



FOUNDATION INVESTIGATION AND DESIGN REPORT

for

**LOVERING CREEK BRIDGE SOUTHBOUND
HIGHWAY 69**

**SITE NO. 46-509/2, W.P. 5263-05-01
DISTRICT 54, SUDBURY**

***PHASE 1, STA. 12+200 TO 15+400
TOWNSHIP OF SERVOS***

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PML Ref.: 06TF052D
Index No.: 427FIR and 428FDR
GEOCRES No.: 41I-214
November 16, 2007



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for

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DISTRICT 54, SUDBURY

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Explanation of Terms Used in Report

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Appendix B – Site Photographs

FOUNDATION INVESTIGATION REPORT

for
Lovering Creek Bridge Southbound
Highway 69
Site No. 46-509/2, W.P. 5263-05-01,
District 54, Sudbury, Ontario

*Phase 1, Sta. 12+200 to 15+400
Township of Servos*

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed new Highway 69 Lovering Creek Bridge Southbound about 43.7 km south of Sudbury. The investigation was conducted for Totten Sims Hubicki Associates Limited on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed Lovering Creek Bridge Southbound carries the new Highway 69 southbound lanes (SBL) between approximate Sta. 14+404.0 and 14+448.0.

This report provides subsurface information pertaining to the proposed new Highway 69 SBL overpass and approaches within about 20 m of the abutments, between approximate Sta. 14+384.0 and 14+468.0.

2. SITE DESCRIPTION AND GEOLOGY

The site is located approximately 43.7 km south of Sudbury, about 500 m east of the existing Highway 69 at Lovering Creek. The creek flows from east to west draining the Lovering Lake into Rock Bay.

The local topography is irregular and comprises wooded ground separated by steep rock ridges. The terrain has a general rugged topography with numerous rock outcrops dominated by the Servos Pluton north of Rock Bay. Soil cover over the rock outcrops is generally sparse. Boulders are also present at the site location, in particular at the south near the toe of a prominent outcrop where a talus slope bouldery deposit is inferred.



The site is generally located in the Precambrian Laurentian peneplane. In general, Metasedimentary rocks of the Huronian Supergroup and gneisses of the Grenville Province underlie the alignment.

3. INVESTIGATION PROCEDURES

The field work for the north and south abutments for Lovering Creek Southbound was carried out during the period from April 4 to April 30, 2007.

The scope of the subsurface investigation comprised 8 boreholes. The boreholes were drilled to depths of 0.0 to 1.1 m at the locations shown on Drawing LCS-1, appended. Two boreholes, LCS-3 and LCS-5, were cored 5.2 and 3.0 m into the bedrock, respectively.

Sutcliffe Rody Quesnel (SRQ) Inc. staked the alignment of Highway 69 at the structure location. Peto MacCallum Ltd. (PML) selected the positions of the boreholes along the staked alignment and determined the ground surface elevations at the borehole locations. SRQ Inc. provided the following temporary benchmarks (TBM) established on existing ground level at the working points (WP) for each of the foundation units:

TBM	DESCRIPTION	ELEVATION (*)
TBM1	Existing ground at North abutment WP	195.7
TBM2	Existing ground at South abutment WP	204.7

(*) Geodetic, metric

The boreholes were advanced by using portable drilling equipment, as required by the very restrictive site accessibility and prevalent weather limitations. All the boreholes, except for boreholes LCS-3 and LCS-5, were advanced using manual augering methods. Boreholes LCS-3 and LCS-5 were advanced using a manually operated electric drill (HILTI) equipped for NQ diamond rock coring and washboring. The power equipment was supplied and operated by a specialist drilling contractor. The drilling crews worked under the full-time supervision of a member of our engineering staff.



Photographs of the NQ-size rock cores obtained in boreholes LCN-3 and LCN-5 are shown in Appendix A.

The boreholes were backfilled in accordance with the MTO guidelines and MOE regulation 903 for borehole abandonment procedures using a bentonite/cement mixture grout.

The groundwater conditions at the borehole locations were assessed during drilling and when appropriate, by measurement of the water level in the open boreholes.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. However, due to the practically non-existent or very shallow inorganic soil cover over the bedrock/probable bedrock surface, insufficient soil samples were recovered from the boreholes for laboratory testing.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations and groundwater observations. Site photographs are provided in Appendix B.

The borehole locations and stratigraphic profile and cross-sections prepared from the borehole data are presented on Drawing LCS-1.

The soil cover revealed in the boreholes varies from 0.0 (at exposed rock outcrops) to 1.1 m deep deposits. The deeper deposit was found under 0.3 m of open water. The soil cover at the borehole locations generally comprises topsoil and peat containing scattered cobbles and boulders mantling bedrock. The local thin deposit of silty sand was encountered in borehole LCS-7.

4.1 Peat and Topsoil

A surficial layer of topsoil locally mixed with cobbles and boulders is present at the ground surface in boreholes LCS-2, LCS-4 and LCS-6 and extends 0.1 to 0.5 m depths from ground surface,



elevations 195.2 to 203.7. Locally in borehole LCS-7 a 600 mm thick layer of peat containing occasional boulders was encountered below 0.3 m of open water to 0.9 m depth, elevation 194.6.

Refusal on probable bedrock below peat/topsoil was encountered in boreholes LCS-2 and LCS-4, elevations 203.7 and 201.9, respectively.

Refusal on probable boulders was encountered in borehole LCS-6 at 0.5 m depth from ground surface, elevation 195.2.

4.2 Silty sand

A localized 200 mm thick deposit of cohesionless silty sand was encountered below peat/topsoil layer in borehole LCS-7 and extends to 1.1 m depth, elevation 194.4. The layer comprises grey silty sand with gravel. The borehole could not be advanced past 1.1 m depth due to presence of probable boulders.

