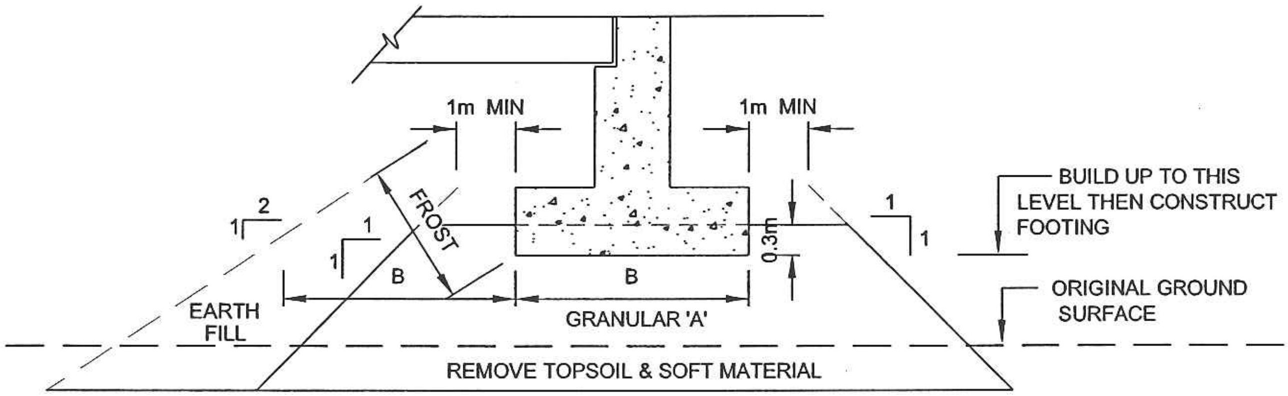
Oct 18/04

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

March , 2004



CROSS-SECTION



LONGITUDINAL SECTION

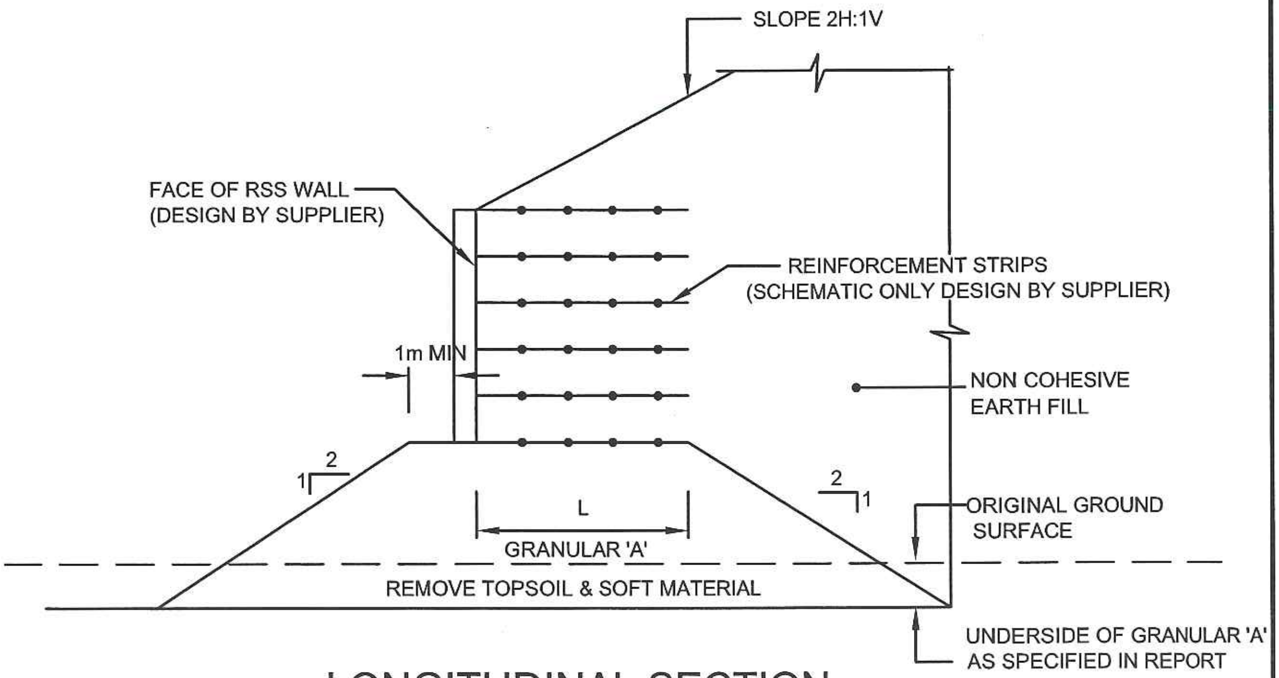
NOT TO SCALE

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.

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

TED35146.DWG



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

THURBER

DWG. NO. FIGURE 2

March , 2004

TED35146 .DWG

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

SYMBOLS

▨	CLAYSTONE
▤	SILTSTONE
░	SANDSTONE
■	COAL
▩	Bedrock (general)

DISCONTINUITY SPACING

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

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

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

RECORD OF BOREHOLE No 420-27

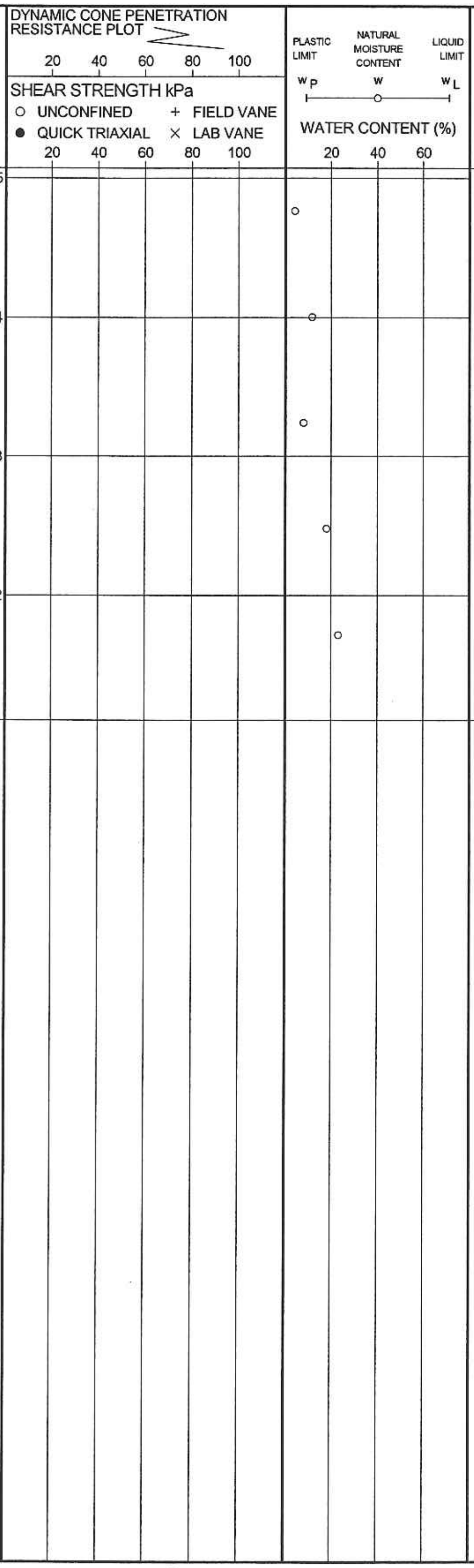
1 OF 1 METRIC

W.P. ____
LOCATION N 5066508.8 E 310779.0 Muskoka Road Overpass (SBL)
ORIGINATED BY GA

HWY 11
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY SS

DATUM Geodetic
DATE 16.04.04 - 16.04.04
CHECKED BY AEG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
365.0												
0.0	**SAND and GRAVEL** Compact Brown Dry to Moist (FILL)		1	SS	19		365					
			2	SS	14		364					
			3	SS	4		363					
362.8 2.2	**SILT**, some sand to sandy, trace clay Compact Grey- Brown Moist to Wet		4	SS	20				0	19	75	7
			5	SS	17		362					
361.0 4.0	END OF BOREHOLE AT 3.96m. AUGER REFUSAL AT 3.96m PROBABLY ON COBBLES OR BOULDERS. WATER LEVEL IN OPEN BOREHOLE AT 2.4m DEPTH UPON COMPLETION.											



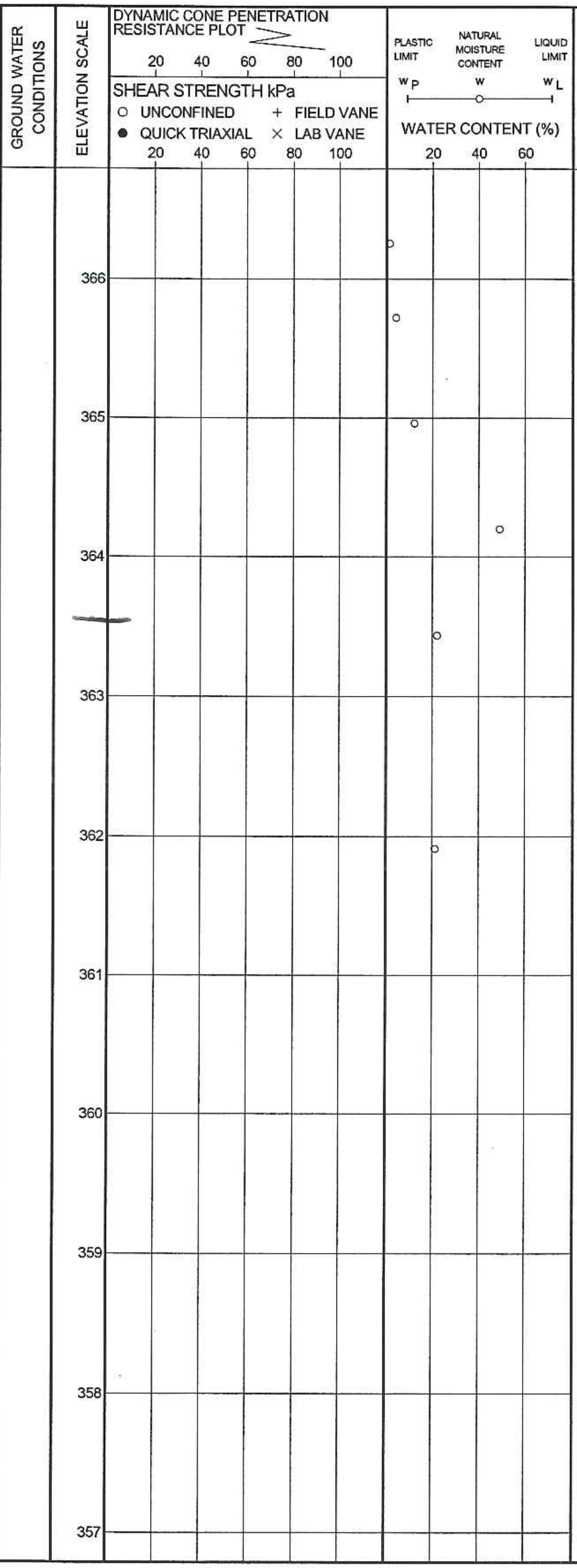
+³, ×³: Numbers refer to Sensitivity   15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 420-29

1 OF 2 **METRIC**

W.P.	LOCATION N 5066519.8 E 310798.5	Muskoka Road Overpass (SBL)	ORIGINATED BY GA
HWY 11	BOREHOLE TYPE Hollow Stem Augers/ NQ Coring		COMPILED BY SS
DATUM Geodetic	DATE 15.04.04 - 15.04.04		CHECKED BY AEG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
366.7						
0.0 366.4	**ASPHALT** (250mm)					
0.2	**SAND and GRAVEL** Compact, Brown, Dry (FILL)	1	SS	29		
366.0 0.7	**SAND**, trace gravel Compact to Very Dense Brown Dry to Moist (FILL)	2	SS	29		
		3	SS	55		
364.5 2.2	**SAND**, some silt, occasional rootlets and trace organics to 2.9m Compact Brown Moist to Wet	4	SS	18		
		5	SS	22		0 86 14 (SI+CL)
		6	SS	24		
361.2 5.5	**SAND** with cobbles and boulders Very Dense inferred Brown Moist	7	SS	50/.00		
359.7 7.0	AUGER REFUSAL AT 7.01m. **GNEISS (BEDROCK)** Fresh to slightly weathered, massive, grey- white with black blotches, strong	1	RUN		FI 0 0 0 0 0	RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=95MPa
		2	RUN		0 0 0 0 0 0	RUN 2# TCR=98%, SCR=98%, RQD=98%, UCS=78MPa



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 420-29

2 OF 2

METRIC

W.P.
HWY 11
DATUM Geodetic
LOCATION N 5066519.8 E 310798.5 Muskoka Road Overpass (SBL)
BOREHOLE TYPE Hollow Stem Augers/ NQ Coring
DATE 15.04.04 - 15.04.04
ORIGINATED BY GA
COMPILED BY SS
CHECKED BY AEG

