FOUNDATION INVESTIGATION AND DESIGN REPORT
MOOSE LAKE ROAD CONNECTION OVERPASS - SBL STRUCTURE
HIGHWAY 69 FOUR LANING
FROM THE SOUTH JUNCTION OF HIGHWAY 69 AND HIGHWAY 529, NORTHERLY FOR 15 KM

W.P. 5196-06-01, SITE No 44-448/2

G.W.P. 5076-06-00

Geocres Number: 41H-78

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 location of a proposed bridge carrying Highway 69 Southbound lanes (SBL) over Moose Lake Road in the Township of Harrison, Ontario. The entire project involves four-laning of Highway 69 from the south junction of Highway 69 and Highway 529 northerly for 15 km.

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

Thurber carried out the investigation as a sub-consultant to MMM Group Limited (MMM), under the Ministry of Transportation Ontario (MTO) Agreement Number 5006-E-0030.

2 SITE DESCRIPTION

The site lies about 100 m west of existing Highway 69, approximately 1.0 km north of the south junction of Highway 69 and Highway 529 in the Township of Harrison, Ontario.

Lands surrounding the proposed Moose Lake Road SBL structure are generally undeveloped forested land with open swamps. Bedrock outcrops and ridges are visible along the existing Highway 69 corridor and at the site. Moose Lake is located approximately 100 m east of existing Highway 69. Lands east of existing Highway 69 are occupied by a few commercial and residential dwellings.

Small creeks/water bodies are observed along the proposed Highway 69 alignment.

Photographs in Appendix E show the general nature of the surrounding lands.

The site lies within the physiographic region known as the Georgian Bay Fringe, which covers Parry Sound and Muskoka. The region is characterized by very shallow overburden and bare rock knobs and ridges. Bedrock is exposed in many areas and intermittent swamps were filled in when glacial lake Algonquin inundated the area. The overburden materials consist of sand, silt and clay. Recent organic deposits of peat and muck occur in abundance in the bedrock hollows and valleys.

The area is underlain by strongly foliated and highly to moderately deformed rocks of Precambrian age of the following types:

- Gneisses of metasedimentary origin.
- Migmatitic rocks and gneisses.
- Felsic igneous rocks (tonalite, granodiorite, monzonite, granite, syenite, derived gneisses).
- Tectonite unit (tectonites, various gneisses).

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out on August 19 and 20, 2009 and consisted of drilling or manual excavation and sampling a total of 14 boreholes (numbered MLR-S-09-01 to MLR-S-09-14) at the foundation elements and approaches of the proposed Highway 69 SBL structure. Borehole advancement within the overburden generally ranged from 0.1 m to 1.2 m where the drill rig encountered refusal. Bedrock outcrops (bedrock exposed at surface) and very shallow bedrock were noted at some borehole locations where visual assessment and manual excavation were conducted to assess the thickness of the overburden. Six boreholes (three at each bridge abutment) were also advanced 3.0 m to 3.2 m into bedrock by NQ size diamond coring.

The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

Drilling was carried out using a track mounted CME 55 drill rig. A combination of hollow-stem auger drilling techniques and rotary coring methods were used to advance the boreholes. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

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

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Two standpipe piezometers (one at each abutment) consisting of 19 mm PVC pipe with slotted screens were 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 – Borehole Completion Details

Foundation Unit	Borehole	Piezometer Tip Depth/ Elevation (m)	Completion Details
SBL STRUCTURE			
South Approach	MLR-S-09-13	None installed	Manual excavation. Very shallow bedrock.
South Abutment	MLR-S-09-01	3.5/197.6	Sand from 3.5 m to 1.4 m, then holeplug to surface.
	MLR-S-09-02	None installed	Manual excavation. Very shallow bedrock.
	MLR-S-09-03	None installed	Manual excavation. Very shallow bedrock.
	MLR-S-09-04	None installed	Borehole backfilled with holeplug to surface.
	MLR-S-09-05	None installed	Borehole backfilled with holeplug to surface.
	MLR-S-09-06	None installed	Manual excavation. Very shallow bedrock.
North Abutment	MLR-S-09-07	None installed	Manual excavation. Very shallow bedrock.
	MLR-S-09-08	None installed	Borehole backfilled with holeplug to surface.
	MLR-S-09-09	3.2/199.8	Sand from 3.1 m to 1.2 m, then holeplug to surface.
	MLR-S-09-10	None installed	Manual excavation. Very shallow bedrock.
	MLR-S-09-11	None installed	Manual excavation.
	MLR-S-09-12	None installed	Borehole backfilled with holeplug to surface.
North Approach	MLR-S-09-14	None installed	Manual excavation. Very shallow bedrock.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. A selected sample was also subjected to gradation analysis. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figure contained in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the rock core samples are shown in Table 1 immediately following the text and on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these sheets and on the "Borehole Locations and Soil Strata" and "Stratigraphic Sections" 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 terms, the relatively thin overburden encountered at this site consists of fibrous peat overlying native sand. Granitic gneiss bedrock was contacted below the peat and the native sand at a shallow depth. More detailed descriptions of the individual strata are presented below.

5.1 Peat

Dark brown fibrous peat was identified at ground surface in all the boreholes. The peat thickness generally ranged from 25 mm to 500 mm at the proposed SBL structure. At the locations of boreholes MLR-S-09-02, MLR-S-09-03, MLR-S-09-06, MLR-S-09-07, MLR-S-09-09 to MLR-S-09-11, MLR-S-09-13 and MLR-S-09-14 visual assessment and manual excavation revealed the presence of bedrock below the peat.

The peat thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.2 Sand

Native fine brown to grey sand containing trace silt, trace gravel to gravelly and occasional organics and cobbles were contacted below the peat in Boreholes MLR-S-09-01, MLR-S-09-04, MLR-S-09-05, and MLR-S-09-12. The thickness of the sand layer varies from 0.1 m to 1.1 m.

A 100-mm thick layer of cobbles was contacted below the sand at 1.1 m depth (Elevation 201.1) in Borehole MLR-S-09-05.

Boreholes where sand was encountered, were terminated below the sand layer upon refusal on probable bedrock at depths ranging from 0.2 m to 1.2 m (Elevations 200.8 to 201.9).

Standard Penetration tests in the sand layer gave SPT N-values ranging from 1 to 6 blows per 0.3 m of penetration, indicating a very loose to loose relative density. An SPT N-value of 137 blows per 0.3 m of penetration, indicating a very dense relative density, was measured in Borehole MLR-S-09-12. However, this isolated result is not considered to be representative.

The moisture content of samples from the sand layer varies between 10% and 22%.

A grain size distribution curve for a gravelly sand sample tested is presented on the Record of Borehole sheet and on Figure B1 in Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	Gravelly Sand (%)
Gravel	23
Sand	59
Silt	16
Clay	2

5.3 Bedrock

The overburden described above is underlain by granitic gneiss bedrock. The bedrock is moderately and slightly weathered to fresh bedrock. The bedrock was generally grey with occasional dark grey, brownish and white bands visible in most cores. Occasional mechanical breaks and sub-vertical fractures were observed in the rock cores.

Bedrock was encountered at shallow depths and it was proved by coring at each abutment. Table 5.1 summarizes depths and elevations to the top of bedrock.

Table 5.1 – Depths and Elevations of Top of Bedrock

Foundation Unit	Borehole	Top of Bedrock	
		Depth (m)	Elevation (m)
South Approach	MLR-S-09-13	0.1	201.5
South Abutment	MLR-S-09-01	0.4*	200.8
	MLR-S-09-02	0.1	201.2
	MLR-S-09-03	0.2	201.2
	MLR-S-09-04	0.2*	201.3
	MLR-S-09-05	1.2*	201.0
	MLR-S-09-06	0.2	201.9
North Abutment	MLR-S-09-07	0.5	202.2
	MLR-S-09-08	0.1*	203.4
	MLR-S-09-09	0.1*	202.9
	MLR-S-09-10	0.1	203.7
	MLR-S-09-11	0.1	202.8
	MLR-S-09-12	1.2*	201.9
North Approach	MLR-S-09-14	0.3	203.3

* Bedrock proved by coring

Core recovery in the bedrock generally ranged from 83% to 100%. The RQD values generally ranged from 72% to 100% indicating fair to excellent rock quality. A lower RQD value of 30%, indicating a poor rock quality, was noted in Borehole MLR-S-09-08 Run 1.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 0 to 5, and greater than 10 in cores from Boreholes MLR-S-09-05 and MLR-S-09-08.

The estimated unconfined compressive strength of the rock cores generally ranges from 71 MPa to 152 MPa, indicating a strong to very strong rock. A lower average unconfined compressive strength values of 26 MPa and 42 MPa were estimated in Boreholes MLR-S-09-01 Run 1 and MLR-S-09-08 Run 1, respectively. Unconfined compressive strengths of 167 MPa and 225 MPa were estimated in Boreholes MLR-S-09-04 Run 1 and MLR-S-09-12 Run 1. These estimated rock strength values are interpreted from 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 1 immediately following the text of this report.