4.3 Cobbles and Boulders

A localized surficial deposit of boulders was encountered at the surface in borehole LCS-1 and extends to the 0.2 m termination depth, elevation 210.2, of the borehole on probable bedrock. Cobbles and boulders were also encountered in other boreholes at the site in particular mixed in the surficial topsoil in borehole LCS-3 where it likely comprises a shallow talus slope. In this borehole, the unit extended from the surface to a depth of 0.5 m, elevation 204.2.

4.4 Bedrock

A detailed description of the rock cores retrieved from boreholes LCS-3 (south abutment) and LCS-5 (north abutment) is provided in Table A and summarized on the record of borehole logs. An outcrop was encountered surficially in boreholes LCS-5 and LCS-8. The bedrock comprises about 4 m of light grey to black gabbro over light grey migmatite in borehole LCS-3, and medium grey migmatite in borehole LCS-5. The rock is typically slightly weathered to unweathered and exhibits medium to high strength.



At the north abutment borehole LCS-5, the bedrock is confirmed by drilling 3.0 m from the surface, elevation 197.6. Photographs of the rock core taken from borehole LCS-5 are shown on Photos 1 and 2, Appendix A. The bedrock is estimated at the same levels or lower than 0.5 and 1.1 m termination depths of boreholes LCS-6 and LCS-7 (elevations 195.2 and 194.4), respectively.

At the south abutment, the bedrock surface is confirmed by drilling 5.2 m from 0.5 to 5.7m depths, elevations 204.2 to 199.0, in borehole LCS-3. Photographs of the rock core taken at borehole LCS-3 are shown on Photos 3 and 4. Bedrock was also inferred in boreholes LCS-2 and LCS-4 by refusal at depths of 0.1 and 0.2 m below ground surface, elevations 201.9 and 203.7, indicating a maximum bedrock surface level difference of 2.3 m between borehole locations. The slope of the bedrock surface between the boreholes at the south abutment ranges from 4 to 19°.

In the north and south abutments boreholes, the measured core recovery typically ranges from 93 to 100%, with one isolated value of 82% in borehole LCS-3. The RQD determined from the north abutment borehole LCS-5 rock cores ranges from 30 to 84%, indicating poor to good rock quality. The range of RQD for borehole LCS-3 at the south abutment is between 11 and 100%, indicating very poor to excellent rock quality.

4.5 Groundwater

Open surface water at elevation 195.5 was observed at borehole LCS-7 during and upon completion of drilling. It is inferred that this water is influenced by the water level in the Lovering Creek. No groundwater was observed in the remaining boreholes during and upon completion of drilling. The water level in the creek was at elevation 194.8 on the west and 195.6 on the east side of the proposed bridge deck.

However, the groundwater is subject to fluctuations at the site due to seasonal conditions and rainfall patterns.



5. CLOSURE

The field work was carried out under the supervision of Mr. N. Lee-Bun, and direction of Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer. Marathon Drilling Co. Ltd. and City Concrete Drilling supplied the soil and rock drilling equipment.

The report was prepared by Mr. C. M. P. Nascimento, P.Eng., and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Yours very truly,

Peto MacCallum Ltd.

A handwritten signature in black ink, appearing to read 'C. M. P. Nascimento', written in a cursive style.



C. M. P. Nascimento, P.Eng.,
Senior Project Engineer

A handwritten signature in black ink, appearing to read 'Brian R. Gray', written in a cursive style.



Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

CN/BRG:nr-lnr



TABLE A
 ROCK CORE DESCRIPTION

HOLE NO.	CORE RECOVERY				CORE DESCRIPTION	
	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
LCS-3	1	0.5 - 1.5	82	11	0.5 - 4.5	GABBRO: Dark grey to black or brown, fine crystalline, with occ. schist like layers, with rust oxidation on partings, medium to high strength, slightly weathered to unweathered, very close to close becoming moderate spaced flat to dipping partings, occ. vertical parting, rough planar, tight to oxidized, variably very poor to excellent quality.
	2	1.5 - 3.3	93	61		
	3	3.3 - 3.9	100	75		
	4	3.9 - 4.5	100	100		
	5	4.5 - 5.7	94	42		
LCS-5	1	0.0 - 0.9	97	84	0.0 - 3.0	MIGMATITE: Medium grey, fine to medium crystalline, slight banding, with pink, coarse crystalline layers, becoming medium to dark grey, garnetiferous, dipping to vertical bands, high strength, slightly weathered to unweathered, close to moderate spaced flat to dipping partings, rough planar, silty to oxidized, becoming tight, poor to good quality.
	2	0.9 - 2.1	95	30		
	3	2.1 - 3.0	100	82		

NOTES: RQD: Rock Quality Designation

Originated: JFW
 Compiled: NR
 Checked: CN

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S SPLIT SPOON	T P THINWALL PISTON
W S WASH SAMPLE	O S OSTERBERG SAMPLE
S T SLOTTED TUBE SAMPLE	R C ROCK CORE
B S BLOCK SAMPLE	P H T W ADVANCED HYDRAULICALLY
C S CHUNK SAMPLE	P M T W ADVANCED MANUALLY
T W THINWALL OPEN	F S FOIL SAMPLE
F V FIELD VANE	

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	l	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	l	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	l	COMPRESSION INDEX
C_s	l	SWELLING INDEX
C_α	l	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
M	m	DRAINAGE PATH
T_v	l	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	l	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	n	l, %	POROSITY	e_{max}	l, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	w	l, %	WATER CONTENT	e_{min}	l, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m^3	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	l	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m^3	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	l	UNIFORMITY COEFFICIENT
ρ_d	kg/m^3	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $\frac{w_L - w_p}{I_p}$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_L	l	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m^3/s	RATE OF DISCHARGE
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_C	l	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	l	HYDRAULIC GRADIENT
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	WTPL		WETTER THAN PLASTIC LIMIT	j	kN/m^3	SEEPAGE FORCE
e	l, %	VOID RATIO						