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L		REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
356.6 10.1	END OF BOREHOLE AT 10.1m. BOREHOLE FILLED WITH DRILL WATER UPON COMPLETION OF CORING.													
							356							

ONTMT4 420MUSKOKA-I.GPJ 15/10/04

+ ³, × ³: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 420-31

1 OF 1

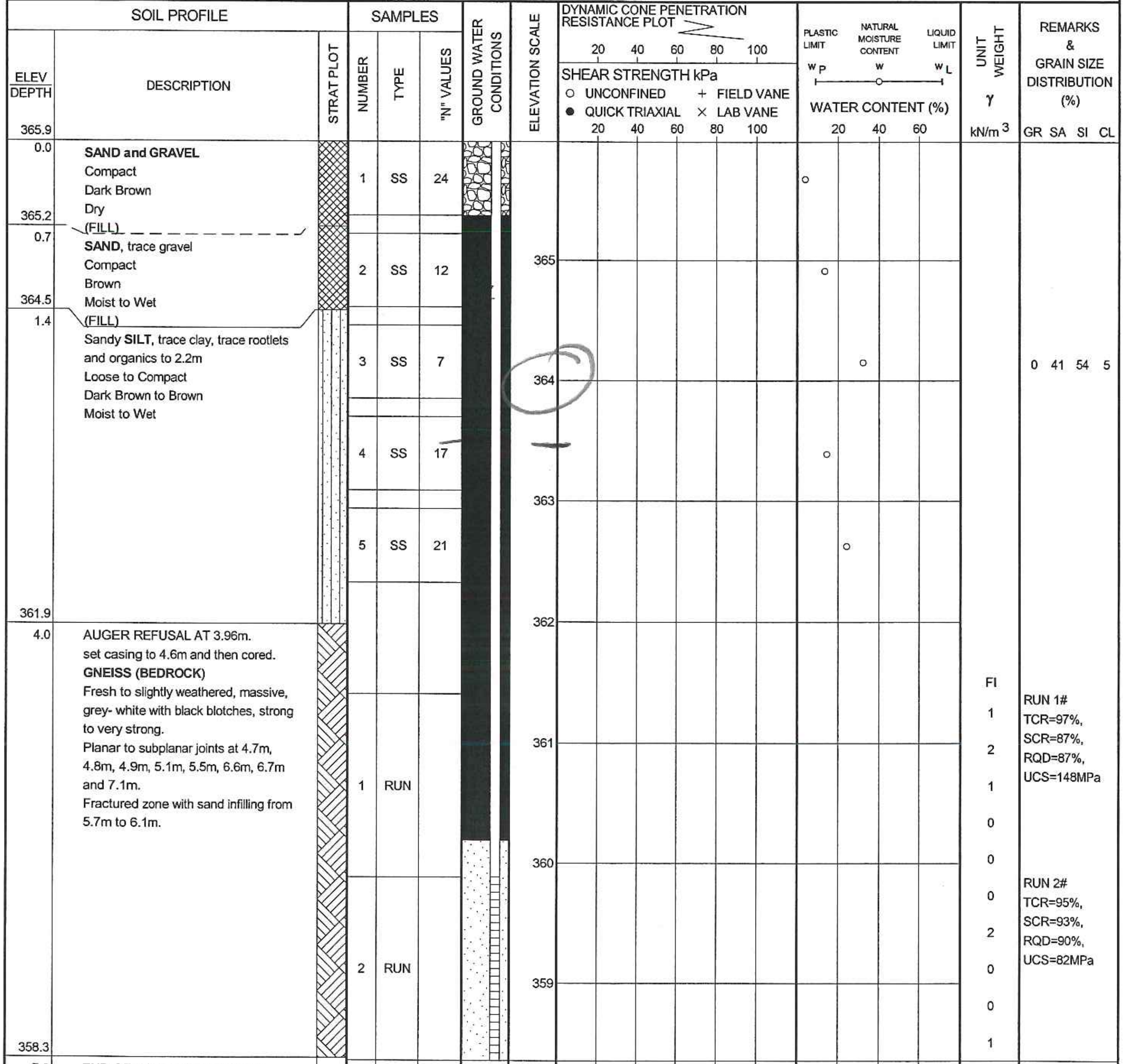
METRIC

W.P.		ORIGINATED BY	GA
HWY	11	COMPILED BY	SS
DATUM	Geodetic	CHECKED BY	AEG

LOCATION N 5066522.7 E 310788.1 Muskoka Road Overpass (SBL)

BOREHOLE TYPE Hollow Stem Augers/ NQ Coring

DATE 15.04.04 - 15.04.04



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
365.9						
0.0	**SAND and GRAVEL** Compact Dark Brown Dry (FILL)	1	SS	24		
365.2 0.7	**SAND**, trace gravel Compact Brown Moist to Wet (FILL)	2	SS	12		
364.5 1.4	Sandy **SILT**, trace clay, trace rootlets and organics to 2.2m Loose to Compact Dark Brown to Brown Moist to Wet	3	SS	7		0 41 54 5
		4	SS	17		
		5	SS	21		
361.9 4.0	AUGER REFUSAL AT 3.96m. set casing to 4.6m and then cored. **GNEISS (BEDROCK)** Fresh to slightly weathered, massive, grey- white with black blotches, strong to very strong. Planar to subplanar joints at 4.7m, 4.8m, 4.9m, 5.1m, 5.5m, 6.6m, 6.7m and 7.1m. Fractured zone with sand infilling from 5.7m to 6.1m.	1	RUN		FI 1 2 1 0 0	RUN 1# TCR=97%, SCR=87%, RQD=87%, UCS=148MPa
		2	RUN		0 2 0 0 1	RUN 2# TCR=95%, SCR=93%, RQD=90%, UCS=82MPa
358.3 7.6	END OF BOREHOLE AT 7.62m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS:					

DATE	DEPTH(m)	ELEVATION(m)
16/04/04	1.3	364.6

ONTMT4 420MUSKOKA-I.GPJ 15/10/04

+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

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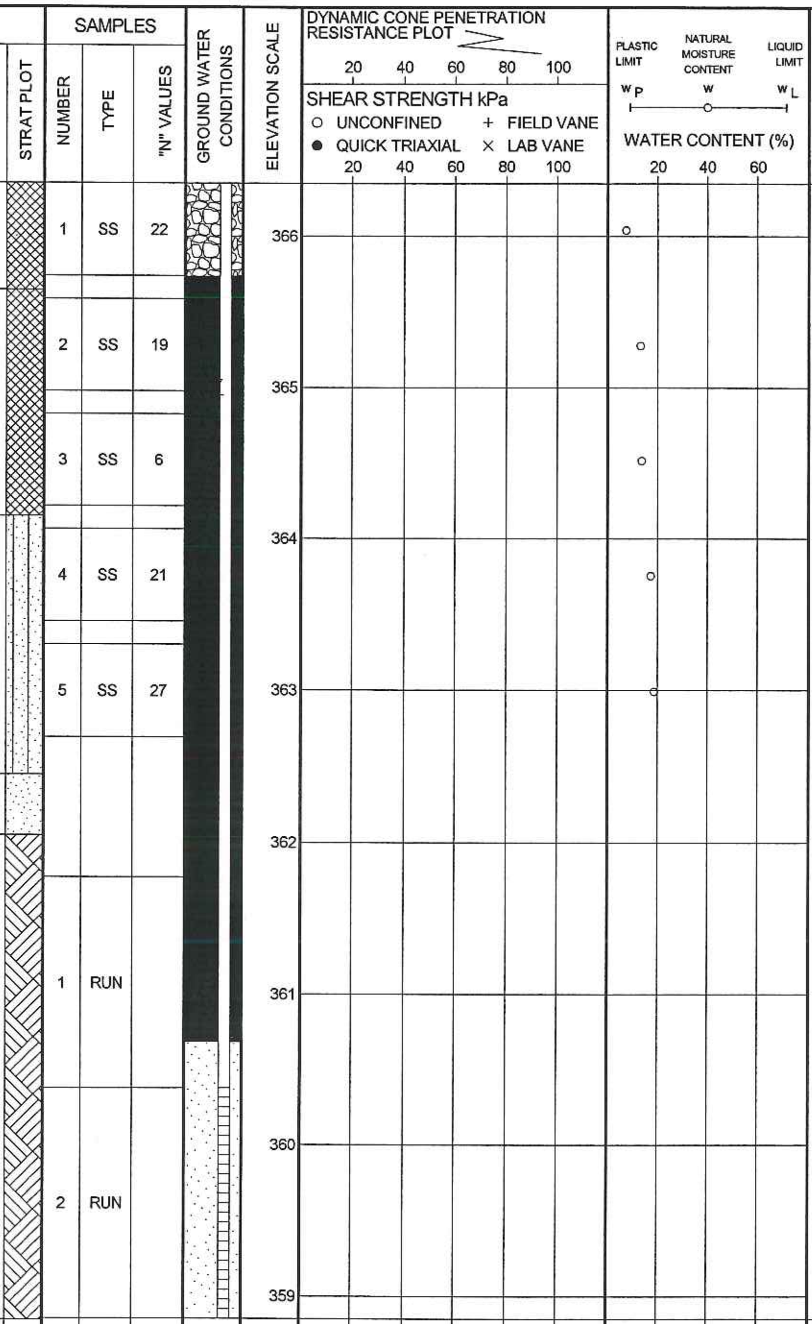
THURBER

RECORD OF BOREHOLE No 420-34

1 OF 1 **METRIC**

W.P.		LOCATION N 5066536.5 E 310796.8 Muskoka Road Overpass (SBL)	ORIGINATED BY GA
HWY	11	BOREHOLE TYPE Hollow Stem Augers/ NQ Coring	COMPILED BY SS
DATUM	Geodetic	DATE 15.04.04 - 15.04.04	CHECKED BY AEG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
366.3							
365.6 0.0	**SAND and GRAVEL** Compact, Dark Brown, Moist (FILL)	1	SS	22	366		
364.1 0.7	**SAND**, trace gravel, trace silt Compact to Loose Brown Moist to Wet (FILL)	2	SS	19			5 91 5 (SI+CL)
		3	SS	6	365		
362.4 2.2	Silty **SAND** Compact Brown Wet	4	SS	21	364		
		5	SS	27	363		0 78 22 (SI+CL)
362.0 3.9	**SAND** cobbles and boulders inferred						
358.8 4.3	AUGER REFUSAL AT 4.26m AND CASING ADVANCED INTO BEDROCK. commence coring from 4.57m. **GNEISS (BEDROCK)** Fresh, massive, grey- white with black blotches, strong to very strong. Subplanar joints from 4.8m to 5.1m.	1	RUN		362 361	FI 0 1 0 0	RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=107MPa
		2	RUN		360 359	0 0 0 0 0	RUN 2# TCR=97%, SCR=97%, RQD=97%, UCS=95MPa
7.5	END OF BOREHOLE AT 7.49m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEVATION (m) 16/04/04 1.4 364.9						



+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

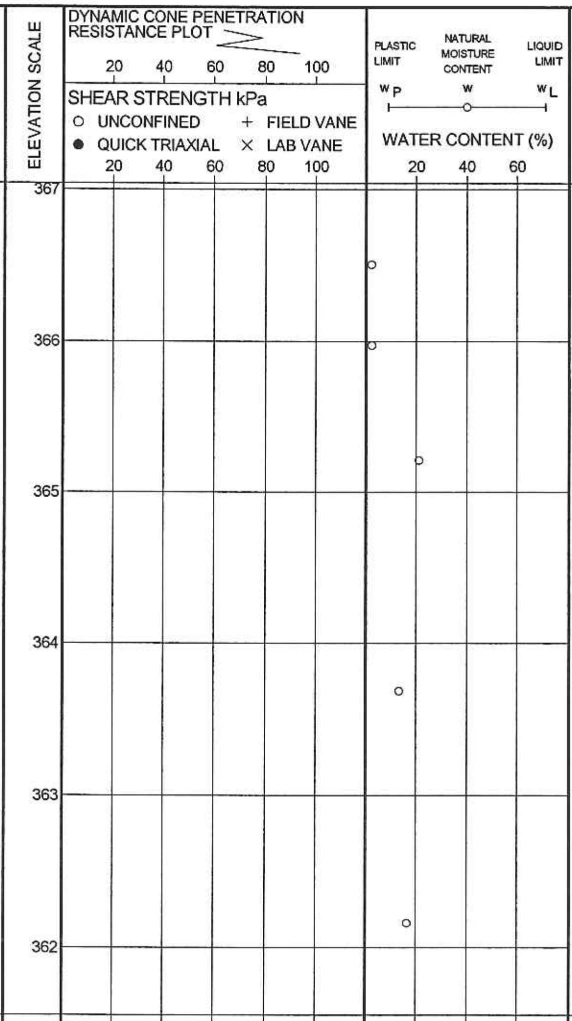
RECORD OF BOREHOLE No 420-38

1 OF 1

METRIC

W.P. ______ LOCATION N 5066527.3 E 310806.5 Muskoka Road Overpass (SBL) ORIGINATED BY GA
HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 13.04.04 - 13.04.04 CHECKED BY AEG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m3	GR	SA	SI	CL
366.9											
0.0 366.7	**ASPHALT** (230mm)	■									
0.2	**SAND and GRAVEL** Compact, Brown, Dry (FILL)	▨	1	SS	28						
366.2 0.7	**SAND**, trace gravel Compact Brown Dry (FILL)	▨	2	SS	30						
365.5 1.4	**SAND and SILT**, trace gravel, trace clay Loose to Compact Brown Moist to Wet		3	SS	8			4	53	40	3
			4	SS	26						
	occasional cobbles below 2.2m		5	SS	17						
	Silty		6	SS	14			0	42	55	3
361.4 5.5	END OF BOREHOLE AT 5.48m. AUGER REFUSAL AT 5.48m ON PROBABLE BEDROCK OR BOULDERS. WATER LEVEL AT 3.0m ON COMPLETION.										