5.4 Water Levels

Water levels were observed in the boreholes during and upon completion of drilling. Two standpipe piezometers were installed to monitor water levels after completion of drilling. The water levels measured in the piezometers are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Foundation Unit	Borehole	Date (2009)	Water Level (m)		Comment
			Depth	Elevation	
South Abutment	MLR-S-09-01	August 25 September 14 September 24 October 7	0.08 0.9 1.2 1.0	201.1 200.3 200.0 200.2	In piezometer
North Abutment	MLR-S-09-09	August 25 September 14 September 24 October 7	0.04 1.3 1.6 1.5	202.9 201.7 201.4 201.5	In piezometer

The piezometric readings indicate that the groundwater levels range from Elevations 200.0 m to 202.9 m.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Borehole locations were selected by Thurber Engineering Ltd. Surveyors from MMM Group Limited staked these locations in the field, confirmed the co-ordinates and obtained the ground surface elevations.

Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied a track mounted CME 55 drill rig and conducted the drilling, sampling and in-situ testing operations.

The field program was supervised on a full time basis by Ms. Eckie Siu of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall supervision of the field program was conducted by Mr. Alastair E. Gorman, P.Eng Interpretation of the data and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. 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 embankment for the proposed Highway 69 SBL structure over Moose Lake Road in the Township of Harrison, Ontario.

Based on the preliminary General Arrangement (GA) drawing provided by MMM Group Limited, a single-span structure supported on two abutments is proposed. The structure will be 23 m long and 14.1 m wide.

Approximately 4.0 m to 6.0 m of rock excavation will be required to establish the proposed profile of Moose Lake Road at elevation 198.0 m.

A general description of the proposed approach embankment for the SBL structure is presented in Table 7.1.

Table 7.1 – General Description of Proposed Structure

Foundation Element	Original ground surface elevation (m)	Elevation at abutment base (m)	Proposed finished grade of Hwy 69 Bridge (m)	Approach embankment height (m)	Excavation to underside of abutment (m)	Excavation to Moose Lake Road base: 198 m (m)
South Abutment	201.2 to 202.2	201.8	204.9	3.1	0.4	3.2 to 4.2
North Abutment	202.7 to 203.8	202.4	205.3	2.9	0.3 to 1.4	4.7 to 5.8

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the investigation.

8 STRUCTURE FOUNDATIONS

The proposed structure is a single-span bridge with two abutments.

The stratigraphy encountered at abutment locations consist of fibrous peat overlying native sand. Thickness of the overburden soils varies from 0.1 m to 1.2 m. Granitic gneiss bedrock was contacted below the peat and the native sand. The groundwater level lies at elevations ranging from 200.0 m to 202.9 m.

Initial consideration was given to the following foundation types:

- Spread footings on native soils
- Spread footings on bedrock
- Augered Caissons (drilled shafts)
- Driven piles

From a geotechnical perspective and based on the subsurface conditions, spread footings founded on bedrock is considered the most cost effective foundation option for supporting the structure at this site.

Spread footings on native soils are not a feasible foundation option at this site due to the presence of bedrock at very shallow depth, generally 0.1 m to 1.2 m below ground surface and very loose conditions of the native sand.

The proposed Moose Lake Road finished grade is at elevation 198.0, which extends within bedrock; placement of engineered fill to support foundation is not considered to be a suitable option at this site.

Use of deep foundations such as piles and caissons is not considered cost effective at this site due to the presence of shallow bedrock.

Consequently, deep foundations, spread footings on native soils and spread footings on engineered fill are not recommended and detailed design recommendations for these foundations types were not developed.

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

8.1 Spread Footings on Bedrock

Based on the subsurface stratigraphy encountered at this site and the proposed Highway 69 and Moose Lake Road grades, the structure could be supported on spread footings bearing on bedrock.

Spread footings can be founded on the undisturbed bedrock at or below elevations given in Table 8.1.

Table 8.1 – Highest Permitted Founding Elevations

Foundation Unit	Borehole	Top of Bedrock		Highest Rock Elevation for the Boreholes Drilled (m)
		Depth (m)	Elevation (m)	
South Abutment	MLR-S-09-01	0.4	200.8	201.9
	MLR-S-09-02	0.1	201.2	
	MLR-S-09-03	0.2	201.2	
	MLR-S-09-04	0.2	201.3	
	MLR-S-09-05	1.2	201.0	
	MLR-S-09-06	0.2	201.9	
North Abutment	MLR-S-09-07	0.5	202.2	203.7
	MLR-S-09-08	0.1	203.4	
	MLR-S-09-09	0.1	202.9	
	MLR-S-09-10	0.1	203.7	
	MLR-S-09-11	0.1	202.8	
	MLR-S-09-12	1.2	201.9	

Spread footings bearing on undisturbed bedrock at or below elevations presented in Table 8.1 may be designed for the following geotechnical resistance:

- Factored geotechnical resistance of 2,000 kPa at Ultimate Limit States (ULS)

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

The ULS resistance was selected taking account of the proximity of the cut rock face.

This resistance value is for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The highest rock elevation encountered in the boreholes at each abutment are presented in Table 8.1. Excavation and backfilling for the footings must be in accordance with OPSS 902 as modified by SP 902S01 (June 2006).

The mass concrete fill must extend beyond the footing perimeter by a sufficient distance to distribute the shear stresses from the footing and prevent stress concentrations under the edge of the footing. This condition must be checked structurally but extension of the mass concrete to 200 mm beyond the edge of the footing should be considered. Similarly, the maximum depth of mass concrete that may be permitted below the footing is a function of the structural behaviour of the concrete and is not an issue of geotechnical resistance.

The bearing surface should be prepared by removing all loose/disturbed material and shattered/loosened rock fragments. Areas requiring subexcavation beneath the underside of footing should be backfilled with the same class of concrete as used in the footing. The same value of resistance as the bedrock may be used where mass concrete of suitable strength is poured in neat contact with clean, sound bedrock surface.

The stability of the rock cut in front of the abutment should be maintained as described in Section 10 of the report.

Footings should be set back a minimum distance of 3.0 m from the crest of the rock cut.

8.2 Lateral Resistance on Bedrock

Initial calculations of the horizontal resistance may be carried out using a value of 0.7 for the ultimate friction factor of concrete poured on rock.

If the frictional component is insufficient, the horizontal resistance may be increased by dowelling the footing into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide only shear resistance. If vertical resistance in tension is required, rock anchors must be included in the design.

The dowel may be considered as acting as a fully embedded pile in the rock and hence will fail when the ultimate lateral resistance of the rock or grout is exceeded. Using lower bound values for the strength of the rock, an ultimate horizontal resistance of 2.6 MN may be assumed for a 50 mm steel dowel embedded 500 mm into the rock. The depth of embedment is measured below the bedrock surface.

The shearing resistance of the selected dowel must be checked structurally.

8.3 Recommended Foundation

From a geotechnical point of view, it is recommended that all foundations for the SBL structure (abutments) be supported on spread footings founded on bedrock.

8.4 Frost Cover

The design depth of frost penetration at this site is 1.9 m.

However, frost penetration is not an issue for footings bearing on bedrock or mass concrete fill placed on bedrock.

9 EXCAVATION

Minor excavation of overburden soils (native sand) and peat removal will be required at this site. Peat is generally 25 mm to 500 mm thick and the layers of sand range from 0.1 m to 1.1 m in thickness. Rock excavation will be required at the SBL structure.

9.1 Earth Excavation

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils within the probable depth of excavation at this site may be classed as Type 3 soils above water level and Type 4 below water level.

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

9.2 Rock Excavation

Rock excavation will be required at this site. For these purposes, the Contractor may elect to use blasting methods.

The Special Provision governing the use of explosives must be included in the contract. The text of the SP is included in Appendix D.

The design of the blast and removal procedures should be the responsibility of the Contractor. However, it is important that his procedures incorporate methods of reducing damage to the founding surfaces or to any adjacent structures. Such methods may include, though not necessarily be limited to, line drilling, pre-splitting and cushion blasting.

10 PERMANENT ROCK CUT

Permanent rock cuts are required to establish the proposed finished grade elevation of Moose Lake Road (approximate elevation 198.0) at this site. The cut will be formed predominantly through about 4.0 m to 6.0 m of the existing bedrock and the structure foundations will be founded on the rock at the top of the cut.

The rock cut in front of the abutments and to 10.0 m beyond the structure limits should conform to OPSD 201.020 with the 1H : 4V option.

Depending on the location, orientation and height of the rock cut with respect to the pattern of joints or fractures in the rock mass, potentially unstable rock wedges may exist below the abutment foundation.

The contract should include an NSSP to this effect.

10.1 Stabilization of Rock Excavation

Depending on the orientation, spacing and characteristics of the pattern of joints and fractures within the rock mass, with respect to the depth and orientation of the cut face, it is possible that potentially unstable blocks or wedges of rock may develop that will impact the stability of the structure foundations.

In order to detect such conditions and ensure that the rock face is sound and capable of supporting the foundations, the Contract must include an NSSP that directs the Contractor to examine the rock face and to take such steps as are necessary to stabilize the face.

Suggested text for the NSSP is included in Appendix D.

11 UNWATERING

Piezometers installed in boreholes revealed that groundwater level is near Elevations 200.0 m to 202.9 m. The groundwater levels are 2.0 m to 4.9 m above base of the cut (Moose Lake Road finished grade). Also seepage from the sides of the cut and surface runoff is expected to occur.