RECORD OF BOREHOLE No LCS-1 1 of 1 METRIC

G.W.P. 5263-05-01 LOCATION Hwy 69(New), Sta. 14+384, o/s 18.8m Lt. of CL Med. ORIGINATED BY N.L.B.
 DIST 54 HWY 69 BOREHOLE TYPE Manual Augering COMPILED BY N.R.
 DATUM Geodetic DATE April 17, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
210.4	Ground Surface																
0.0	Boulders	○															
0.2	End of Borehole																
	Refusal on probable bedrock																
	* Borehole dry																

RECORD OF BOREHOLE No LCS-2 1 of 1 **METRIC**

G.W.P. 5263-05-01 LOCATION Hwy 69(New), Sta. 14+404, o/s 25.8m Lt. of CL Med. ORIGINATED BY N.L.B.
 DIST 54 HWY 69 BOREHOLE TYPE Manual Augering COMPILED BY N.R.
 DATUM Geodetic DATE April 24, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80					
203.9	Ground Surface															
0.0	Topsoil															
0.2	End of Borehole															
	Refusal on probable bedrock															
	* Borehole dry															

RECORD OF BOREHOLE No LCS-3 1 of 1 **METRIC**

G.W.P. 5263-05-01 LOCATION Hwy 69(New), Sta. 14+404, o/s 18.8m Lt. of CL Med. ORIGINATED BY N.L.B.
 DIST 54 HWY 69 BOREHOLE TYPE Portable Rotary Drill and Washboring COMPILED BY N.R.
 DATUM Geodetic DATE April 24, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
						20 40 60 80 100											
204.7	Ground Surface																
0.0	Topsoil cobbles and boulders																
204.2	Bedrock																
0.5	Gabbro: dark grey to black or brown medium to high strength slightly weathered to unweathered very poor to excellent quality		1	RC NQ	REC 82%												RQD 11%
			2	RC NQ	REC 93%												RQD 61%
			3	RC NQ	REC 100%												RQD 75%
			4	RC NQ	REC 100%												RQD 100%
	Migmatite: light grey medium to high strength unweathered poor quality		5	RC NQ	REC 94%												RQD 42%
199.0	End of borehole																
5.7																	
	* Borehole charged with drilling water																

RECORD OF BOREHOLE No LCS-4 1 of 1 **METRIC**

G.W.P. 5263-05-01 LOCATION Hwy 69(New), Sta. 14+404, o/s 11.9m Lt. of CL Med. ORIGINATED BY N.L.B.
 DIST 54 HWY 69 BOREHOLE TYPE Manual Augering COMPILED BY N.R.
 DATUM Geodetic DATE April 24, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							
						20	40	60	80	100					
202.0	Ground Surface														
0.0	Topsoil														
	End of Borehole														
	Refusal on probable bedrock														
	* Borehole dry														

RECORD OF BOREHOLE No LCS-5 1 of 1 METRIC

G.W.P. 5263-05-01 LOCATION Hwy 69(New), Sta. 14+448, o/s 25.8m Lt. of CL Med. ORIGINATED BY N.L.B.
 DIST 54 HWY 69 BOREHOLE TYPE Portable Rotary Drill and Washboring COMPILED BY N.R.
 DATUM Geodetic DATE April 04, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W		
						20	40	60	80	100						
197.6	Ground Surface															
0.0	Bedrock															
	Migmatite medium grey high strength slightly weathered unweathered poor to good quality		1	RC NQ	REC 97%										RQD 84%	
			2	RC NQ	REC 95%										RQD 30%	
			3	RC NQ	REC 100%										RQD 82%	
194.6	End of borehole															
3.0																
	* Borehole charged with drilling water															

RECORD OF BOREHOLE No LCS-6 1 of 1 METRIC

G.W.P. 5263-05-01 LOCATION Hwy 69(New), Sta. 14+448, o/s 18.8m Lt. of CL Med. ORIGINATED BY N.L.B.
 DIST 54 HWY 69 BOREHOLE TYPE Manual Augering COMPILED BY N.R.
 DATUM Geodetic DATE April 30, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			W _L	Y
						20	40	60	80	100								
195.7	Ground Surface																	
0.0	Topsoil cobbles and boulders																	
195.2	End of borehole																	
0.5	Refusal on probable boulders																	
	* Borehole dry																	

RECORD OF BOREHOLE No LCS-7 1 of 1 METRIC

G.W.P. 5263-05-01 LOCATION Hwy 69(New), Sta. 14+448, o/s 11.8m Lt. of CL Med. ORIGINATED BY N.L.B.
 DIST 54 HWY 69 BOREHOLE TYPE Manual Augering COMPILED BY N.R.
 DATUM Geodetic DATE April 17, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80						100	SHEAR STRENGTH kPa	
											○ UNCONFINED	+	FIELD VANE						
											● QUICK TRIAXIAL	x	LAB VANE						
195.5	Ground Surface																		
0.0	Water																		
0.3	Peat, coarse fibrous boulders Dark brown Wet																		
194.6																			
0.9																			
194.4	Silty sand, with gravel cobbles Grey Wet																		
1.1																			
	End of borehole Refusal on probable boulders																		

* 2007 04 30
 ▽ Water level observed during drilling
 ▼ Water level measured after drilling

RECORD OF BOREHOLE No LCS-8 1 of 1 **METRIC**

G.W.P. 5263-05-01 LOCATION Hwy 69(New), Sta. 14+468, o/s 18.8m Lt. of CL Med. ORIGINATED BY N.L.B.
 DIST 54 HWY 69 BOREHOLE TYPE Manual Augering COMPILED BY N.R.
 DATUM Geodetic DATE April 30, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa				w _p	w		
						○ UNCONFINED	+	FIELD VANE							
						● QUICK TRIAXIAL	x	LAB VANE							
									20	40	60	80	100		
									20	40	60			KN/m ³	GR SA SI CL
201.9	Ground Surface														
0.0	Bedrock at surface														
	* Borehole dry														