+3, ×3: Numbers refer to Sensitivity

20 15-⊕-5 10 (%) STRAIN AT FAILURE

ONTMT4 420MUSKOKA-I.GPJ 15/10/04

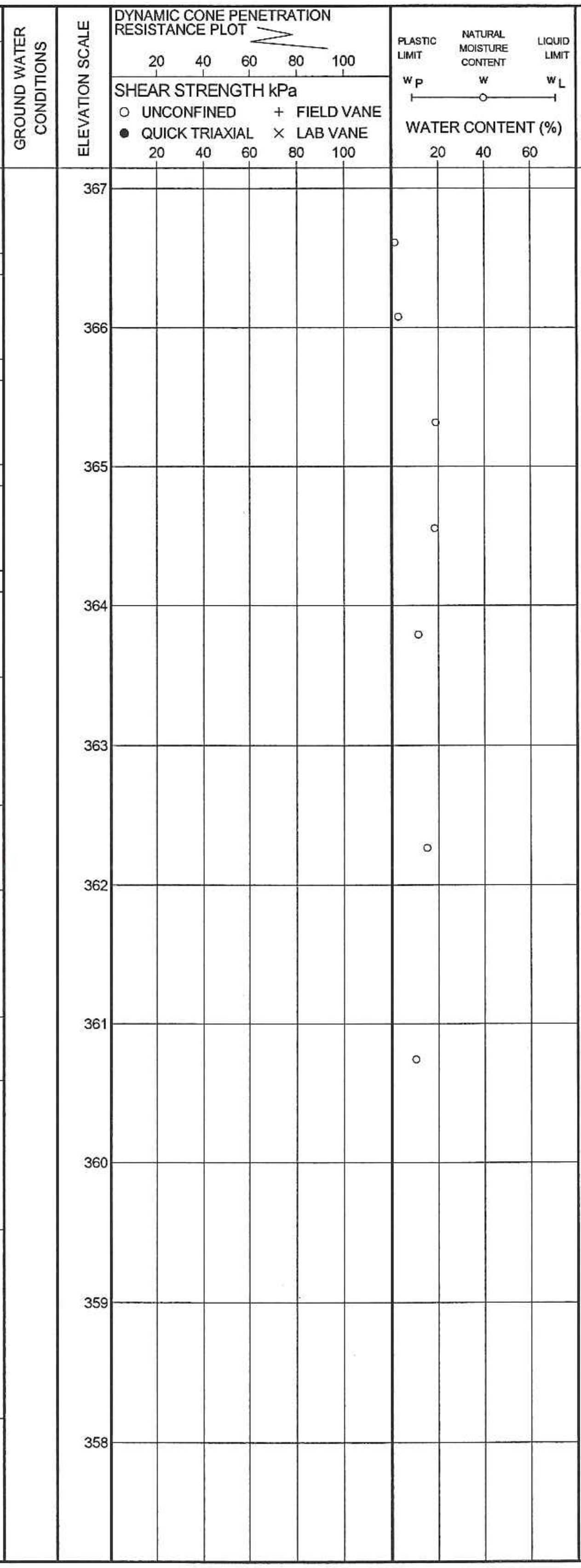
RECORD OF BOREHOLE No 420-39

1 OF 2

METRIC

W.P.		ORIGINATED BY	GA
HWY	11	COMPILED BY	SS
DATUM	Geodetic	CHECKED BY	AEG
LOCATION	N 5066531.0 E 310810.0 Muskoka Road Overpass (SBL)		
BOREHOLE TYPE	Hollow Stem Augers/ NQ Coring		
DATE	13.04.04 - 13.04.04		

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
367.1						
366.8 0.0	**ASPHALT** (200mm)					
0.2	**SAND and GRAVEL** Dense, Brown, Dry (FILL)	1	SS	32		
366.4 0.7	**SAND**, trace gravel Dense Brown Dry (FILL)	2	SS	38		
365.7 1.4	**SAND and SILT** to sandy **SILT**, trace gravel Compact Brown Moist to Wet	3	SS	25		
		4	SS	11		
	occasional cobbles	5	SS	12		
		6	SS	20		
361.5 5.6	**SAND**, some gravel, occasional cobbles and boulder Very Dense Brown Moist to Wet	7	SS	68		
359.5 7.6	AUGER REFUSAL AT 7.62m. **GNEISS (BEDROCK)** Weathered to slightly weathered, massive, grey-white with black blotches, strong. Vertical to subvertical joints at 8.5m to 9.0m and 9.7m to 9.9m. Fractured zone from 7.6m to 7.7m	1	RUN		FI <5 5 1 <5	RUN 1# TCR=100%, SCR=75%, RQD=45%, UCS=74MPa
		2	RUN		<10 1 0 <5	RUN 2# TCR=100%, SCR=100%, RQD=83%, UCS=54MPa



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 420-39 2 OF 2 METRIC

W.P. ______ LOCATION N 5066531.0 E 310810.0 Muskoka Road Overpass (SBL) ORIGINATED BY GA
HWY 11 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY SS
DATUM Geodetic DATE 13.04.04 - 13.04.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
356.6		▨					357			1				
10.5	END OF BOREHOLE AT 10.49m. BOREHOLE FILLED WITH DRILL WATER UPON COMPLETION OF CORING.									2				

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

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 420-40

1 OF 1

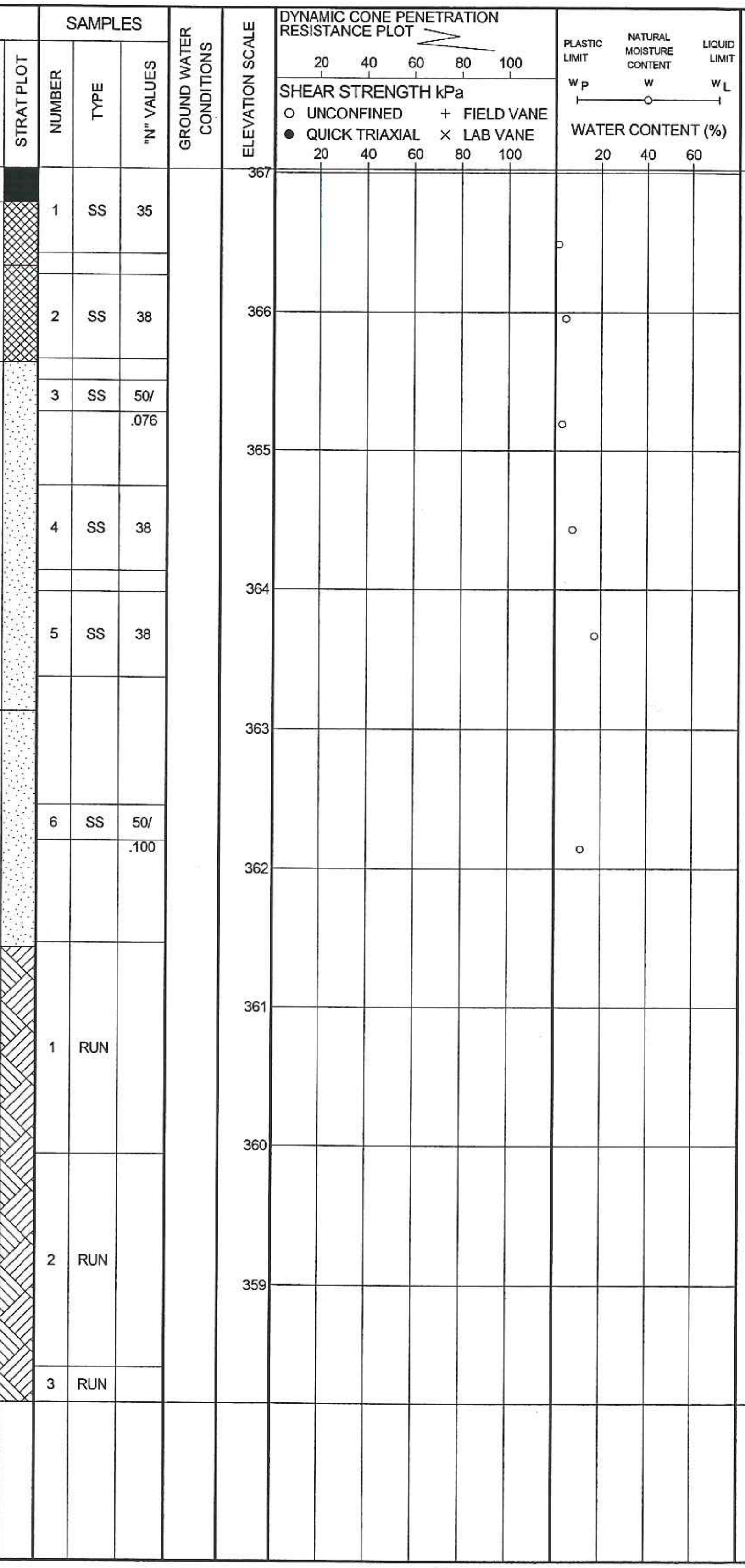
METRIC

W.P. ______
HWY 11
DATUM Geodetic
LOCATION N 5066557.7 E 310816.5 Muskoka Road Overpass (SBL)
BOREHOLE TYPE Hollow Stem Augers/ NQ Coring
DATE 16.04.04 - 16.04.04
ORIGINATED BY GA
COMPILED BY SS
CHECKED BY AEG

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
366.9							
0.0 366.7	**ASPHALT** (250mm)						
0.2	**SAND and GRAVEL** Dense, Brown, Dry (FILL)	1	SS	35			
366.2 0.7	**SAND**, trace gravel Dense Brown Dry (FILL)	2	SS	38			
365.5 1.4	**SAND**, trace gravel, trace silt, occasional cobbles and/or boulders from 1.7m to 2.1m Very Dense to Dense Brown Moist to Wet	3	SS	50/ .076			
		4	SS	38			
		5	SS	38			6 90 5 (SI+CL)
363.0 3.9	**SAND** with cobbles and boulders Very Dense Brown Moist	6	SS	50/ .100			
361.3 5.6	AUGER REFUSAL AT 5.56m. **GNEISS (BEDROCK)** Fresh to slightly weathered, massive, pinkish- white with black bands, strong Vertical joint at 7.2m to 7.5m.	1	RUN			FI 1 1 2 0	RUN 1# TCR=100%, SCR=92%, RQD=90%, UCS=64MPa
		2	RUN			1 2 1 0 0 0	RUN 2# TCR=100%, SCR=100%, RQD=85%, UCS=76MPa
358.1		3	RUN			2	RUN 3# TCR=100%, SCR=100%, RQD=60%, UCS=MPa
8.9	END OF BOREHOLE AT 8.86m. BOREHOLE FILLED WITH DRILL WATER UPON COMPLETION OF DRILLING.						