The design of foundations bearing on bedrock will not be influenced by the groundwater, but the Contractor must make provision to control the groundwater seepage and use sump pumps or perimeter ditches to remove any accumulated water from the footing base prior to placing concrete and permit construction in the dry.

12 APPROACH EMBANKMENTS

Approach embankment construction using either earth fill or rock fill is feasible on the foundation conditions encountered at this site.

Height of the proposed approach fills range from 2.9 m to 3.1 m and the fills will be constructed on bedrock. Negligible foundation settlements are expected if the peat and any loose wet soils are removed from under the approach embankment. Post construction settlement of the embankment fill mass is estimated to be in the order of 0.5% of the embankment height, approximately 15 mm to 20 mm.

Rock fill embankments should be overbuilt in accordance with current Northeastern Region policies and guidelines.

The global, internal and surficial stability of the approach embankment fills 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, select subgrade material or non-cohesive earth fill will have stable side slopes at inclinations of up to 2H:1V.

It is recommended that all peat and loose sand be stripped prior to constructing the approach fills. Embankment construction should be in accordance with SP206S03, dated July 2007.

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

13 BACKFILL TO ABUTMENTS

Backfill to the abutment should 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 specify grading limits for 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 must be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill must be placed to the extents shown in OPSD 3101.150, and rock backfill must be placed to the extents shown in OPSD 3101.200. All granular material should meet the requirements of SP 110F13 Amendment to OPSS 1010, March 1993.

Compaction equipment to be used adjacent to the abutment walls must be restricted in accordance with SSP 105S10.

The design of the abutment must incorporate a subdrain as shown in OPSD 3101.150 or OPSD 3101.200, as applicable.

14 EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where:

P_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see table 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 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 14.1.

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.

The factors in Table 14.1 are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

Table 14.1 – Earth Pressure Coefficient (K)

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 (Limited to 250 mm size) $\phi = 42°, \gamma = 19$ 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*	0.2	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

15 SEISMIC CONSIDERATIONS

15.1 Seismic Design Parameters

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

- Velocity Related Seismic Zone 1
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.08

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

15.2 Liquefaction Potential

The foundation soils at the abutments are not in danger of liquefaction under earthquake loading.

15.3 Retaining Wall Dynamic Earth Pressures

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

For the design of retaining walls, the coefficients of horizontal earth pressure in Table 15.1 may be used:

Table 15.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
	Granular A or Granular B Type II $\phi = 35°$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42°$; $\gamma = 19.0$ 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	0.47	0.34	0.58	0.22	0.31
Passive (K_{PE})	3.6	-	3.2	-	4.9	-
At Rest (K_{OE})**	0.53	-	0.57	-	0.43	-

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

16 CONSTRUCTION CONCERNS

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

- The potential for the development of unstable rock wedges under the abutment footings.
- Disturbance of the bedrock under the foundations due to blasting or other excavation procedures.
- Variations in the elevation of the bedrock surface, necessitating the use of mass concrete fill to prepare the design founding elevation.

17 CLOSURE

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

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

Thurber Engineering Ltd.

Rocío Palomeque Reyna, P.Eng., M.Eng.
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
OCT 4/10
PROVINCE OF ONTARIO

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Senior Foundations Engineer

LICENSED PROFESSIONAL ENGINEER
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Oct 4/10
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Report reviewed by:
P.K. Chatterji, P.Eng., Ph.D.
Review Principal

TABLE 1 -Point Load Test Results

MOOSE LAKE ROAD OVERPASS - SBL STRUCTURE

HWY 69 FOUR-LANING

FROM THE SOUTH JUNCTION OF HIGHWAY 69 AND HIGHWAY 529 TO NAISCOOT LAKE

19-5161-21

MLR-S-09-01	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH			CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	BREAK	UCS (Mpa)				
RUN #1	2	9	0.84	6.0	D	47.21	48.31	ok	62.96				
	2	9	0.84	13.0	A	47.15	48.31	ok	111.23				
	3	4	1.02	9.8	A	47.01	48.91	ok	83.25				
RUN #2	4	3	1.30	4.0	D	47.11	50.31	ok	42.11				
	5	4	1.63	6.0	D	47.31	50.13	ok	62.76				
	5	4	1.63	2.0	A	47.31	50.13	ok	16.59				
	6	8	2.03	11.5	D	47.08	51.84	ok	121.19				
	8	4	2.54	9.0	D	47.10	50.41	ok	94.78				
	8	5	2.57	18.9	A	47.10	50.80	ok	155.67				
RUN #3	9	2	2.79	19.0	D	47.03	46.34	ok	200.56				
	10	2	3.10	16.8	D	47.18	49.01	ok	176.47				
	10	2	3.10	24.0	A	47.18	50.31	ok	198.90				
	10	9	3.28	6.5	D	47.01	50.21	ok	68.66				
	11	5	3.48	0.0	D	47.13	50.48	ok	0.00		AVERAGE	MAX	MIN
										RUN #1:	86	111	63
										RUN #2:	82	156	17
										RUN #3:	129	201	0

MLR-S-09-04	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH			CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	BREAK	UCS (Mpa)				
RUN #1	0	11	0.28	10.8	D	47.02	44.24	ok	114.04				
	0	11	0.28	13.1	A	47.02	44.24	ok	120.26				
	2	6	0.76	14.2	D	47.10	47.00	ok	149.55				
	2	6	0.76	9.0	A	9.00	47.10	ok	283.45				
RUN #2	3	6	1.07	6.3	D	47.13	47.34	ok	66.28				
	5	0	1.52	18.0	A	47.25	49.86	ok	150.05				
	6	3	1.91	22.2	D	47.36	53.64	ok	231.81				
	5	0	1.52	5.8	D	47.25	52.36	ok	60.78				
	6	3	1.91	23.0	A	47.25	42.25	ok	217.98				
	7	10	2.39	12.5	D	47.31	49.34	ok	130.74				
RUN #3	9	2	2.79	5.0	D	47.13	48.34	ok	52.61				
	10	8	3.25	15.2	D	47.21	48.44	ok	159.50		AVERAGE	MAX	MIN
										RUN #1:	167	283	114
										RUN #2:	143	232	61
										RUN #3:	106	160	53

TABLE 1 -Point Load Test Results

MOOSE LAKE ROAD OVERPASS - SBL STRUCTURE

HWY 69 FOUR-LANING

FROM THE SOUTH JUNCTION OF HIGHWAY 69 AND HIGHWAY 529 TO NAISCOOT LAKE

19-5161-21

MLR-S-09-05	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH			CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	BREAK	UCS (Mpa)				
RUN #2	4	5.5	1.36	16.5	D	47.19	98.50	ok	173.26				
	4	4.5	1.33	17.5	A	47.18	44.88	ok	158.45				
	5	5.5	1.66	16.0	D	47.20	102.88	ok	167.95				
	5	4.5	1.64	21.5	A	47.25	51.76	ok	174.10				
	7	6	2.29	11.0	D	47.19	129.39	ok	115.50				
	7	4	2.24	9.5	A	47.14	39.11	ok	95.76				
	8	2	2.49	7.0	D	47.28	80.46	ok	73.29				
RUN #3	9	1	2.77	0.5	D	47.26	69.12	ok	5.24				
	9	8	2.95	9.0	A	47.09	46.68	ok	79.16				
	10	7.5	3.24	6.0	D	47.06	64.47	ok	63.27				
	10	5	3.18	9.0	A	47.04	48.07	ok	77.45		AVERAGE	MAX	MIN
	11	8	3.56	11.3	D	48.02	93.47	ok	114.98	RUN #2:	137	174	73
	11	7	3.53	21.0	A	47.15	52.82	ok	167.68	RUN #3:	85	168	5

MLR-S-09-08	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH			CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	BREAK	UCS (Mpa)				
RUN #1	0	7	0.18	0.0	D	97.20	90.00	ok	0.00				
	1	9	0.53	5.5	D	47.20	163.50	ok	57.73				
	0	7	0.18	4.5	D	47.20	52.50	ok	47.24				
	1	4	0.41	11.0	A	47.20	80.00	ok	63.62				
RUN #2	3	11	1.19	12.0	D	47.21	109.50	ok	125.92				
	3	11	1.19	17.5	A	47.21	52.50	ok	140.25				
	4	11	1.50	11.0	D	47.21	300.00	ok	115.43				
	5	2	1.57	20.0	A	47.21	72.50	ok	124.81				
	6	1	1.85	17.0	D	47.21	201.50	ok	178.39				
	5	10	1.78	18.0	A	47.21	70.50	ok	114.79				
	7	10	2.39	14.0	A	47.21	65.50	ok	94.52				
	8	1	2.46	13.5	D	47.21	109.00	ok	141.66				
											AVERAGE	MAX	MIN
RUN #3	9	3	2.82	13.0	D	47.20	180.50	ok	136.46	RUN #1:	42	64	0
	9	5	2.87	16.0	A	47.20	49.50	ok	134.23	RUN #2:	129	178	95
	10	7	3.23	14.5	D	47.20	146.50	ok	152.21	RUN #3:	141	152	134