+7, x5: Numbers refer to Sensitivity 20 15-5 10 (%) STRAIN AT FAILURE

METRIC

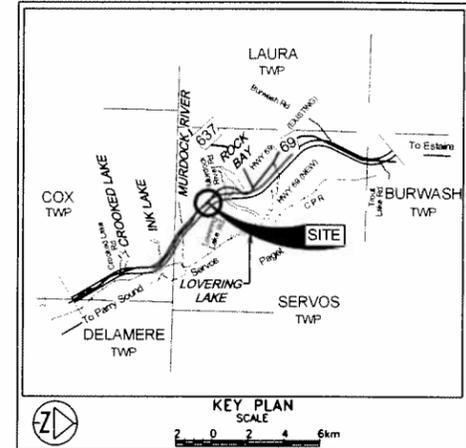
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No
WP No 5263-05-01



LOVERING CREEK BRIDGE SBL
HIGHWAY 69
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



LEGEND

- Borehole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE Blows/0.3m (60° Cone, 475 J / blow)
- W L at time of investigation Mar-Apr 2007
- ▽ Head
- ▽ ARTESIAN WATER Encountered
- ⊥ PIEZOMETER

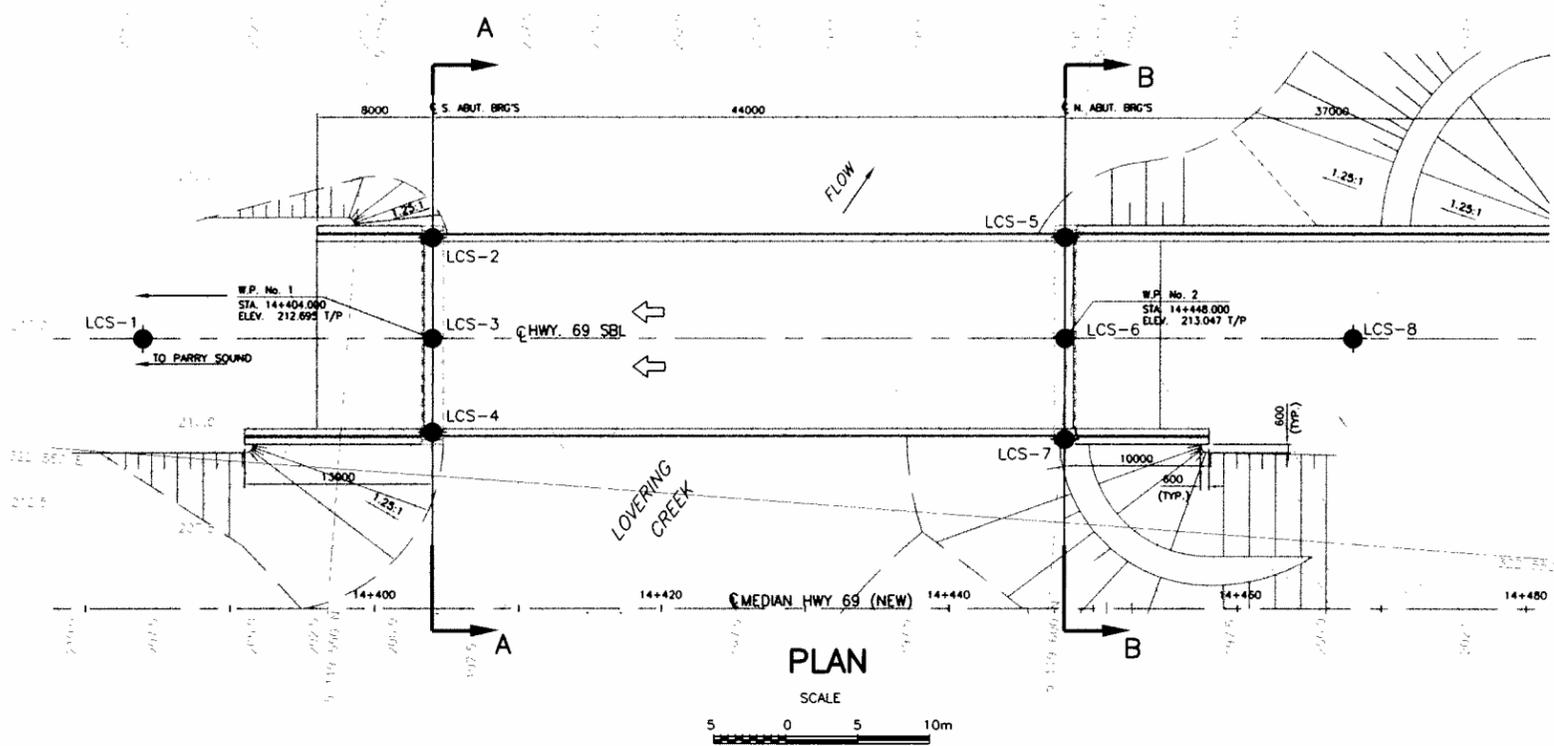
BH No	ELEVATION	STATION	OFFSET CL MEDIAN
LCS-1	210.4	14+384	18.8m Lt.
LCS-2	203.9	14+404	25.8m Lt.
LCS-3	204.7	14+404	18.8m Lt.
LCS-4	202.0	14+404	11.8m Lt.
LCS-5	197.6	14+448	25.8m Lt.
LCS-6	195.7	14+448	18.8m Lt.
LCS-7	195.5	14+448	11.8m Lt.
LCS-8	201.9	14+468	18.8m Lt.