1.9

3.7



+ ³, × ³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 420-42

1 OF 1

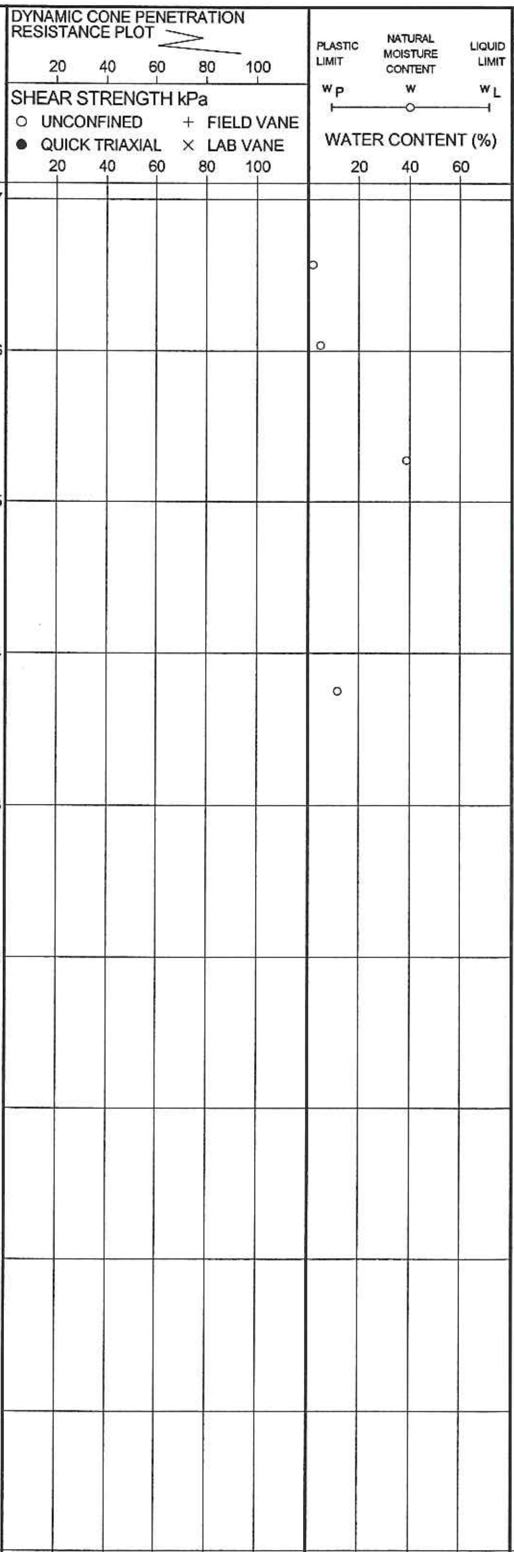
METRIC

W.P.
HWY 11
DATUM Geodetic
LOCATION N 5066554.9 E 310819.5 Muskoka Road Overpass (SBL)
BOREHOLE TYPE Hollow Stem Augers/ NQ Coring
DATE 13.04.04 - 14.04.04
ORIGINATED BY GA
COMPILED BY SS
CHECKED BY AEG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
367.0							
366.8 0.0	**ASPHALT** (230mm)				367		
366.3 0.2	**SAND and GRAVEL** Compact, Brown, Dry (FILL)	1	SS	30			
365.6 0.7	**SAND**, trace gravel Dense Brown Dry (FILL)	2	SS	33	366		
364.1 1.4	**SAND**, trace gravel, trace silt, some organics to 2.2m, occasional cobbles and/or boulders below 2.4m Compact Dark Brown Moist	3	SS	20	365		
		~~4~~	~~SS~~	~~50/~~ .00			
360.9 2.9	**SAND** some gravel, with cobbles and/or boulders Dense Brown Wet	5	SS	50/ .152	364 363 362		
358.0 6.1	**GNEISS (BEDROCK)** Fresh, massive, pinkish to grey-white with black blotches, strong. Vertical joint at 8m to 8.3m.	1	RUN		361 360	FI 0 0 0 0	RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=71MPa
	Fractured zone from 8m to 8.7m.	2	RUN		359	0 <5 >5 <5	RUN 2# TCR=100%, SCR=100%, RQD=67%, UCS=59MPa
9.0	END OF BOREHOLE AT 9.01m. BOREHOLE FILLED WITH DRILL WATER UPON COMPLETION OF DRILLING.						

1.9

4.7



+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

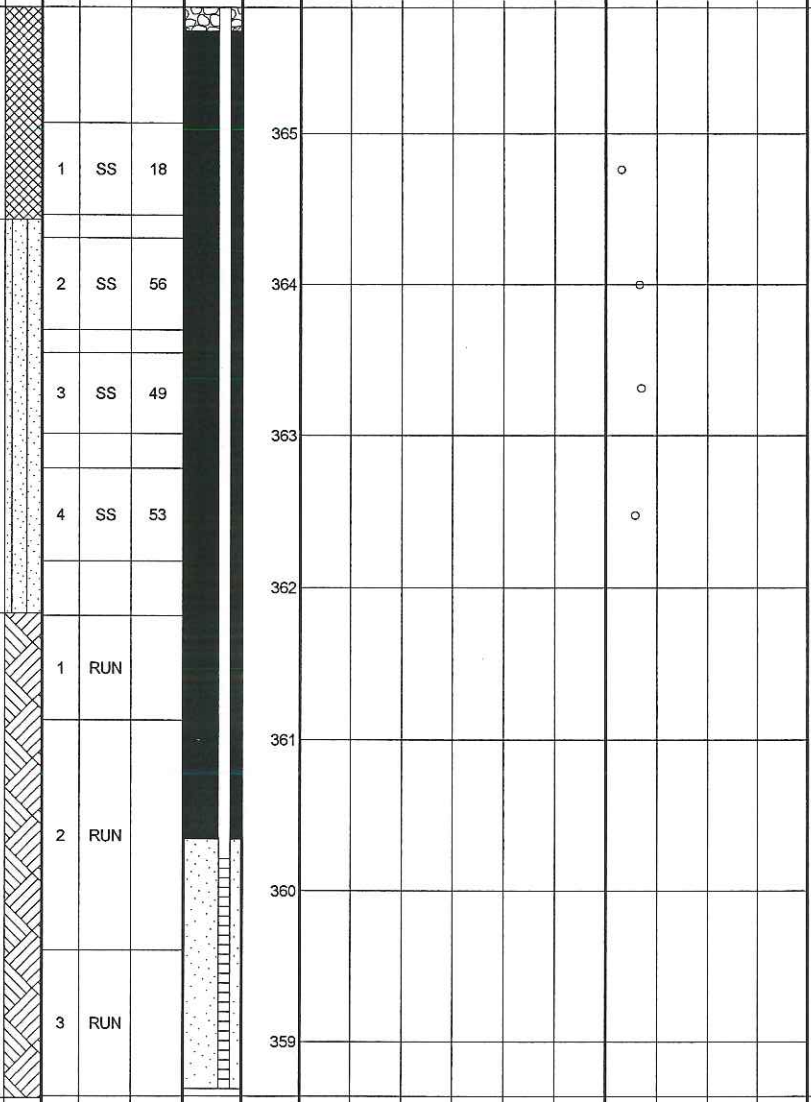
THURBER

RECORD OF BOREHOLE No 420-45

1 OF 1

METRIC

W.P.		LOCATION	N 5066552.4 E 310829.4 Muskoka Road Overpass (SBL)	ORIGINATED BY	SL
HWY	11	BOREHOLE TYPE	Hollow Stem Augers/ NQ Coring	COMPILED BY	SS
DATUM	Geodetic	DATE	27.04.04 - 27.04.04	CHECKED BY	AEG



ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
365.7 0.0	**SAND and GRAVEL** Compact Dark Brown to Brown Moist (FILL)	1	SS	18		
364.3 1.4	Silty **SAND**, trace gravel Very Dense Brown Wet	2	SS	56		
		3	SS	49		2 66 32 (SI+CL)
	occasional cobbles below 2.9m.	4	SS	53		
361.7 4.0	AUGER REFUSAL AT 4.01m. **BEDROCK (GNEISS)** Fresh, massive, pinkish to grey- white with subhorizontal black banding, strong. Vertical joint at 6.7m to 6.8m. Rubble zone from 4.0m to 4.1m.	1	RUN		3 0 1	RUN 1# TCR=96%, SCR=89%, RQD=85%, UCS=75MPa
		2	RUN		2 1 4 3	RUN 2# TCR=98%, SCR=95%, RQD=78%, UCS=60MPa
		3	RUN		3 4 4	RUN 3# TCR=100%, SCR=100%, RQD=53%, UCS=84MPa
358.5 7.2	END OF BOREHOLE AT 7.18m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS:					

DATE	DEPTH(m)	ELEVATION(m)
28/04/04	1.1	364.6
18/06/04	1.4	364.3

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 420-46

1 OF 1 **METRIC**

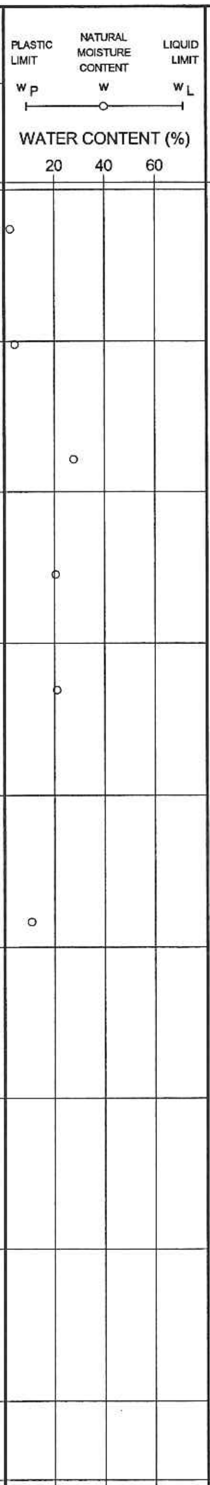
W.P.		LOCATION N 5066569.5 E 310827.4 Muskoka Road Overpass (SBL)	ORIGINATED BY GA
HWY 11		BOREHOLE TYPE Hollow Stem Augers/ NQ Coring	COMPILED BY SS
DATUM Geodetic		DATE 14.04.04 - 14.04.04	CHECKED BY AEG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
367.0					367		
0.0	**SAND and GRAVEL** Compact, Brown, Dry (FILL)	1	SS	15			
366.3 0.7	**SAND**, trace gravel, trace silt Compact Brown Dry (FILL)	2	SS	14	366		4 87 9 (SI+CL)
365.6 1.4	Silty **SAND** Compact Brown Moist to Wet	3	SS	12	365		
		4	SS	25			0 81 19 (SI+CL)
		5	SS	19	364		0 88 12 (SI+CL)
362.9 4.1	**SAND**, some gravel, occasional cobbles and/or boulders Very Dense Brown Moist	6	SS	50/ .152	363		
362.0 5.0	AUGER REFUSAL AT 5.02m. **GNEISS (BEDROCK)** Slightly weathered to fresh, massive, pinkish- white with black blotches, moderately strong to very strong. Biotite shist layer from 5.0m to 5.1m.	1	RUN		362 361	FI 2 <5 1 1 0	RUN 1# TCR=98%, SCR=93%, RQD=83%, UCS=138MPa
		2A	RUN			0 0	RUN 2A# TCR=73%, SCR=62%, RQD=62%, UCS=23MPa
359.9 7.0	Very coarse grained, black **BIOTITE** layer No recovery from 7.5m to 7.9m	2B	SS		360	0 0 0	
358.7 8.2	**BIOTITE SHIST**	3	RUN		359	1	RUN 3# TCR=100%, SCR=89%, RQD=84%, UCS=MPa
358.4 8.6	END OF BOREHOLE AT 8.55m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEVATION(m) 15/04/04 1.6 365.4 16/04/04 1.5 365.5						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

UNIT WEIGHT γ kN/m³



+ ³, × ³ : Numbers refer to Sensitivity

15-⊕-5 20 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 420-51

1 OF 1

METRIC

W.P.		LOCATION	N 5066563.9 E 310840.5 Muskoka Road Overpass (SBL)
HWY	11	BOREHOLE TYPE	Hollow Stem Augers/ NQ Coring
DATUM	Geodetic	DATE	28.04.04 - 28.04.04

ORIGINATED BY SL
COMPILED BY SS
CHECKED BY AEG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
366.5					
0.0	**SAND and GRAVEL** Compact Dark Brown Moist (FILL)	1	SS	32	
365.2 1.3	Silty **SAND** Dense Brown Moist to Wet	2	SS	34	
364.3 2.2	**SAND**, some gravel, trace silt, occasional cobbles and/or bouldes Very Dense Brown Wet auger refusal at 2.7m on probable cobbles and boulders.	3	SS	50/ .100	17 80 4 (SI+CL)
362.6 3.9	**GNEISS (BEDROCK)** Fresh, massive, pinkish- white with black blotches, strong to very strong. Subvertical joint from 6.1m to 6.2m.	1	RUN		RUN 1# TCR=100%, SCR=87%, RQD=73%, UCS=76MPa
		2	RUN		RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=98MPa
		3	RUN		RUN 3# TCR=100%, SCR=91%, RQD=61%, UCS=113MPa
359.3 7.2	END OF BOREHOLE AT 7.2m. BOREHOLE FILLED WITH DRILL WATER UPON COMPLETION OF DRILLING.				