MLR-S-09-09	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH			CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	BREAK	UCS (Mpa)				
RUN #1	3	5	1.04	0.0	D	47.21	114.73	ok	0.00				
	0	7	0.18	5.5	D	47.28	89.38	ok	57.58				
	1	10.5	0.57	8.3	D	47.15	57.50	ok	86.74				
	2	1	0.64	14.3	A	47.21	55.04	ok	110.10				
	3	4.5	1.03	9.5	D	47.20	77.85	ok	99.72				
RUN #2	3	7	1.09	18.0	A	47.30	57.03	ok	135.10				
	4	3	1.30	12.0	D	47.25	40.74	ok	125.76				
	7	3	2.21	12.5	D	47.20	69.09	ok	131.21				
	7	2	2.18	15.5	A	47.21	36.79	ok	163.64				
	8	4	2.54	16.0	D	47.14	68.24	ok	168.28				
											AVERAGE	MAX	MIN
RUN #3	8	8	2.64	12.0	D	47.10	105.51	ok	126.38	RUN #1:	71	110	0
	8	9	2.67	18.3	A	47.14	46.48	ok	160.93	RUN #2:	145	168	126
	10	3	3.12	11.0	D	47.19	64.47	ok	115.50	RUN #3:	134	161	116

TABLE 1 -Point Load Test Results

MOOSE LAKE ROAD OVERPASS - SBL STRUCTURE

HWY 69 FOUR-LANING

FROM THE SOUTH JUNCTION OF HIGHWAY 69 AND HIGHWAY 529 TO NAISCOOT LAKE

19-5161-21

MLR-S-09-12	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH			CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	BREAK	UCS (Mpa)				
RUN #1	0	4	0.10	21.5	D	47.23	102.14	ok	225.46				
RUN #2	4	8.5	1.44	8.5	A	47.20	54.95	ok	65.77				
	4	4	1.32	13.5	D	47.22	121.20	ok	141.62				
	6	2	1.88	14.0	D	47.18	100.20	ok	147.05				
	7	1	2.16	2.0	D	47.21	72.86	ok	20.99				
	7	10	2.39	13.0	D	47.20	87.19	ok	136.46				
	7	9	2.36	17.0	A	47.23	44.31	ok	155.33				
	9	1	2.77	12.0	D	47.14	82.07	ok	126.21				
RUN #3	10	1	3.07	7.5	D	47.22	85.24	ok	78.68				
	10	2	3.10	19.0	A	47.19	51.89	ok	153.71				
	11	3	3.43	12.0	D	47.26	94.55	ok	125.72				
	11	4	3.45	16.5	A	47.25	46.67	ok	144.77		AVERAGE	MAX	MIN
	12	2	3.71	11.5	D	47.21	87.76	ok	120.68	RUN #1:	225	225	225
	12	4	3.76	26.0	A	47.22	50.82	ok	213.66	RUN #2:	113	155	21
	14	1	4.29	14.5	D	47.19	61.53	ok	152.26	RUN #3:	141	214	79

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.

<u>UNIFIED SOILS CLASSIFICATION</u>

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

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

TERMS

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

STRENGTH CLASSIFICATION

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

THURBER

Ministry of Transportation Ontario

RECORD OF BOREHOLE No MLR-S-09-01

METRIC

G.W.P. 5076-06-00 | LOCATION N 5 053 668.4 E 235 547.6 | ORIGINATED BY ES

HWY 69 | BOREHOLE TYPE Hollow Stem Augers / NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2009-08-19 - 2009-08-19 | CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
201.2							
0.0	PEAT: (100mm)						
0.1 / 200.8 / 0.4	SAND, fine grained, trace gravel Brown to Grey Damp	1	SS	100/ 0.075	201	FI	
	BEDROCK, granitic gneiss, slightly weathered to fresh, grey, occasional mechanical breaks. Coring started at 0.5m. Sub-vertical fractures at 0.5m	1	RUN		200	2, 4, 2	RUN 1# TCR=92%, SCR=85%, RQD=85%, UCS=26MPa
		2	RUN		199	1, 1, 1, 0	RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=82MPa
197.6		3	RUN		198	3, 1, 0	RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=129MPa
3.5	END OF BOREHOLE AT 3.5m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.						

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Aug 25/09	0.08	201.12
Sep 14/09	0.90	200.30
Sep 24/09	1.20	200.00
Oct 07/09	1.00	200.20

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 40 80 120 160 200

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

GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity — 20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No MLR-S-09-02

METRIC

G.W.P. 5076-06-00
LOCATION N 5 053 672.7 E 235 545.1
ORIGINATED BY ES
HWY 69
BOREHOLE TYPE Visual Assessment and Manual Excavation
COMPILED BY AN
DATUM Geodetic
DATE 2009-08-19 - 2009-08-19
CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (40 80 120 160 200)	WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
201.3											
0.0	PEAT: (100mm)										
0.1	BEDROCK BELOW PEAT.						201				

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; X LAB VANE

PLASTIC LIMIT w_P; NATURAL MOISTURE CONTENT w; LIQUID LIMIT w_L

ONTMT4S 6121.GPJ 10/23/09

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No MLR-S-09-03

METRIC

G.W.P. 5076-06-00 LOCATION N 5 053 671.8 E 235 553.5 ORIGINATED BY ES

HWY 69 BOREHOLE TYPE Visual Assessment and Manual Excavation COMPILED BY AN

DATUM Geodetic DATE 2009-08-19 - 2009-08-19 CHECKED BY RPR

SOIL PROFILE			SAMPLES						DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40 80 120 160 200	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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE					
201.4												
0.0	**PEAT**, trace roots Dark Brown Moist (150mm)						201					
0.2	BEDROCK BELOW PEAT.											

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No MLR-S-09-04

METRIC

G.W.P.	5076-06-00	LOCATION	N 5 053 676.1 E 235 550.9	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Hollow Stem Augers / NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2009-08-19 - 2009-08-19	CHECKED BY	RPR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
201.5								
0.0	**PEAT:** (75mm)	1	SS	100/ 0.075				
0.1	**SAND**, fine grained Grey Damp							
0.2	**BEDROCK**, granitic gneiss, slightly weathered to fresh, grey, occasional mechanical break. Coring started at 0.2m	1	RUN			201	FI 2 0 0	RUN 1# TCR=83%, SCR=83%, RQD=83%, UCS=167MPa
	Sub-vertical fractures at the following depths: 50mm at 2.2m 50mm at 2.7m	2	RUN			200	1 2 0 2	RUN 2# TCR=100%, SCR=95%, RQD=95%, UCS=143MPa
198.1		3	RUN			199	5 0 1	
3.4	END OF BOREHOLE AT 3.4m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40 80 120 160 200

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

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No MLR-S-09-05

METRIC

G.W.P. 5076-06-00 LOCATION N 5 053 674.9 E 235 558.9 ORIGINATED BY ES

HWY 69 BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2009-08-19 - 2009-08-19 CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
202.2										
0.0	**PEAT:** (150mm)									
0.2	**SAND**, trace gravel Very Loose Brown Moist		1	SS	1		202			
	Wet		2	SS	100/ 0.150					
201.0 1.2	Layers of cobbles		1	RUN					FI	
	BEDROCK, granitic gneiss, slightly weathered to fresh, grey, occasional mechanical breaks Coring started at 1.2m		2	RUN			201		3, 2, 3	RUN 2# TCR=100%, SCR=95%, RQD=95%, UCS=137MPa
	Sub-vertical fractures: 75mm at 1.2m 50mm at 2.7m 275mm at 3.9m						200		3, 2	
			3	RUN			199		1, 2, 0, 0	RUN 3# TCR=100%, SCR=85%, RQD=85%, UCS=85MPa
198.0	75mm sub-horizontal fractures at 4.1m								>10	
4.2	END OF BOREHOLE AT 4.2m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 40 80 120 160 200

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No MLR-S-09-06

METRIC

G.W.P. 5076-06-00 LOCATION N 5 053 679.2 E 235 556.5 ORIGINATED BY ES

HWY 69 BOREHOLE TYPE Visual Assessment and Manual Excavation COMPILED BY AN

DATUM Geodetic DATE 2009-08-19 - 2009-08-19 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40 80 120 160 200)	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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
202.1											
0.0	**PEAT:** (150mm)						202				
0.2	BEDROCK BELOW PEAT.										