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

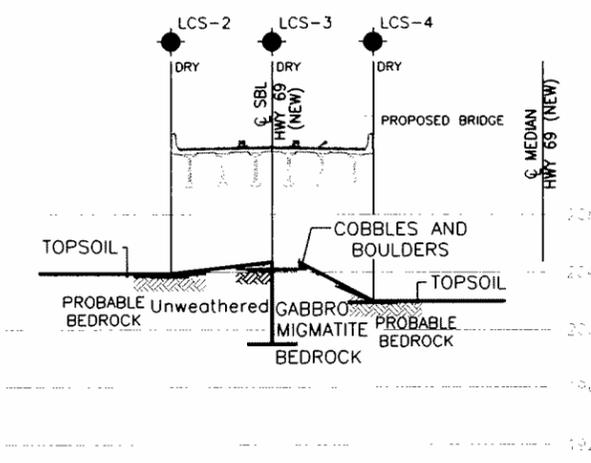
DATE	BY	DESCRIPTION

Geocres No. 421-214

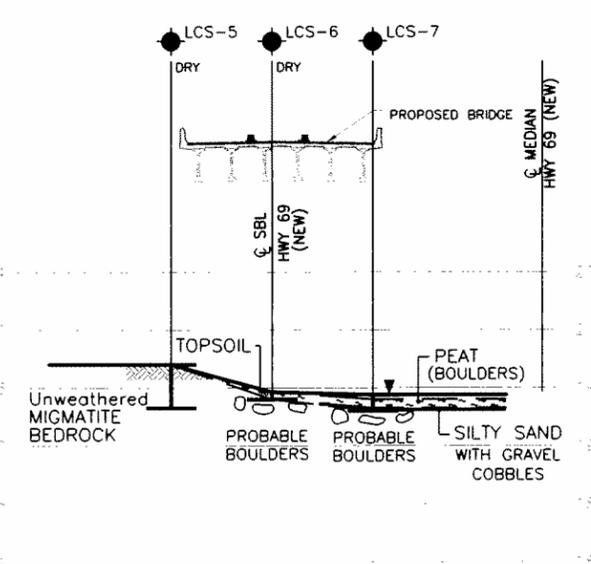
HWY No 69	42-91088-NBL_SBL_LOVERING CREEK.dwg.	DIST SUDBURY
SUBM'D MNR	470_ph_base_nod83_212.dwg.	DATE NOV. 16, 2007
DRAWN NA	Phase1_Contours.dwg	SITE 46-509/2
		DWG LCS-1