Unit weight column (FI): 1, 2, 0, 0, 0, 0, 4, 0, 0, 3, 2

ELEVATION SCALE
366
365
364
363
362
361
360

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

ONTMT4 420MUSKOKA-I.GPJ 15/10/04

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 420-52

1 OF 1

METRIC

W.P.	LOCATION N 5066581.2 E 310847.4 Muskoka Road Overpass (SBL)	ORIGINATED BY SL
HWY 11	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 28.04.04 - 28.04.04	CHECKED BY AEG

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

SOIL PROFILE			SAMPLES								REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
366.9														
0.0	**SAND and GRAVEL** Compact Brown Moist (FILL)		1	SS	15		366		○					
365.5 1.4	**SAND**, trace gravel, trace silt Dense Brown Moist to Wet		2	SS	33		365		○					
			3	SS	40				○		8	89	3 (SI+CL)	
364.0 2.9	**SAND and GRAVEL**, trace silt, occasional cobbles and boulders Dense Brown Wet		4	SS	50/ .152		364		○		40	54	6 (SI+CL)	
363.2 3.7	END OF BOREHOLE AT 3.7m. AUGER REFUSAL AT 3.7m ON PROBABLE BEDROCK OR BOULDERS. WET CAVE AT 0.86m UPON COMPLETION.													

ONTMT4 420MUSKOKA-I.GPJ 15/10/04

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

$15 \oplus 5$ (20 above, 10 below) (%) STRAIN AT FAILURE

Appendix B

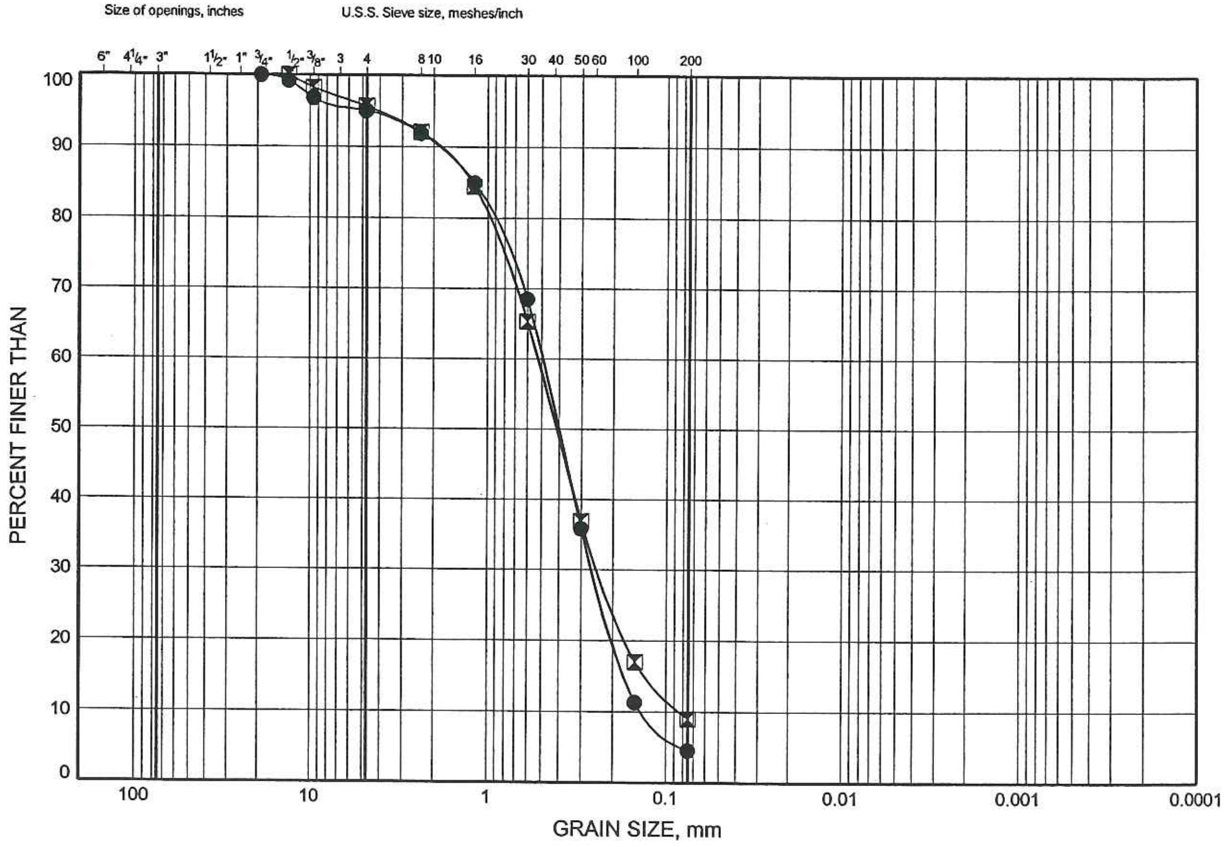
Laboratory Test Results

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND (FILL)



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	420-34	1.07	365.18
☒	420-46	1.07	365.88