ONTMT4S 6121.GPJ 10/23/09

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

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No MLR-S-09-07

METRIC

G.W.P. 5076-06-00 | LOCATION N 5 053 688.4 E 235 536.2 | ORIGINATED BY ES

HWY 69 | BOREHOLE TYPE Visual Assessment and Manual Excavation | COMPILED BY AN

DATUM Geodetic | DATE 2009-08-19 - 2009-08-19 | CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER (SAMPLES)	TYPE	"N" VALUES	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) 40 80 120 160 200	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 (%) GR SA SI CL
202.7 / 0.0	**PEAT**, fibrous, some sand, occasional roots Dark Brown Damp		1	GS							
202.2 / 0.5	BEDROCK BELOW PEAT.						202				

$+^3, \times^3$: Numbers refer to Sensitivity   15-20-5-10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No MLR-S-09-08

METRIC

G.W.P. 5076-06-00 LOCATION N 5 053 692.7 E 235 533.7 ORIGINATED BY ES

HWY 69 BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2009-08-20 - 2009-08-20 CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
203.5								
0.0	PEAT: (125mm)							
0.1	**BEDROCK**, granitic gneiss, slightly weathered to fresh, grey, occasional mechanical breaks Coring started at 0.1m Sub-vertical fractures: 75mm at 0.3m 275mm at 0.7m 50mm at 1.1m 125mm at 2.0m 25mm at 3.1m	1	RUN			203	FI >10 5 >10	RUN 1# TCR=100%, SCR=30%, RQD=30%, UCS=42MPa
		2	RUN			202 201	0 0 0 1	RUN 2# TCR=100%, SCR=100%, RQD=92%, UCS=129MPa
		3	RUN				1 0 3	RUN 3# TCR=100%, SCR=100%, RQD=88%, UCS=152MPa
200.2 3.3	END OF BOREHOLE AT 3.3m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE: 40 80 120 160 200
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No MLR-S-09-09

METRIC

G.W.P. 5076-06-00 | LOCATION N 5 053 691.7 E 235 542.1 | ORIGINATED BY ES

HWY 69 | BOREHOLE TYPE Visual Assessment and Manual Excavation / NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2009-08-20 - 2009-08-20 | CHECKED BY RPR

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT (FI)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
203.0 / 0.0	**PEAT:** (25mm)				203		
	BEDROCK, granitic gneiss, slightly weathered to fresh, grey, occasional mechanical breaks. Coring started at 0.1m	1	RUN			FI, 0, 0	RUN 1# TCR=100%, SCR=98%, RQD=98%, UCS=71MPa
	425mm sub-vertical fractures at 1.6m; sub-horizontal fractures at 2.0m	2	RUN		202, 201	2, 0, 1, 2, 2	RUN 2# TCR=100%, SCR=72%, RQD=72%, UCS=145MPa
199.8 / 3.2		3	RUN		200	0, 1, 0	RUN 3# TCR=92%, SCR=92%, RQD=92%, UCS=134MPa

END OF BOREHOLE AT 3.2m.
Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Aug 25/09	0.04	202.96
Sep 14/09	1.30	201.70
Sep 24/09	1.60	201.40
Oct 07/09	1.50	201.50

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 40 80 120 160 200); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GR SA SI CL; STRAT PLOT; GROUND WATER CONDITIONS.

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

RECORD OF BOREHOLE No MLR-S-09-10

METRIC

G.W.P. 5076-06-00	LOCATION N 5 053 696.1 E 235 539.6	ORIGINATED BY ES
HWY 69	BOREHOLE TYPE Visual Assessment and Manual Excavation	COMPILED BY AN
DATUM Geodetic	DATE 2009-08-19 - 2009-08-19	CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (40 80 120 160 200)	WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
203.8											
0.0 / 0.1	**PEAT:** (75mm)										
	BEDROCK BELOW PEAT.						203				

SHEAR STRENGTH legend: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE. PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L.

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No MLR-S-09-11

METRIC

G.W.P. 5076-06-00	LOCATION N 5 053 694.9 E 235 547.5	ORIGINATED BY ES
HWY 69	BOREHOLE TYPE Visual Assessment and Manual Excavation	COMPILED BY AN
DATUM Geodetic	DATE 2009-08-19 - 2009-08-19	CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40 80 120 160 200	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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
202.9											
0.0	**PEAT:** (25mm)										
	BEDROCK BELOW PEAT.						202				

+3, ×3: Numbers refer to Sensitivity

15-⌖-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No MLR-S-09-12

METRIC

G.W.P. 5076-06-00 LOCATION N 5 053 699.2 E 235 544.9 ORIGINATED BY ES

HWY 69 BOREHOLE TYPE Hollow Stem Augers / NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2009-08-19 - 2009-08-19 CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
203.1 / 0.0	PEAT: (75mm)							
0.1	Gravelly SAND, some silt, trace clay; Loose to Very Dense; Brown; Moist	1	SS	6	203	○		
		2	SS	137		○	FI	23 59 16 2
201.9 / 1.2	BEDROCK, granitic gneiss, slightly weathered to fresh, grey. Coring started at 1.2m	1	RUN		202		0	RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=225MPa
	Sub-vertical fractures: 75mm at 1.9m; 100mm at 2.2m; 75mm at 2.7m; 75mm at 4.1m	2	RUN		201		1, 0, 2, >5, 1	RUN 2# TCR=85%, SCR=85%, RQD=85%, UCS=113MPa
		3	RUN		200		1, 1, 1, 0	RUN 3# TCR=100%, SCR=100%, RQD=95%, UCS=141MPa
198.8	50mm sub-horizontal fractures at 4.1m				199		>3	
4.3	END OF BOREHOLE AT 4.3m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 40 80 120 160 200
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   (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No MLR-S-09-13

METRIC

G.W.P.	5076-06-00	LOCATION	N 5 053 665.3 E 235 557.3	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Visual Assessment and Manual Excavation	COMPILED BY	AN
DATUM	Geodetic	DATE	2009-08-19 - 2009-08-19	CHECKED BY	RPR

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE (40 80 120 160 200)	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 (%) GR SA SI CL
201.6											
0.0	**PEAT:** (50mm)										
	BEDROCK BELOW PEAT.						201				

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

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No MLR-S-09-14

METRIC

G.W.P. 5076-06-00

LOCATION N 5 053 702.6 E 235 535.9

ORIGINATED BY ES

HWY 69

BOREHOLE TYPE Visual Assessment and Manual Excavation

COMPILED BY AN

DATUM Geodetic

DATE 2009-08-19 - 2009-08-19

CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (40 80 120 160 200)	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 (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
203.6											
0.0	**PEAT:** (250mm)										
203.4 / 0.3	BEDROCK BELOW PEAT.										
							203				

ONTMT4S 6121.GPJ 10/23/09

+ 3, × 3: Numbers refer to Sensitivity

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

Appendix B

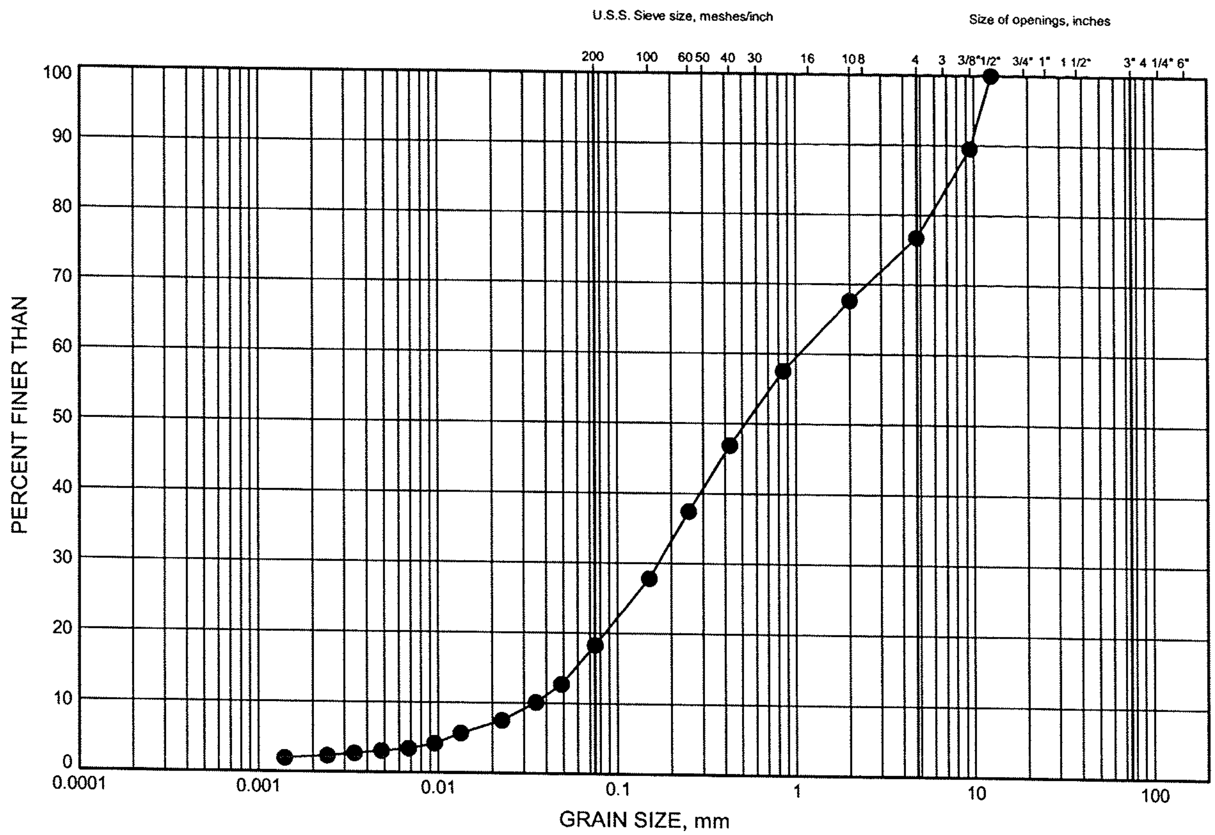
Laboratory Test Results

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B1

GRAVELLY SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MLR-S-09-12	0.89	202.23