PLAN
SCALE
5 0 5 10m

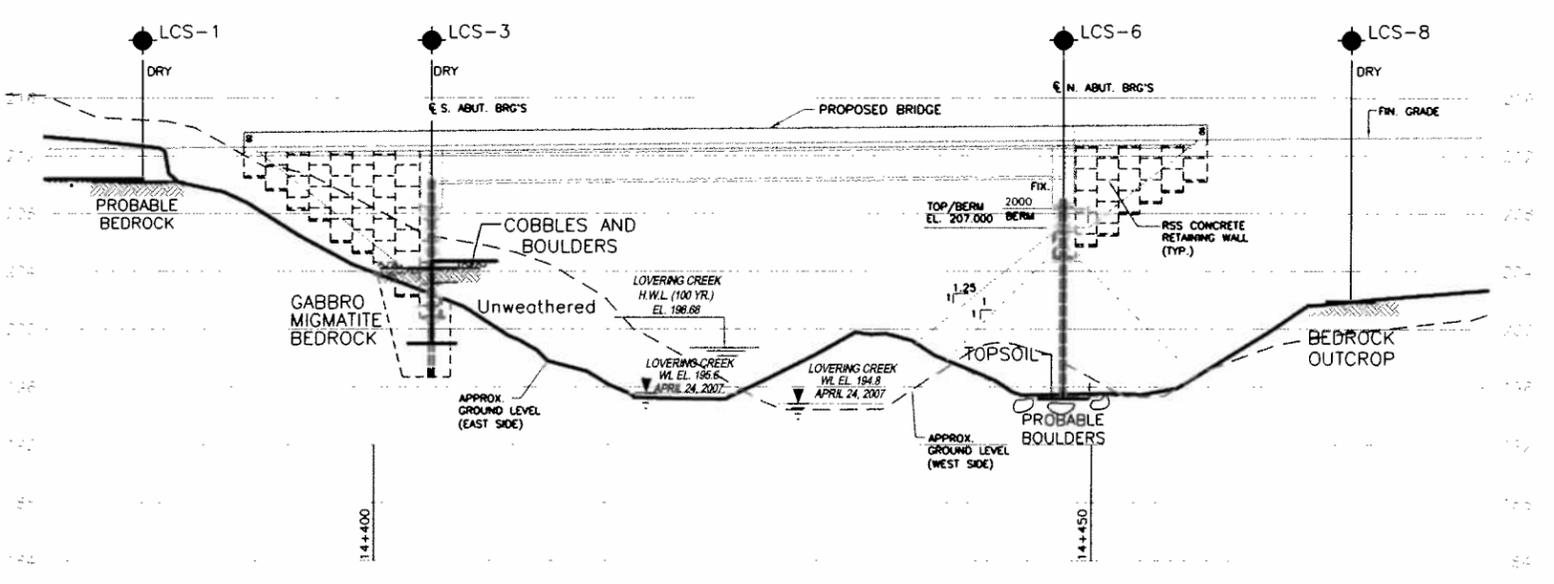


A-A



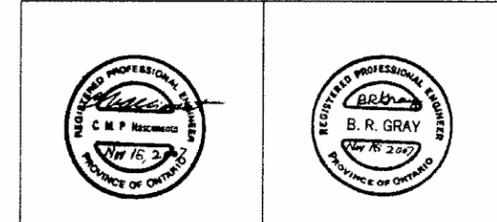
B-B

SCALE
5 0 5 10m



SBL PROFILE

SCALE
5 0 5 10m



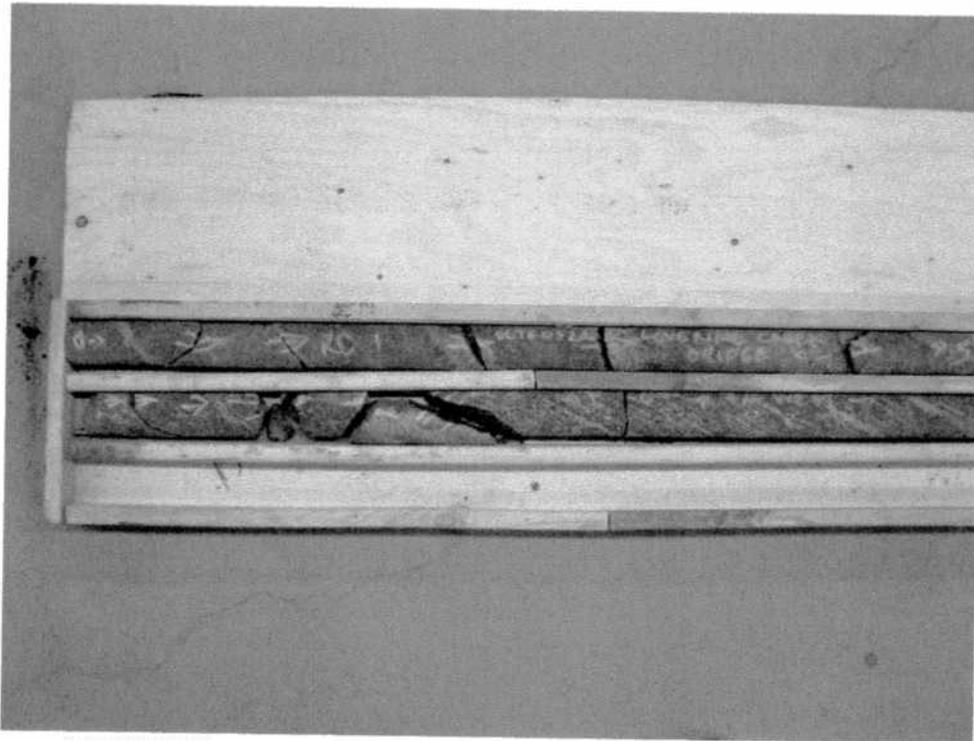
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Phase1_Contours.dwg