Date June 2004

Project 743-93-01

THURBER

Prep'd SS

Chkd. RA

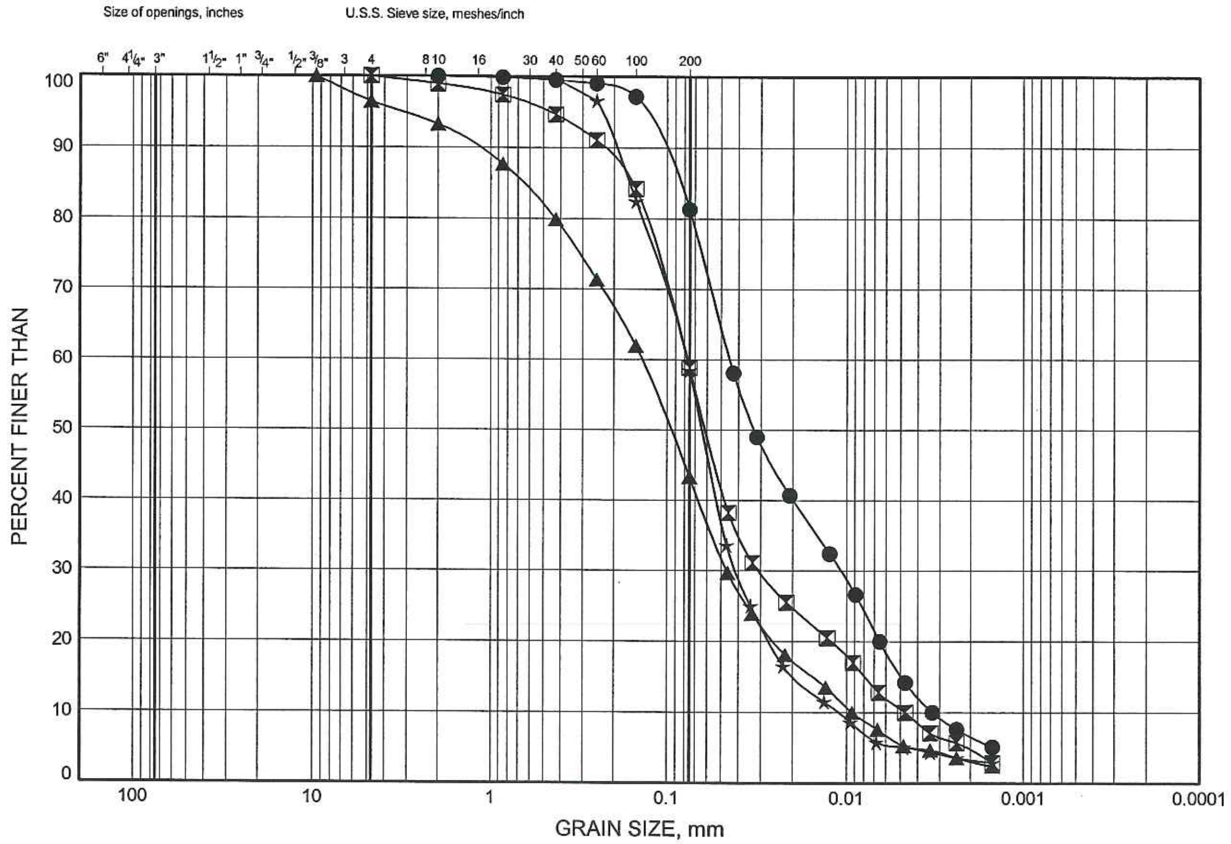
THURBGSD 420MUSKOKA-I.GPJ 21/06/04

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND AND SILT



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

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

THURBGSD 420MUSKOKA-I.GPJ 06/08/04

Date August 2004

Project 743-93-01

THURBER

Prep'd SS

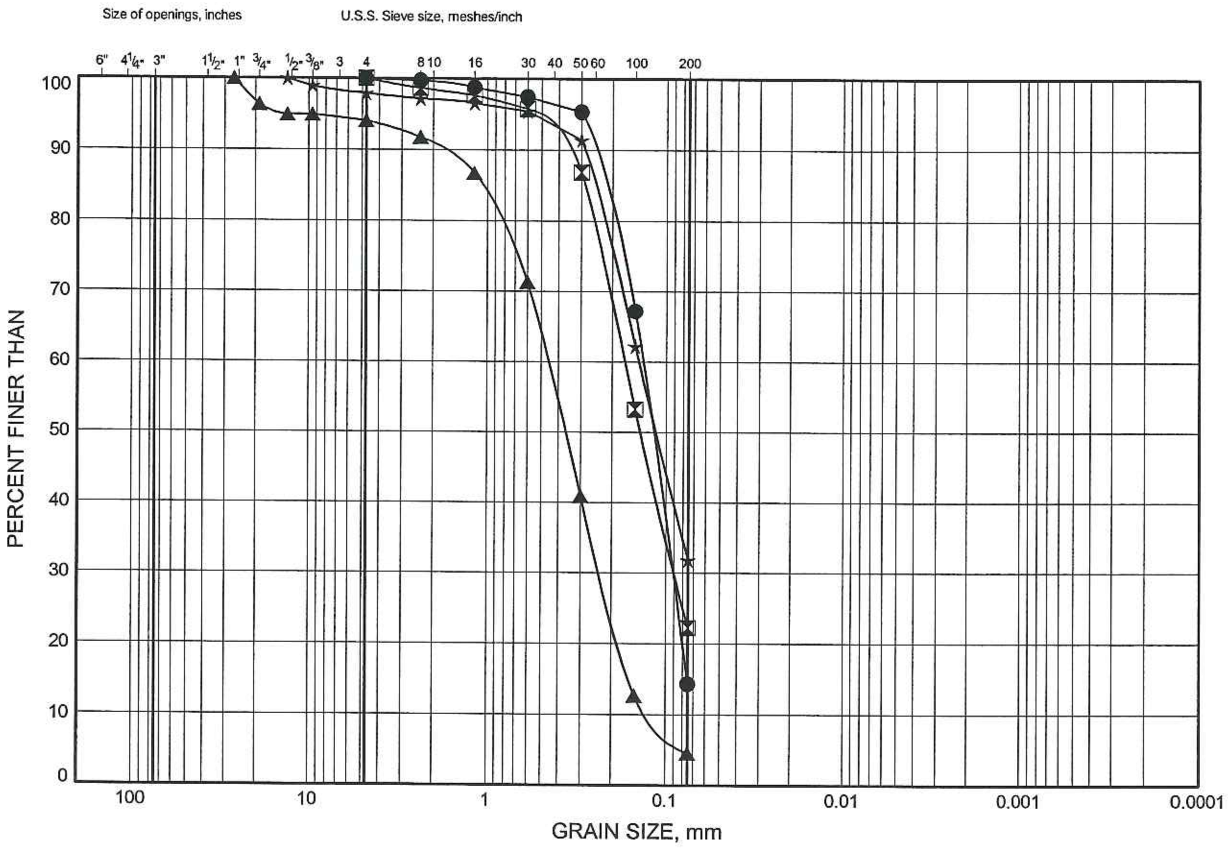
Chkd. RA

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B3A

SAND to SILTY SAND



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

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

THURBGSD 420MUSKOKA-I.GPJ 06/08/04

Date August 2004

Project 743-93-01

THURBER

Prep'd SS

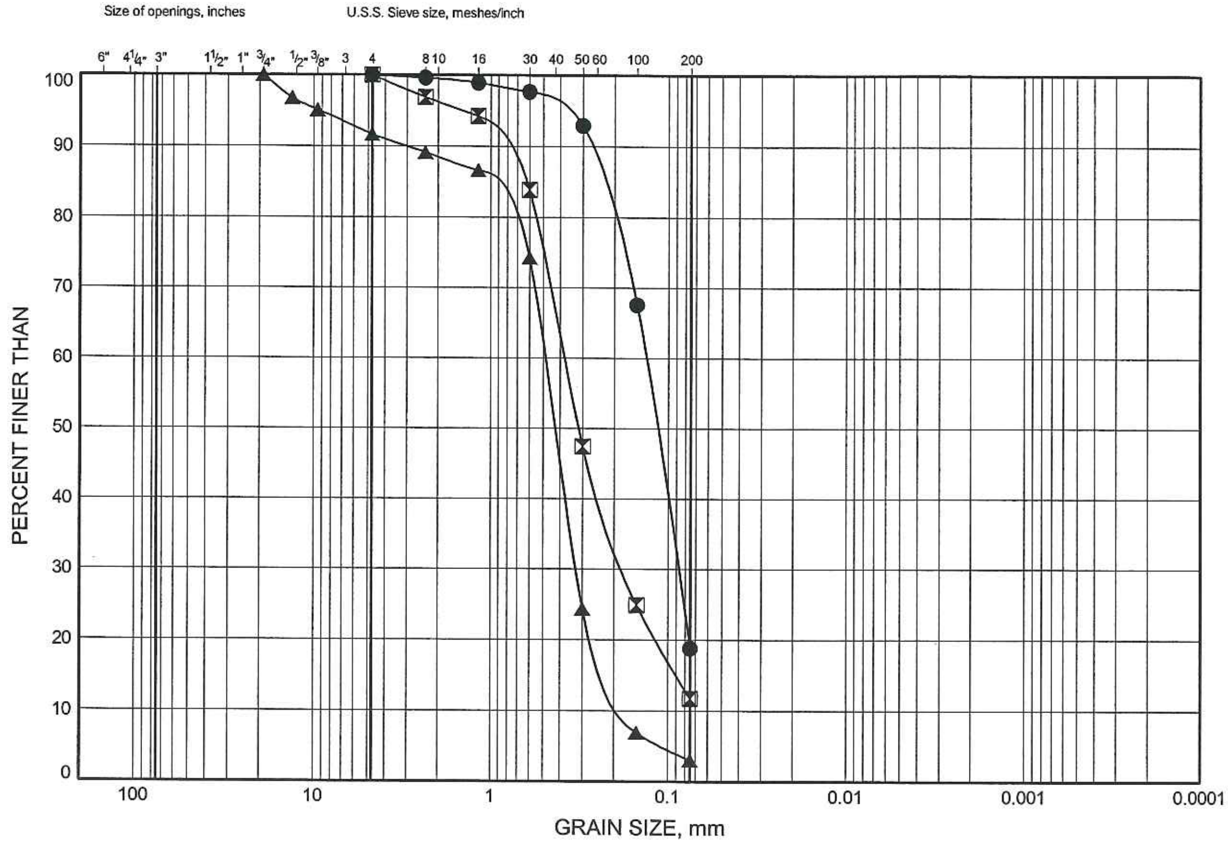
Chkd. RA

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B3B

SAND to SILTY SAND



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

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

Date August 2004

Project 743-93-01

THURBER

Prep'd SS

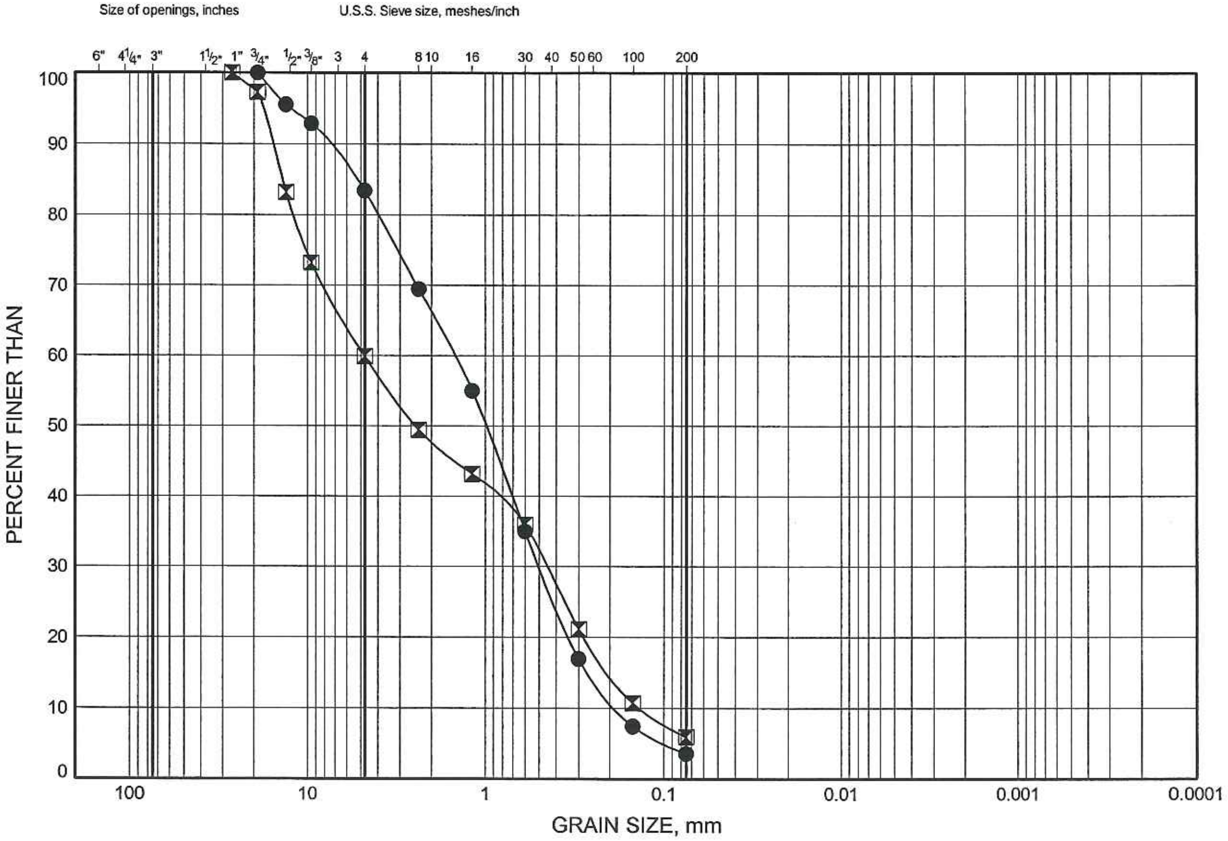
Chkd. RA

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND, SOME GRAVEL



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

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

Date August 2004

Project 743-93-01

Prep'd SS

Chkd. RA

TABLE B1 - Point Load Test Results
Muskoka Road Overpass SBL

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

Total Rock Core

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

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

Total Rock Core

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

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

Total Rock Core

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

TABLE B1 - Point Load Test Results
Muskoka Road Overpass SBL

Depth				UCS
feet	**Inches**	**m**	**Is50**	**(MPa)**
420-39				
25	6	7.77	4.30	103.23
26	1	7.95	2.90	69.52
27	8	8.43	2.02	48.45
29	10	9.09	1.93	46.35
30	4	9.25	2.72	65.31
31	1	9.47	2.19	52.67
32	2	9.80	0.00	0.00
32	11	10.03	3.12	74.79
33	6	10.21	2.59	62.15
34	1	10.39	3.34	80.06

Total Rock Core

Average	Minimum	Maximum	
60	0	103	MPa

Run #	Average
1	73.74
2	54.47

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

Depth				UCS
feet	**Inches**	**m**	**Is50**	**(MPa)**
420-40				
18	7	5.66	4.83	115.87
19	6	5.94	1.19	28.44
20	3	6.17	2.85	68.47
21	4	6.50	3.16	75.84
22	6	6.86	1.45	34.76
23	7	7.19	3.03	72.68
25	0	7.62	3.25	77.95
26	0	7.92	2.85	68.47
27	0	8.23	4.26	102.18
28	10	8.79	2.46	58.99

Total Rock Core

Average	Minimum	Maximum	
70	28	116	MPa

Run #	Average
1	64.68
2	76.05

Depth				UCS
feet	**Inches**	**m**	**Is50**	**(MPa)**
420-42				
20	3	6.17	3.42	82.16
21	2	6.45	3.73	89.54
22	1	6.73	2.28	54.78
23	3	7.09	2.77	66.36
24	4	7.42	2.55	61.10
25	1	7.65	1.98	47.40
25	8	7.82	2.28	54.78
27	6	8.38	3.03	72.68
28	6	8.69	3.07	73.74
29	0	8.84	1.89	45.29

Total Rock Core

Average	Minimum	Maximum	
65	45	90	MPa

Run #	Average
1	70.79
2	58.78

TABLE B1 - Point Load Test Results
Muskoka Road Overpass SBL

Depth				UCS
feet	**Inches**	**m**	**Is50**	**(MPa)**
420-45				
13	6	4.11	3.42	82.16
14	4	4.37	3.73	89.54
15	0	4.57	2.28	54.78
15	10	4.83	2.77	66.36
16	8	5.08	2.55	61.10
17	6	5.33	1.98	47.40
18	6	5.64	2.28	54.78
19	8	5.99	3.03	72.68
20	8	6.30	4.03	96.68
21	8	6.60	5.03	120.68
22	8	6.91	3.07	73.74
23	6	7.16	1.89	45.29

Total Rock Core

Average	Minimum	Maximum	
72	45	121	MPa

Run #	Average
1	75.49
2	60.46
3	84.10

Depth				UCS
feet	**Inches**	**m**	**Is50**	**(MPa)**
420-46				
17	3	5.26	7.86	188.55
18	1	5.51	4.35	104.28
19	3	5.87	6.10	146.42
20	0	6.10	5.71	136.94
21	0	6.40	4.83	115.87
21	10	6.65	0.97	23.17
23	0	7.01	0.00	0.00
23	10	7.26	0.00	0.00

Total Rock Core

Average	Minimum	Maximum	
89	0	189	MPa

Run #	Average
1	138.41
2A	23.17
2B	0.00

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

Depth				UCS
feet	**Inches**	**m**	**Is50**	**(MPa)**
420-51				
13	6	4.11	2.51	60.13
14	5	4.39	3.89	93.31
15	8	4.78	4.32	103.68
17	0	5.18	4.23	101.60
18	6	5.64	3.89	93.31
19	11	6.07	3.89	93.31
21	0	6.40	4.67	111.97
22	0	6.71	4.97	119.23
23	5	7.14	4.58	109.90

Total Rock Core

Average	Minimum	Maximum	
98	60	119	MPa

Run #	Average
1	76.72
2	97.98
3	113.70

Appendix C

Record of Borehole Sheets
(Previous Investigation)

RECORD OF BOREHOLE No 7-1

1 OF 1 **METRIC**

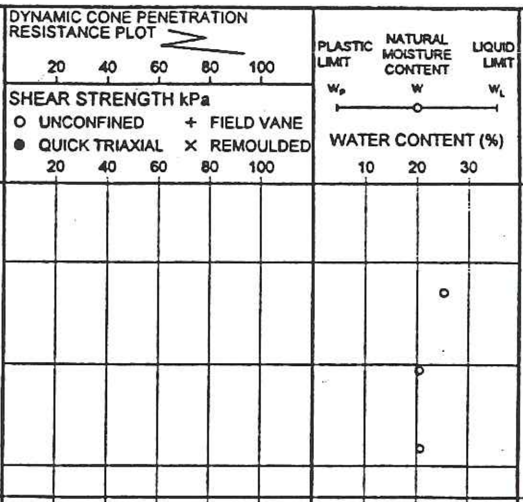
PROJECT 991-1193
W.P. 335-98-00
DIST 54 HWY 11
DATUM GEODETIC
LOCATION N 5066468.88; E 310835
BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS
DATE Feb.29/00
ORIGINATED BY SB
COMPILED BY DKB
CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
364.77	GROUND SURFACE										
0.00	Topsoil	1	SS	2							
364.47 0.30	Sandy Silt, trace clay and organics Very loose Brown Wet organic content = 5.5%										
364.08 0.69	Sand, trace to some silt Compact Brown Wet	2 3	SS SS	22 22	▽	364 363		0	82	18	0
362.56 2.21	Silt and Sand, trace clay Compact Brown Wet	4	SS	15		362					
361.80 3.08	Sand, some gravel, trace silt, occ. cobbles and/or boulders Very dense Brown Wet	5	SS	55/.05							

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

RECORD OF BOREHOLE No 7-2

1 OF 1 METRIC

PROJECT 991-1193
W.P. 335-98-00
DIST 54 HWY 11
DATUM GEODETIC
LOCATION N 5066487.78; E 310829.15
BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS
DATE Feb.29/00
ORIGINATED BY SB
COMPILED BY DKB
CHECKED BY ASP