W.P.# 5076-06-00

Prepared By AN

Checked By RPR

THURBER

GRAIN SIZE DISTRIBUTION - THURBER 6121.GPJ 10/6/09

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Footing on Native Soil	Footings on Bedrock	Driven Piles	Augered Caissons
Advantages: i. Economical to install. ii. Ease of construction	*Advantages:* i. Bedrock comparatively close to the underside of the girders. ii. Short abutment stem. iii. High values of geotechnical resistance are available on the bedrock iv. Allows footing to be placed close to edge of the rock cut.	*Advantages:* i. High geotechnical resistance available by driving piles to achieve resistance in the bedrock. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed.	*Advantages:* i. High geotechnical resistance available for units founded on bedrock. ii. Construction of caissons could continue in freezing weather.
Disadvantages: i. Soil conditions encountered at this site are considered unsuitable.	*Disadvantages:* i. Stepped footing may be required ii. High cost of excavation, if any is required iii. Mass concrete fill required to create a level founding surface. iv. Groundwater control will be required	*Disadvantages:* i. Proximity of bedrock surface to the underside of the girders mitigates against the use of driven piles unless bedrock is excavated to accommodate an integral abutment. ii. Higher unit cost compared to footings	*Disadvantages:* i. Higher unit cost compared to other foundation options such as footings or driven piles. ii. High risks associated with inflow of groundwater. iii. Soils are considered to be unsuitable. iv. Proximity of bedrock surface and short length of caissons not likely to justify costs.
NOT RECOMMENDED	**RECOMMENDED**	**NOT RECOMMENDED**	**NOT RECOMMENDED**

Appendix D

List of SPs and OPSS, and Suggested Text for Selected NSSP

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

- OPSS 572
- OPSD 201.020
- OPSS 902 as amended by Special Provision 902S01.
- OPSD 3101.200.
- SP 110F13 Amendment to OPSS 1010, March 1993.
- SSP 105S10.
- OPSD 3101.150
- OPSD 3101.200
- SP206S03, dated July 2007.

2. Suggested text for a NSSP on Stabilization of Rock Excavation

The Contractor is advised that the natural pattern of joints and fractures within the rock mass may lead to the development of potentially unstable wedges, or other unfavourable conditions, below the structure foundations.

The Contractor shall hire a rock slope stability engineer who shall be a Professional Engineer registered in the Province of Ontario and who has a minimum of five (5) years of experience in assessing rock slope stability and bearing resistance on rock slopes. The rock slope stability engineer shall also have a minimum of five (5) years of experience in designing rock stabilization works that include, but are not necessarily limited to: rock bolts, rock anchors, mass concrete and sprayed concrete.

On completion of scaling and mucking in accordance with SP206S03, the Contractor's rock slope stability engineer shall:

1. Inspect the rock face in front of the foundations and 10 m beyond the structure limits and identify all potentially unstable rock wedges or blocks or other conditions unfavourable for the support of the structure foundations

2. Design such reinforcement or remedial works as shall be necessary to ensure the long term stability of the structure foundations.

3. Sign and stamp the designs and accompanying drawings and submit them to the Contractor.

On receipt of the designs and drawings from the rock slope stability engineer, the Contractor shall construct the required reinforcement and remedial works.

On completion of these works, the Quality Verification Engineer shall submit a Certificate of Conformance to the Contract Administrator.

NSP 98 NORTHEASTERN

REQUIREMENTS FOR BLASTING

1. GENERAL

1.1 The Contractor shall comply with OPSS 120 unless otherwise noted in the Contract.

1.2 For the purpose of work related to blasting, a Blasting Consultant is defined as: A Professional Engineer licensed to practice in the Province of Ontario with a minimum of five years (5) experience related to blasting.

The Blasting Consultant shall be retained by the Contractor and shall be independent of the Contractor and any subcontractor doing blasting work. The Blasting Consultant shall be required to complete the specified monitoring of vibration levels and provide a report detailing the vibration levels and copies of the recorded ground vibration documents to the Contractor and the Contract Administrator immediately following each blast and prior to the next blast.

1.3 All blasting shall be designed and carried out in a manner, such that no damage occurs to the buildings or equipment.

1.4 Under no circumstances will the Contractor blast within the vicinity of any TransCanada Pipelines, Union Gas Pipelines, Ontario Hydro lines and Bell Fibre optics lines without a representative from TransCanada Pipelines, Ontario Hydro or Bell Canada on site.

The Contractor shall notify the following appropriate representative 72 hours in advance of blasting.

* Fill - in as appropriate for the project

Hydro One Networks Inc.
45 Sarjeant Drive
Box 6700
Barrie, ON
L4M 5N5

Attn: Arthur Conlon
Telephone : (888) 238-2398
Fax: (705) 746-7293

Bell Canada
9 High Street
Huntsville, ON
P1H 1P2

Attn: Timothy Beachy
Telephone: (705) 789-9638
Fax: (705) 789-6223

The Contractor shall notify the following appropriate representative 10 days in advance of blasting.

* Fill - in as appropriate for the project

TransCanada Pipelines Ltd.
801 Seventh Avenue S.W.
P.O. Box 2535, Station M
Calgary, Alberta

T2P 2N6

Attn: Elio Ramos
Telephone: (403) 261-8256
and Ron Marsh
(705) 840-7454

Union Gas Limited
P.O. Box 3040
36 Charles Street East
North Bay, ON
P1B 8K7

Attn: Jeff Peroff
Telephone: (705) 475-7923
Fax: (866) 252-2012

2.0 DESIGN AND SUBMISSION REQUIREMENTS

2.1 Section 120.04.02 b) iii is deleted and replaced with:

A letter signed by the Contractor certifying that a pre-blast survey has been carried out in accordance with the Pre-Blast Survey subsection. A copy of the pre-blast survey shall be provided to the Contract Administrator.

3.0 EQUIPMENT

3.1 Section 120.06.02 is amended by the addition of the following:

The transducer used to measure ground vibration levels shall be coupled to the ground by either pinning or burying.

4.0 CONSTRUCTION

4.1 The last bullet (c) of Section 120.07.03 is deleted and replaced with the following:

(c) Clear quality 35 mm photographs and/or DVD videos, and written report necessary for proper recording of areas of concern and condition of the property. Photographs and DVDs shall be clearly labeled as to location.

4.2 Section 120.07.03 is amended by the addition of the following:

The contractor will be supplied with a "Permission to Enter For Pre-Blast and Post-Blast Inspection" form that the contractor shall use to record the permission to carry out an inspection.

4.3 Section 120.07.05.02 is amended by the addition of the following:

TransCanada Pipelines has requirements in addition to Table 1. When providing vibration monitoring within 100m of the TransCanada pipeline, the Contractor shall ensure that the following vibration limits in the vicinity of the pipeline are measured and are met:

- Vibrations are to be controlled to a maximum peak particle velocity of 50mm/s above the pipeline.

- Should any two consecutive seismographic readings fall between 50 and 80mm/s the pipeline is to be exposed and monitored to ensure that a third reading taken on the pipe falls below 50mm/s.
- should any seismographic reading taken below the pipe fall above 80mm/s or taken on the pipe fall above 50mm/s the loading pattern should be adjusted to fall below these limits.
- Delays shall be designed to prevent double readings.

When blasting is within 100m of the TransCanada pipeline, a TransCanada representative and /or blasting consultant may be present to monitor vibrations and any other effects on the pipeline.

5.0 RESPONSIBILITY

This special provision in no way intends to remove any of the responsibility for a safe blast from the Blasting Contractor.

6.0 BASIS OF PAYMENT

Compensation for the Contractor to provide blast monitoring and schedule his operations in accordance with these requirements, including all equipment, labour and materials, shall be deemed to be included in the contract bid price for the various tender items.

Note to designer:

Confirm with the Contracts Office whether the minimum distance for the Pre-Blast Survey should be increased.