NOTES:
1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

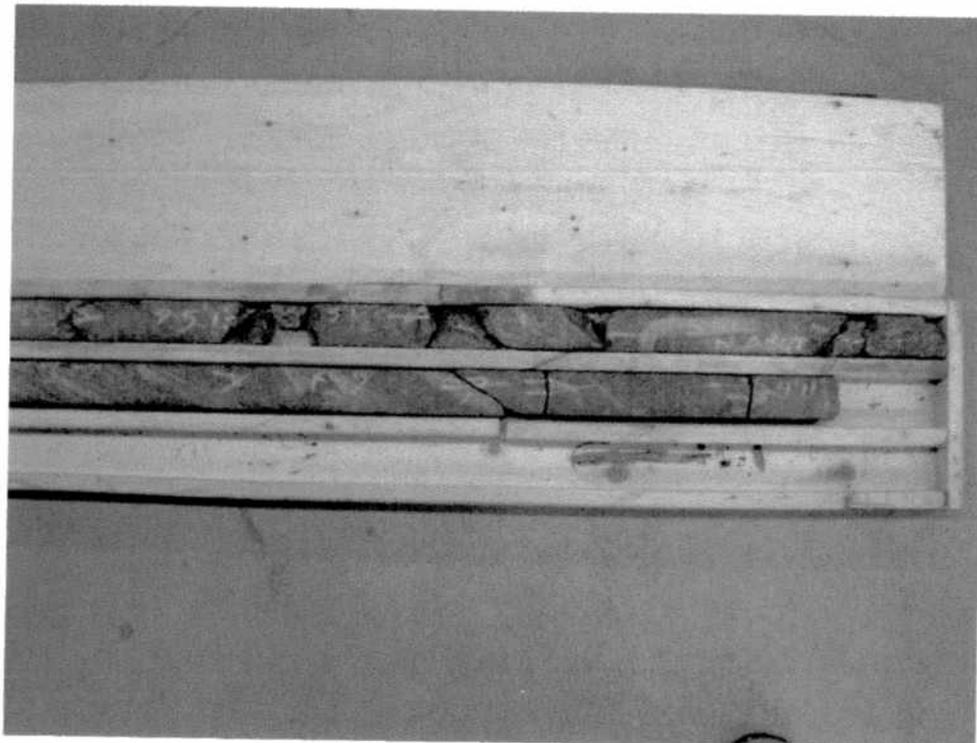


APPENDIX A

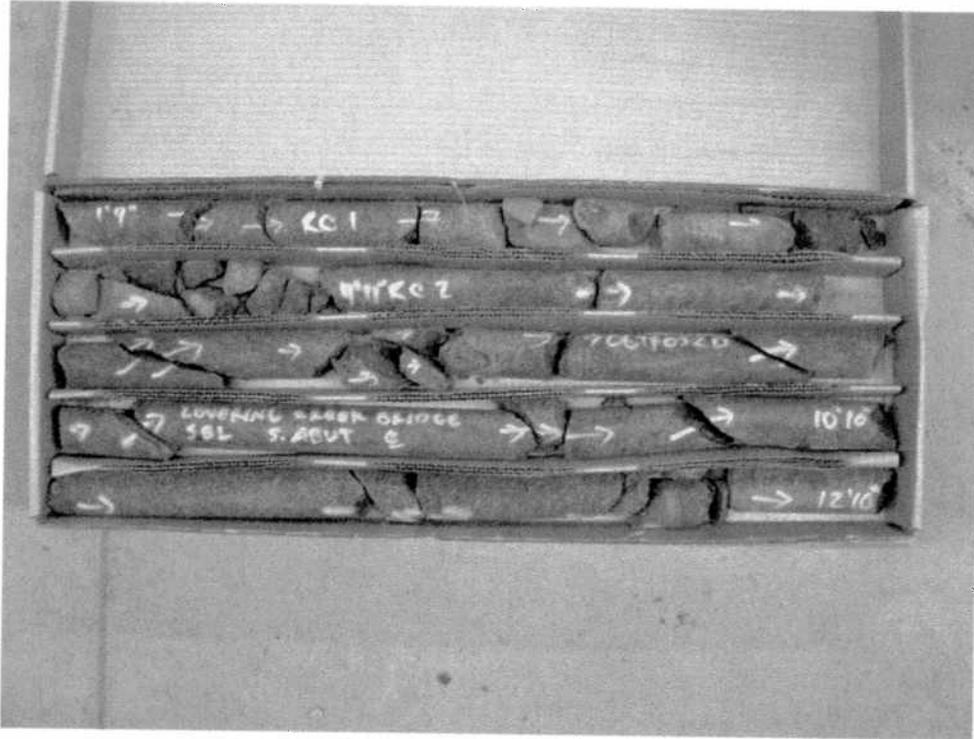
Rock Core Photographs



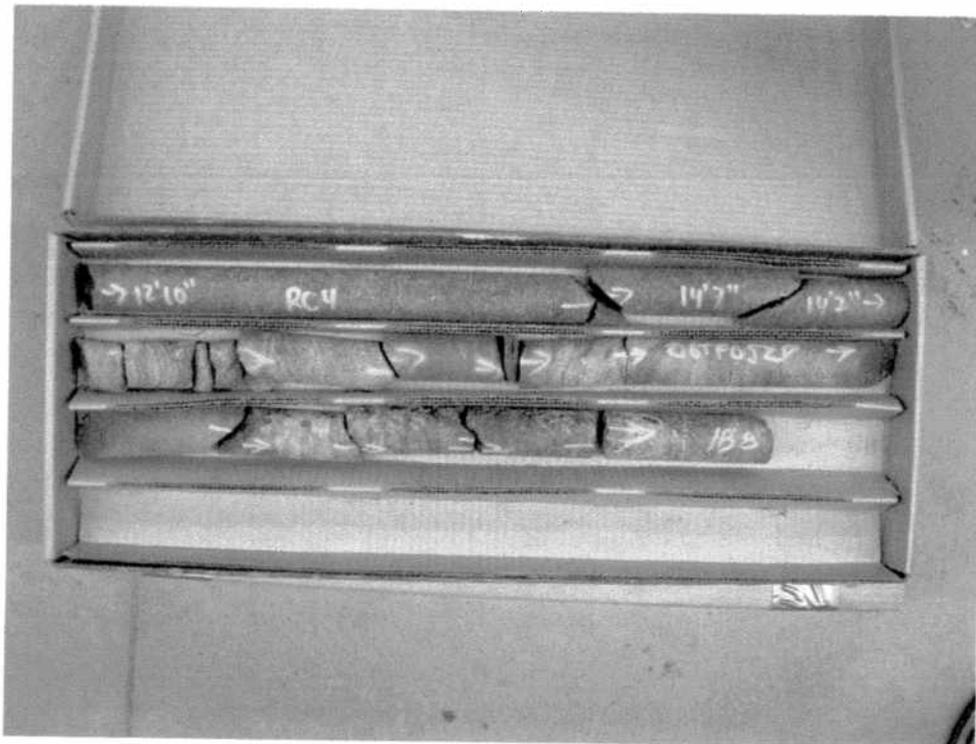
Photograph 1: Rock core from borehole LCS-5



Photograph 2: Rock core from LCS-5



Photograph 3: Rock core from LCS-3 – RC-1 to RC-3



Photograph 4: Rock core from LCS-3 – RC-4 and RC-5

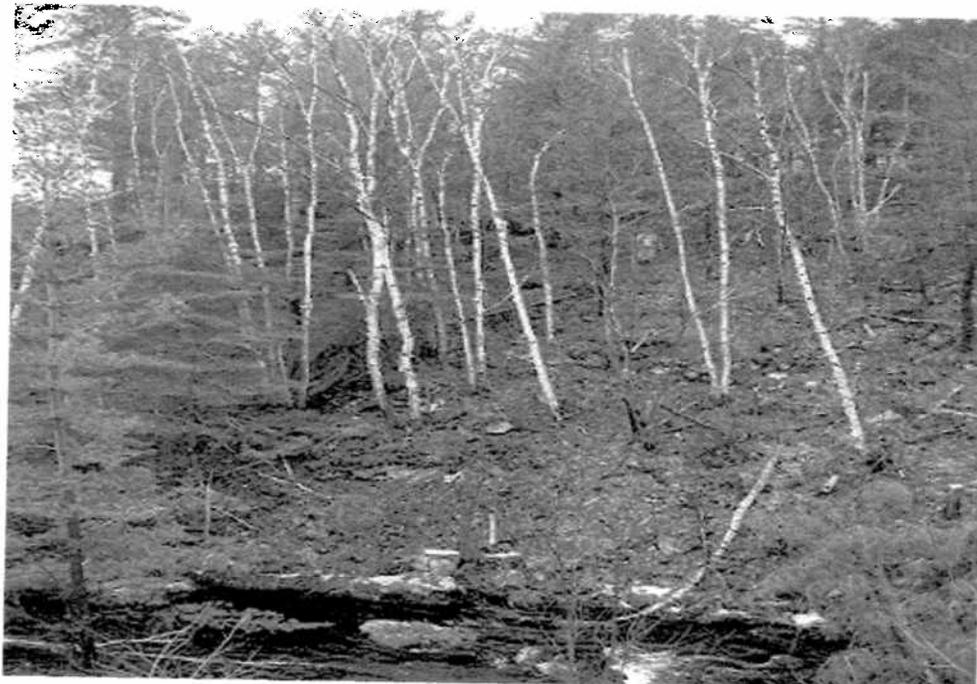


APPENDIX B

Site Photographs



Photograph 1: North abutment of SBL. Note rock exposure and boulders at left of photo (April 2, 2007).