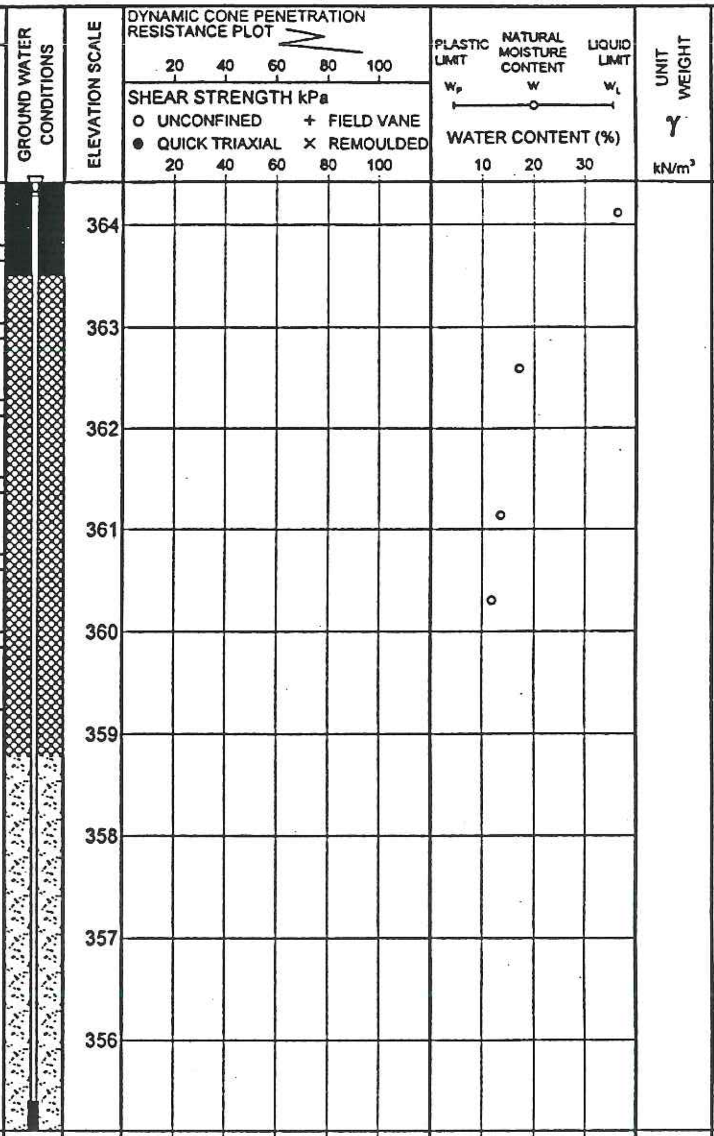
SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
364.42	GROUND SURFACE
0.00	Topsoil
0.15	Sandy Silt, trace clay and organics Very loose Brown Wet
363.73 / 0.69	Sand, trace silt Compact Brown Wet
362.97 / 1.45	Silt and Sand, trace clay Compact to loose Brown Wet
361.45 / 2.97	Sand, some gravel to gravelly sand, trace silt, occ. cobbles and/or boulders Compact to very dense Brown Wet
359.19 / 5.23	Slightly weathered to fresh, grey-white with dark grey blotches, moderately jointed, coarse-grained, strong BIOTITE GNEISS. Bedrock cored from 5.23m to 9.32m depth. For bedrock coring details refer to Record of Drillhole 7-2
355.10 / 9.32	END OF HOLE

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	2				
2	SS	25				
3	SS	19	0	44	49	7
4	SS	9				
5	SS	21	16	74	10	0
6	SS	44				
7	SS	53				

REMARKS & GRAIN SIZE DISTRIBUTION (%)



Note:
1. Water level measured in piezometer at ground surface upon completion of installation.
2. Water level measured in piezometer at 0.2m depth (El. 364.2m) on March 8, 2000.
3. Water level measured in piezometer at 0.1m depth (El. 364.3m) on March 26, 2000.

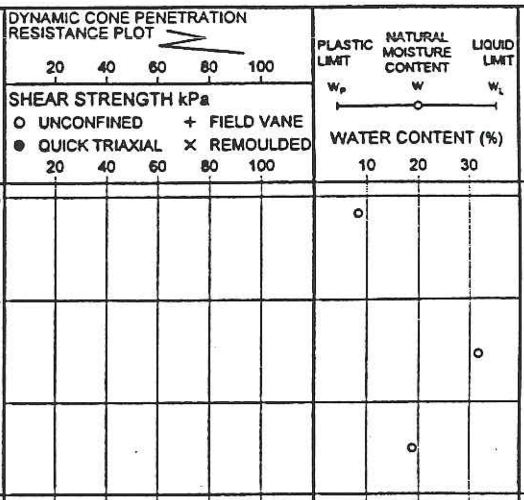
$+^3, \times^3$: Numbers refer to Sensitivity
$O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 7-3

1 OF 1 **METRIC**

PROJECT 991-1193
W.P. 335-98-00
DIST 54 HWY 11
DATUM GEODETIC
LOCATION N 5066559.56; E 310803.80
BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS
DATE Feb.29/00
ORIGINATED BY SB
COMPILED BY DKB
CHECKED BY ASP

SOIL PROFILE		SAMPLES						REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
366.14	GROUND SURFACE										
0.00	Sand and Gravel, trace silt Compact to dense Brown Moist (Fill)	1	SS	22		366					
		2	SS	36		365					
364.69 1.45	Silt and Sand, trace gravel, trace clay and organics Loose Brown Wet non-plastic Atterberg limit test result for Sample 3A organic content=3.1%	3	SS	8		364		2	38	48	8
364.31 1.83	Sand, some silt, trace gravel Dense Brown Moist	4	SS	34				4	81	15	0
363.09 3.05	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.										



+³, ×³: Numbers refer to Sensitivity   ○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 7-4

1 OF 1 **METRIC**

PROJECT 991-1193
W.P. 335-98-00
DIST 54 HWY 11
DATUM GEODETIC

LOCATION N 5066579.07; E 310794
BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS
DATE Feb.29/00

ORIGINATED BY SB
COMPILED BY DKB
CHECKED BY ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
367.00	GROUND SURFACE										
0.00	Sandy Silt, trace clay and organics/decaying wood matter Loose Blackish brown Moist		1	SS	4	∇			○		
366.31 0.69	Sand, some gravel, trace silt, occ. cobbles and/or boulders Very dense Brown Wet		2	SS	59		366				
			3	SS	100/.03				○		
365.26 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.										

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

Appendix D

Foundation Comparison

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

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Piles	Footing on Native Soil	Footing on Engineered Fill	Footing on Bedrock
North & South Abutment	***Advantages:*** 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. Allows for the construction of an integral abutment structure. ***Disadvantages:*** i. Higher unit cost compared to footings. ii. Requires site preparation such as bedrock excavation to allow for construction of an integral abutment structure.	***Advantages:*** i. Lower cost than deep foundations ***Disadvantages:*** i. Comparatively longer abutment stem. ii. Low geotechnical resistance may result in uneconomically large footing sizes. **NOT RECOMMENDED**	***Advantages:*** ii. Lower cost than deep foundations ***Disadvantages:*** i. Low geotechnical resistance may result in uneconomically large footing sizes. ii. Higher cost than footing on native soil.	***Advantages*** i. High geotechnical resistance available ***Disadvantages*** i. Sloping bedrock surface exists at the south abutment. ii. Relatively long abutment stems will be required if footings are founded directly on bedrock. iii. Requires mass concrete fill to raise abutment footings to desired founding elevation. **NOT RECOMMENDED**

DRAFT

THURBER

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Piles	Footing on Native Soil	Footing on Engineered Fill	Footing on Bedrock
North Pier	*Advantages:* i High geotechnical resistance available by seating piles in bedrock. *Disadvantages:* i Higher unit cost compared to footings.	*Advantages:* i. Lower cost than deep foundations *Disadvantages:* i. May need dewatering to maintain a stable excavation.	Not suitable	*Advantages:* i High available geotechnical resistance *Disadvantages:* i Higher unit cost compared to footings on native ground. ii. Requires mass concrete fill to raise footing to desired founding elevation.
South Pier	*Advantages:* i High geotechnical resistance available by seating piles in bedrock. *Disadvantages:* i Higher unit cost compared to footings	*Advantages:* Not suitable.	*Advantages:* i. Spread footings founded at higher depths will reduce the cost of excavation ii. Less expensive than deep foundations. *Disadvantages:* i Lower available geotechnical resistance	*Advantages:* i High available geotechnical resistance *Disadvantages:* i Higher unit cost compared to footings on native ground. ii. Requires mass concrete fill to raise footing to desired founding elevation.

Appendix E

Special Provisions

DRAFT

THURBER

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

The following Special Provisions are referenced in this report:

110F13
105S10
Amendment to OPSS 206, December 1993
902S01
903S01

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

501.08.02 Method A shall be replaced by the following:

5.0.08.02 Method a

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

Appendix F

Drawing

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

HWY 11
CONT No
WP No

MUSKOKA ROAD
OVERPASS SBL
BOREHOLE LOCATIONS AND SOIL STRATA

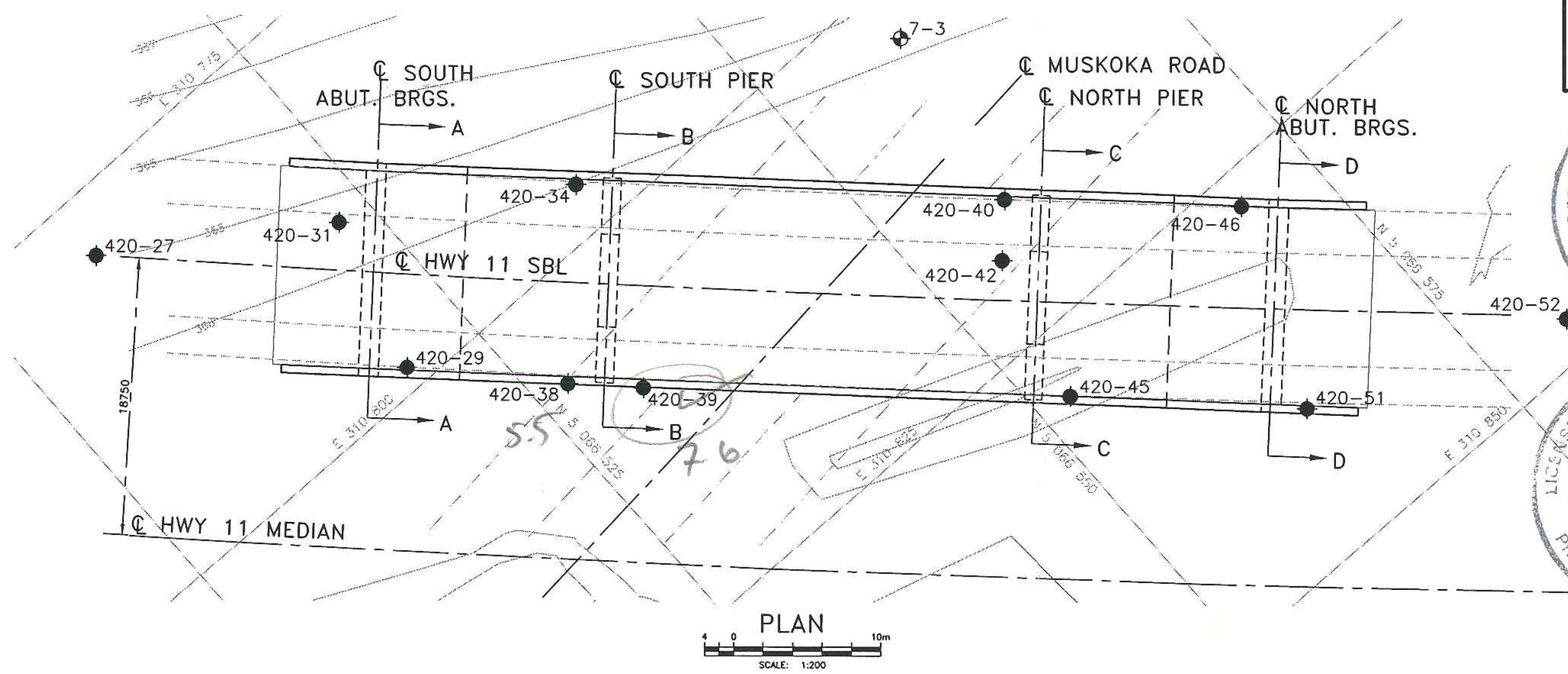
SHEET

Marshall Macklin Monaghan
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

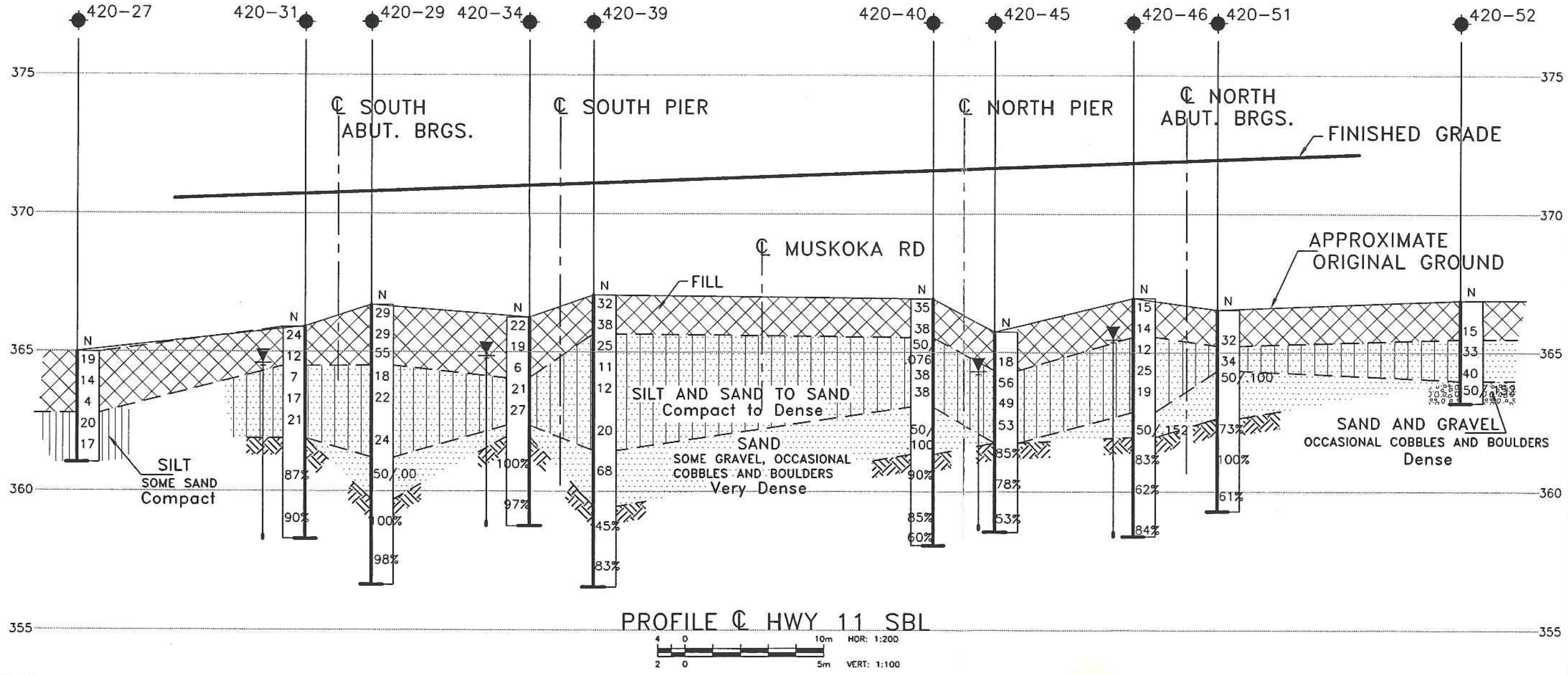
THURBER ENGINEERING LTD.

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
Oct 18/04
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
K. K. CHATTERJI
Oct 18/04
PROVINCE OF ONTARIO



PLAN



PROFILE ℄ HWY 11 SBL

KEYPLAN

LEGEND

●	BoreHole by THURBER
⊕	Dynamic Cone Penetration Test (cone)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▼	WL at Apr 16, 2004 and Jun 18, 2004
⊥	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
420-27	365.0	5 066 508.8	310 779.0
420-29	366.7	5 066 519.8	310 798.5
420-31	365.9	5 066 522.7	310 788.1
420-34	366.3	5 066 536.5	310 796.8
420-38	366.9	5 066 527.3	310 806.5
420-39	367.1	5 066 531.0	310 810.0
420-40	366.9	5 066 557.7	310 816.5
420-42	367.0	5 066 554.9	310 819.5
420-45	365.7	5 066 552.4	310 829.4
420-46	367.0	5 066 569.5	310 827.4
420-51	366.5	5 066 563.9	310 840.5
420-52	366.9	5 066 581.1	310 847.4
7-3	366.1	5 066 559.6	310 803.8

— NOTE —

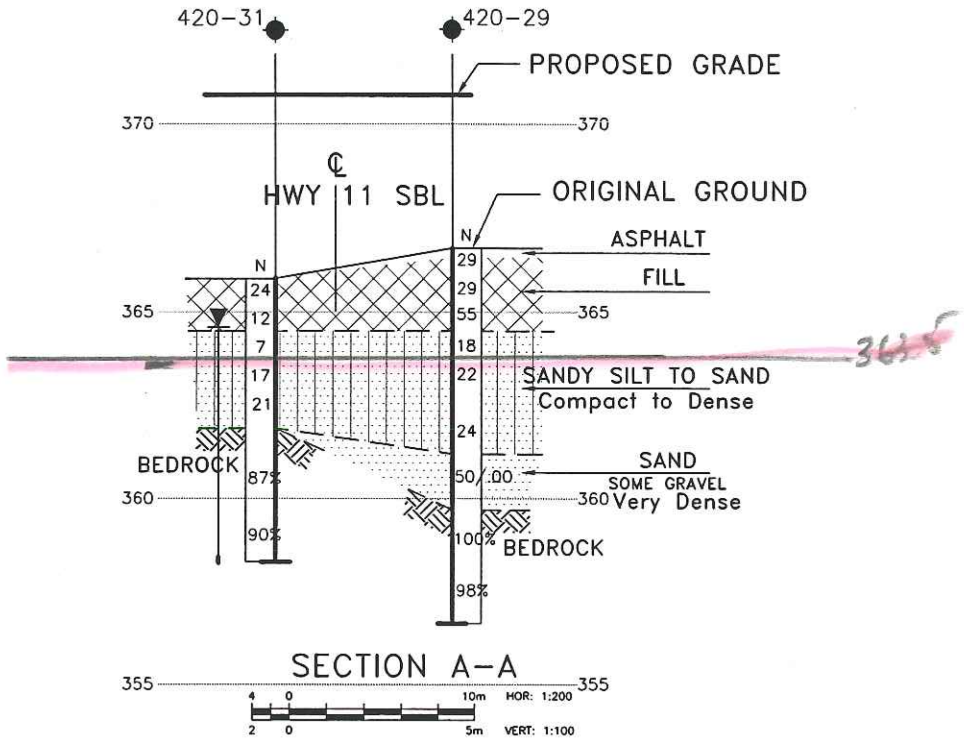
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

BENCHMARK
DHO BM 349-67 TABLET SET HORIZONTALLY IN ROCK 65.8 LT OF 18+085.7
BM ELEV. : 368.485

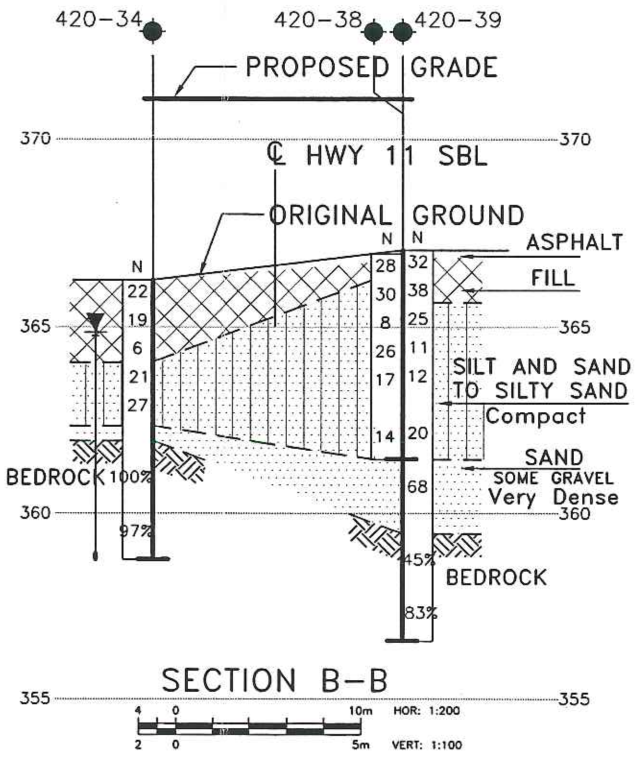
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS DATE	BY	DESCRIPTION

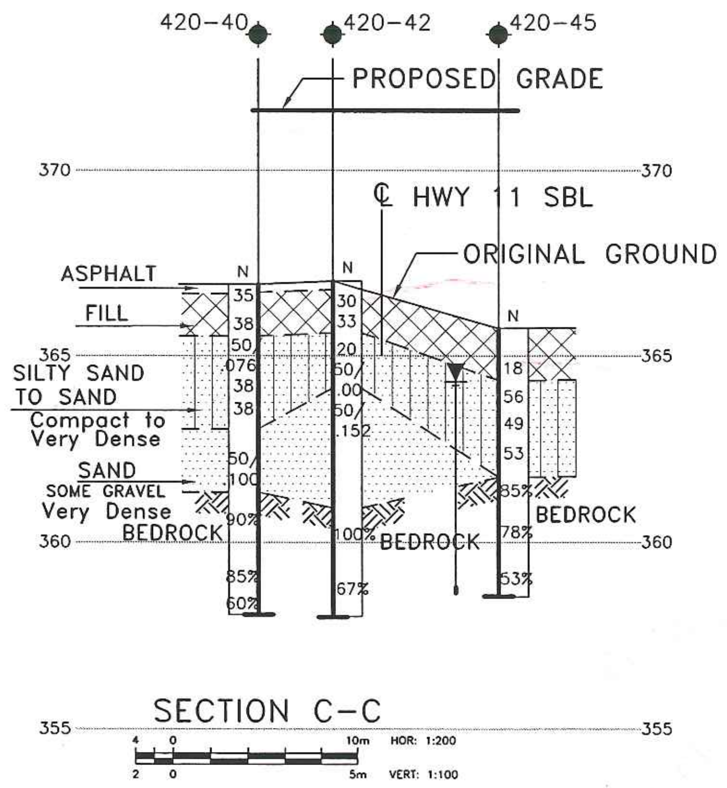
DESIGN RA	CHK AEG	CODE CHBDC 2000	LOAD CL-625-ONT	DATE Aug 2004
DRAWN SS	CHK AEG	SITE 44-420	STRUCT	SCHEME / DWG P1



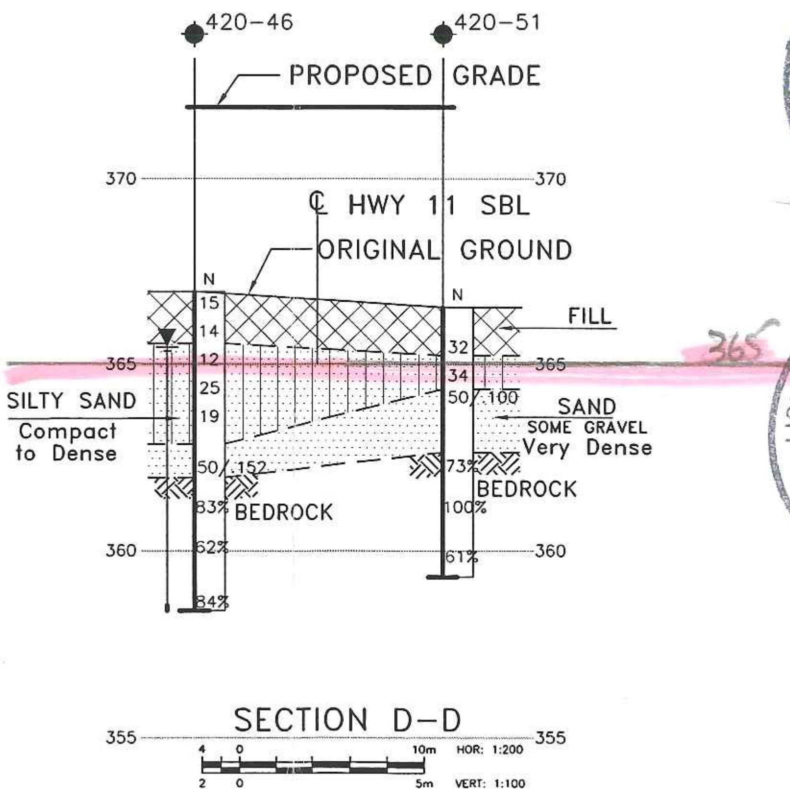
SECTION A-A



SECTION B-B



SECTION C-C



SECTION D-D

Subexcavate for perched abutment

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

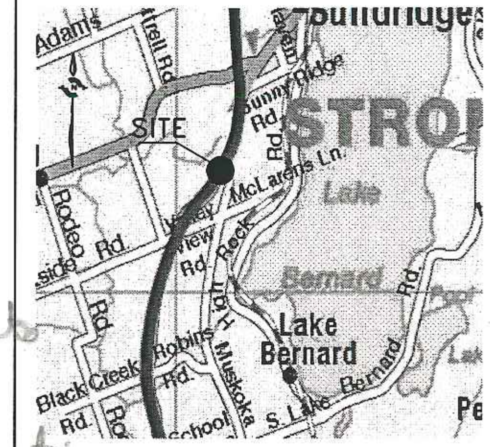
HWY 11
CONT No
WP No

MUSKOKA ROAD
OVERPASS SBL
SOIL STRATA

SHEET

Marshall
Macklin
Monaghan
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.
THURBER



KEYPLAN

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
Oct 18/04
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
04/18/04
PROVINCE OF ONTARIO

LEGEND

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N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	WL at March, 04, 2004
⊤	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
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REVISIONS			
DATE	BY	DESCRIPTION	

DESIGN RA	CHK AEG	CODE CHBDC 2000	LOAD CL-625-ONT	DATE Aug 2004
DRAWN SS	CHK AEG	SITE 44-420	STRUCT	SCHEME	DWG P2