Appendix E

Site Photographs

Photograph – Moose Lake Road SBL Structure, South Approach, Borehole MLR-S-09-13

Photograph – Moose Lake Road SBL Structure, South Abutment, Borehole MLR-S-09-02

Photograph – Moose Lake Road SBL Structure, South Abutment, Borehole MLR-S-09-06

Photograph – Moose Lake Road SBL Structure, North Abutment, Borehole MLR-S-09-10

Appendix F

Drawing
Borehole Locations and Soil Strata

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

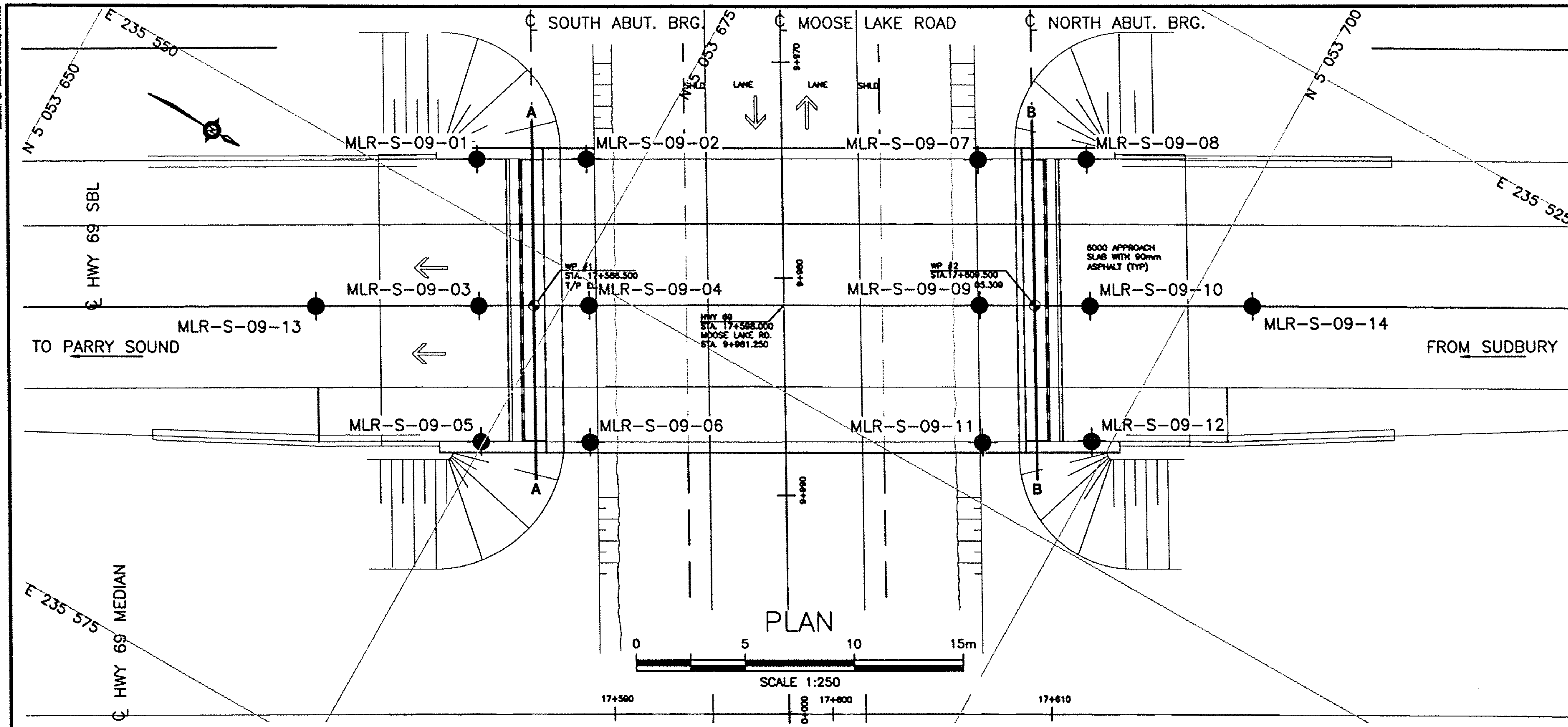
CONT No
WP No 5196-06-01

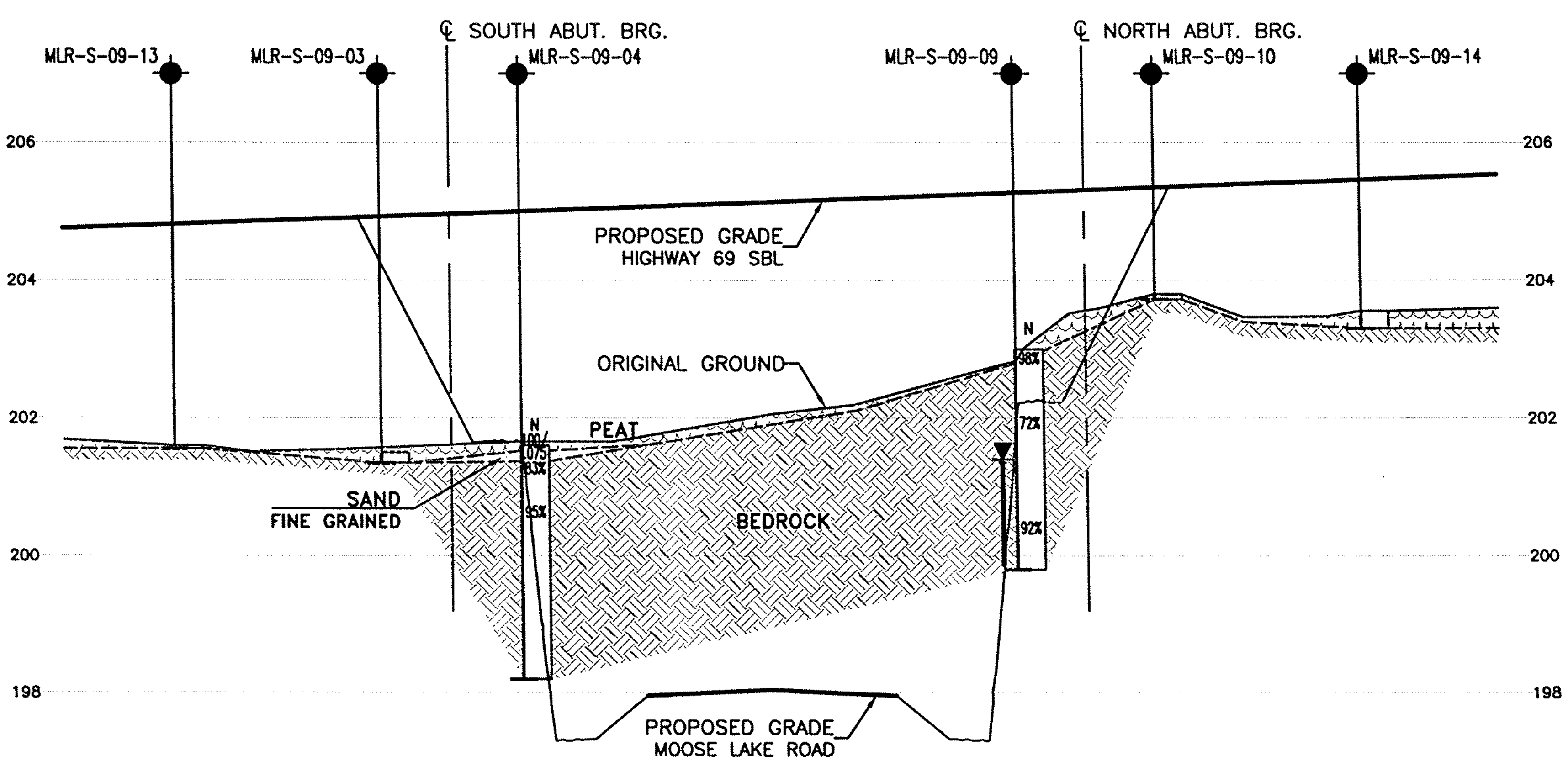
HIGHWAY 69 FOUR-LANING
MOOSE LAKE ROAD OVERPASS
SOUTHBOUND LANES
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

MMM GROUP

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

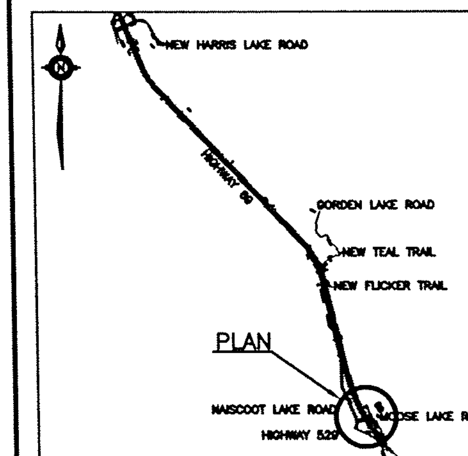




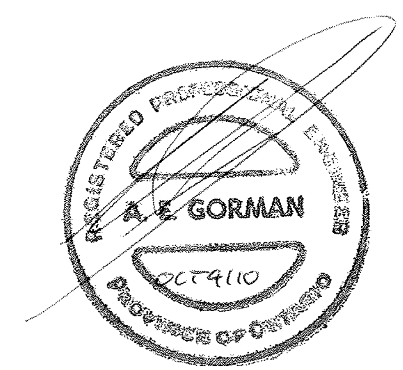
PROFILE ALONG HIGHWAY 69 SBL

0 5 10 15m H 1:250

0 2 4 6m V 1:100



KEYPLAN



LEGEND

	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
MLR-S-09-01	201.1680	5053668.3710	235547.6240
MLR-S-09-02	201.3250	5053672.7220	235545.1240
MLR-S-09-03	201.3640	5053671.7570	235553.5110
MLR-S-09-04	201.5390	5053676.1230	235550.9840
MLR-S-09-05	202.1960	5053674.8950	235558.9200
MLR-S-09-06	202.0970	5053679.2250	235556.4580
MLR-S-09-07	202.6930	5053688.3750	235536.2000
MLR-S-09-08	203.4640	5053692.6800	235533.7280
MLR-S-09-09	203.0050	5053691.6920	235542.0590
MLR-S-09-10	203.8210	5053696.1050	235539.5850
MLR-S-09-11	202.8970	5053694.8760	235547.5190
MLR-S-09-12	203.1170	5053699.1830	235544.9990
MLR-S-09-13	201.6240	5053665.2810	235557.2520
MLR-S-09-14	203.6150	5053702.6060	235535.8830

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

2) This drawing is for subsurface information only. Surface details, proposed road grades and features are for conceptual illustration.

GEOCRES No. 41H-78

BENCHMARK
NAME VCP 100
ELEVATION 203.554
DESCRIPTION TOP OF RIB 19mm x 1.82m 17+339.32 119.98 RIGHT TWP OF HARRISON

REVISIONS

DATE	BY	DESCRIPTION

DESIGN RPR	CHK AEG	CODE	LOAD	DATE OCT. 2010	
DRAWN MFA	CHK PKC	SITE 44-448/2	STRUCT	DWG 2-1	

CONT No
WP No 5196-06-01

HIGHWAY 69 FOUR-LANING
MOOSE LAKE ROAD OVERPASS
SOUTHBOUND LANES
BOREHOLE LOCATIONS AND SOIL STRATA

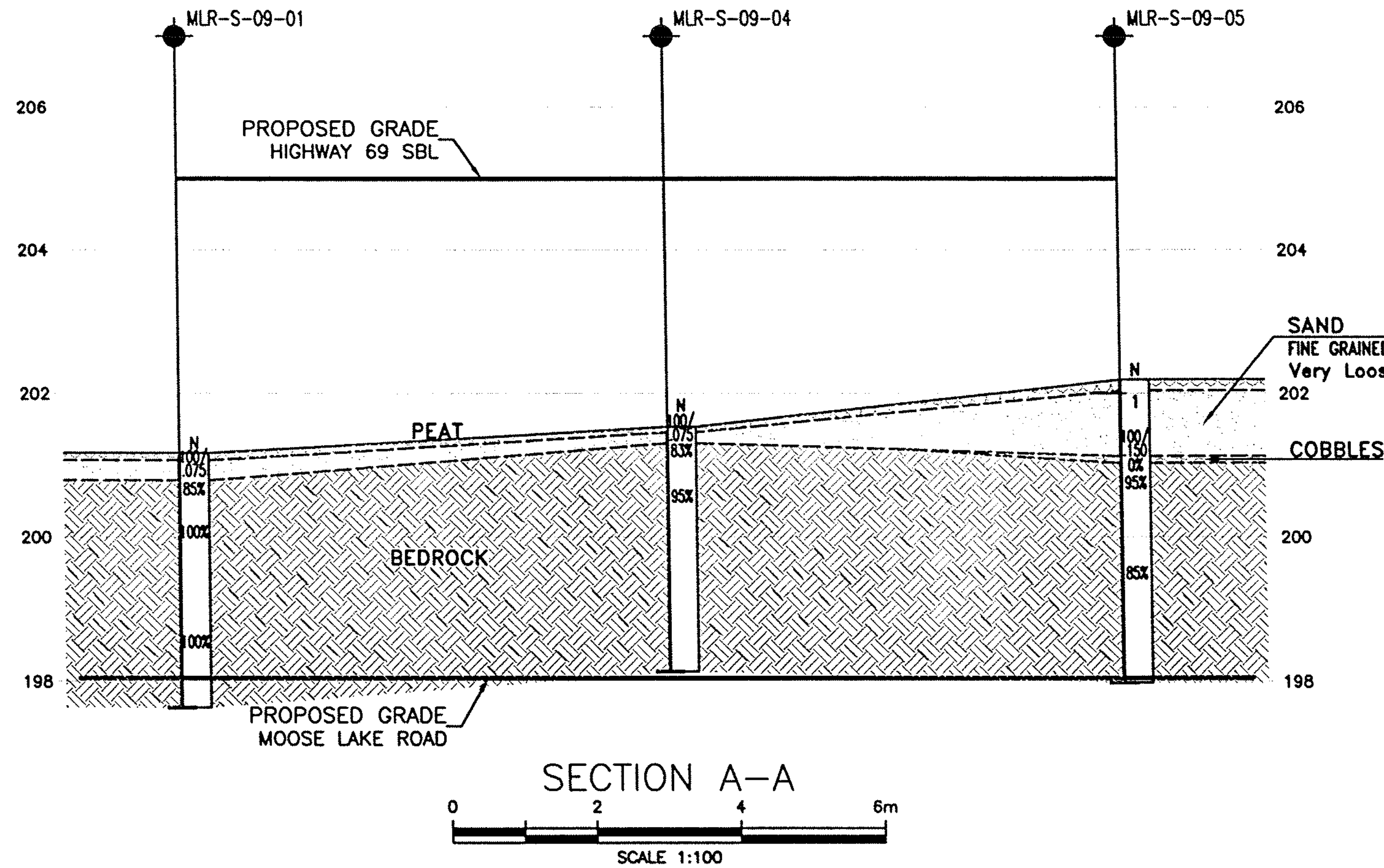
SHEET

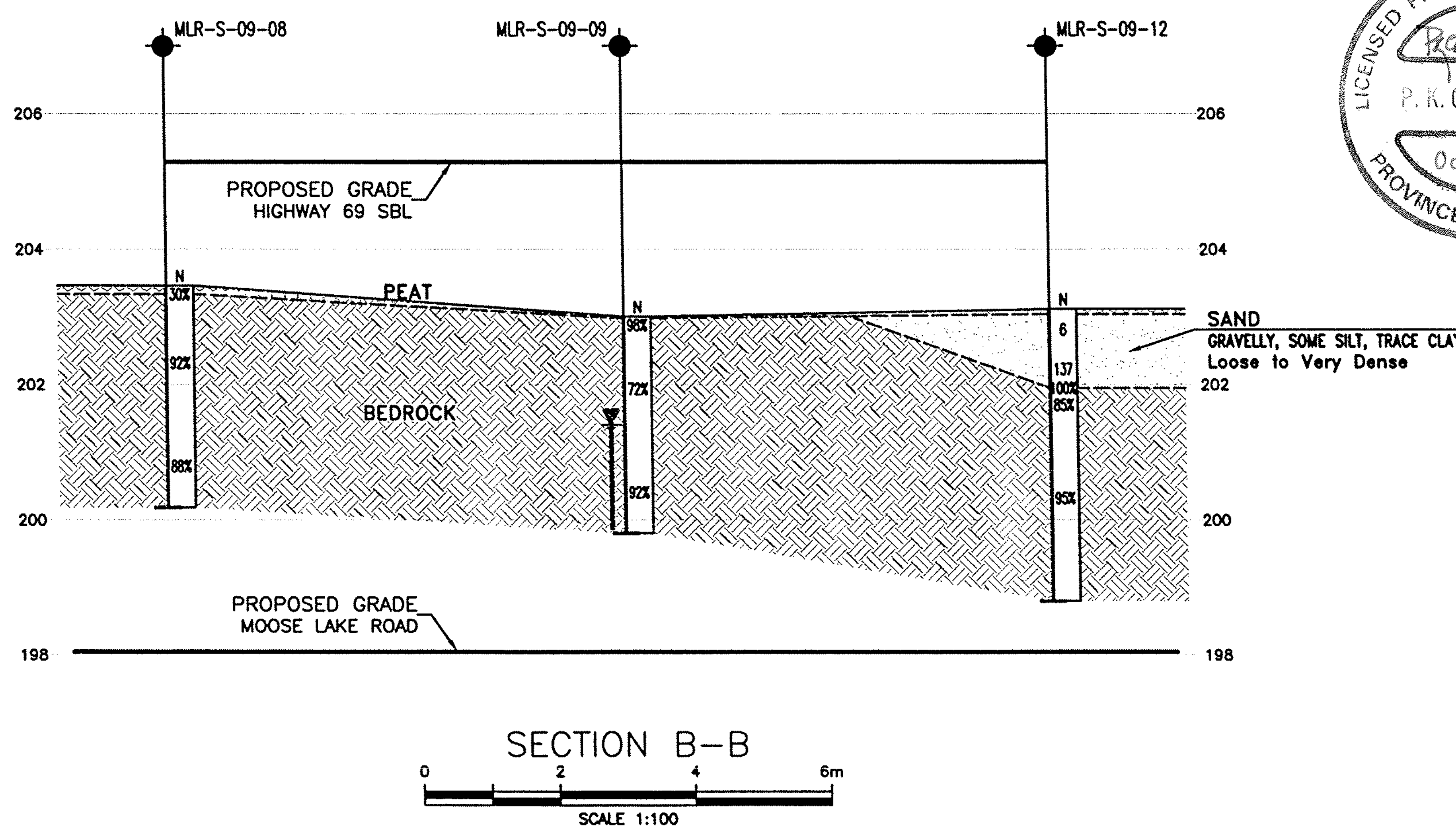
MMM GROUP

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

METRIC

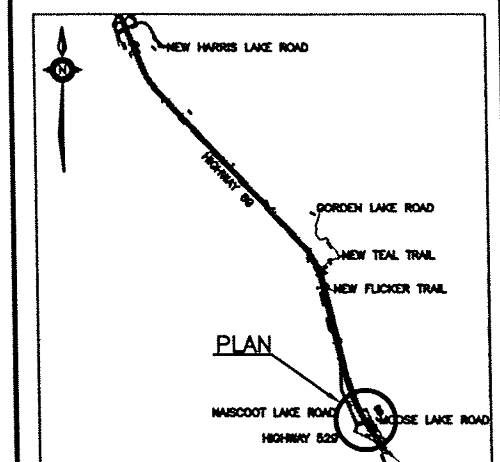
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN





A. E. GORMAN
REGISTERED PROFESSIONAL ENGINEER
PROVINCE OF ONTARIO
Oct 4/10

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
PROVINCE OF ONTARIO
Oct 4/10



KEYPLAN

LEGEND

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DESIGN RPR	CHK AEG	CODE	LOAD	DATE OCT. 2010
DRAWN MFA	CHK PKC	SITE 44-448/2	STRUCT	DWG 2-2