Photograph 2: Looking south from north abutment of SBL. Note steep treed slope with boulders typical of talus slope in front of Lovering Creek (April 2, 2007).



Photograph 3: South abutment of SBL facing east. Note talus slope and boulders in foreground (April 4, 2007)



Photograph 4: North abutment of SBL facing east. Note a wet area at the lower left of the photograph (April 4, 2007).



FOUNDATION DESIGN REPORT

for

**LOVERING CREEK BRIDGE SOUTHBOUND
HIGHWAY 69
SITE NO. 46-509/2, W.P. 5263-05-01
DISTRICT 54, SUDBURY**

***PHASE 1, STA. 12+200 TO 15+400
TOWNSHIP OF SERVOS***

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TORONTO, ONTARIO
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Distribution:

- 3 cc: Totten Sims Hubicki Associates Limited for distribution to MTO, Project Manager + one digital copy of text (WORD format) and Drawings (PDF format)
- 1 cc: Totten Sims Hubicki Associates Limited for distribution to MTO, Pavements and Foundations Section + one digital copy of text (WORD format) and Drawings (PDF format)
- 2 cc: Totten Sims Hubicki Associates Limited
- 1 cc: PML Toronto
- 1 cc: PML Kitchener

PML Ref.: 06TF052D
Index No.: 428FDR
GEOCRES No.: 411-214
November 16, 2007



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Table 1 – List of Ontario Provincial Standard Documents referenced in report

Table 2 – Gradation Specification for Sand Fill in Pre-augered Holes at Integral Abutments

Figure 1 - Abutment on Compacted Fill Showing Granular 'A' Core

FOUNDATION DESIGN REPORT

for
Lovering Creek Bridge Southbound
Highway 69
Site No. 46-509/2, W.P. 5263-05-01
District 54, Sudbury, Ontario

*Phase 1, Sta. 12+200 to 15+400
Township of Servos*

1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations and the approach embankments for the proposed new Lovering Creek Bridge Southbound to be located about 43.7 km south of Sudbury, Ontario. The investigation was conducted for Totten Sims Hubicki Associates Limited on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed bridge will carry the new Highway 69 southbound lanes (SBL) over the existing Lovering Creek between approximate Sta. 14+404.0 and 14+448.0.

The proposed structure consists of a 44.0 m long, single span structure. The approach embankments to the south and north abutments will be raised about 8.1 to 17.4 m above existing grade, respectively at the abutments.

In summary, the subsurface soil stratigraphy revealed in the boreholes generally comprised a non-existent to very shallow soil cover comprising mostly of surficial topsoil over a localized 0.5 m thick talus slope layer of cobbles and boulders (south abutment and approach) and exposed rock outcrops or peat over localized silty sand (north abutment and approach). Numerous cobbles and boulders were encountered in the soil cover.

The depth to the bedrock ranges from 0.1 to 0.5 m below grade at the south abutment and 0.0 to >1.1 m at the north abutment, where a localized water pond was encountered at elevation 195.5, at about the water level in the Lovering Creek.



Based on the encountered subsurface condition and the height of the embankments over the existing ground surface the bridge foundations, it is not considered practical to place the foundations on the native inorganic soils since these layers are very thin (less than 0.5 m) and typically saturated, consequently susceptible to disturbance. The bridge abutments may be founded on spread footing placed on bedrock or engineered fill. Alternatively, deep foundations using driven piles may be used provided that the rock is excavated at the south abutment to allow for an adequate pile length or the piles are driven through a granular fill pad to bedrock at the north abutment. The driven piles are typically used for integral abutments design.

The presence of numerous boulders within the native soils indicates that the installation of drilled cast-in-place concrete caissons will not be practical. The boulders should also be considered when excavating the south abutment area for the installation of piles for the integral abutment solution. The trench through the gentle talus slope deposit in the south abutment should be sloped at 1H:1V in view of potential voids and or loose zones within the deposit.

Blasting of rock in the vicinity of the Lovering Creek should consider the Department of Fisheries of Ontario (DFO) regulations and guidelines for this activity.

The construction of the RSS wall at both the north and south abutments will require that the zone of the approach embankment immediately behind the abutments and below the wall be constructed with structural fill to minimize the postconstruction settlements under the RSS footings.

The "red flag" issues outlined in the preceding paragraph and the recommended methods of overcoming these issues noted in the following sections of the report are intended to alert and aid the designer and the contractor. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the MTO for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable construction quality remain the responsibility of the contractor.



All elevations presented in this report are in meters. A list of the Ontario Provincial Standard documents Referenced in this report is enclosed in Table 1.

2. FOUNDATIONS

2.1 General

The 0.5 m thick talus slope containing numerous boulders at the south abutment may contain loose zones and the 0.5 m thick localized silty sand layer at the north abutment is saturated. These native soils are not considered adequate founding subsoil.

It is considered feasible to support the foundations of the proposed abutments on spread footings constructed on the bedrock underlying both abutments. The bridge may also be founded on spread footings constructed on engineered fill.

The bedrock surface at the boreholes drilled for the abutments of the proposed Highway 69 SBL bridge was found at or within 8.1 and 17.4 m of proposed grade at the bridge deck.

Conventional, semi-integral and integral abutments are considered feasible at this site with some rock excavation at the south abutment to achieve minimum pile lengths, based on the foregoing considerations. The type of foundation employed to support the foundation loads of the proposed structure and the system of bridge design will be dictated by structural considerations, economic considerations and construction constraints. From a foundation engineering perspective, use of integral abutments supported on piles driven to bedrock is the preferred type of foundation abutments.

All footings and/or pile caps subject to frost action should be provided with 2.0 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 0.6 m of soil cover. Footings bearing directly on bedrock do not require protection from frost.



The seismic site coefficient for the conditions at this site is 1.0 (soil profile Type 1, Canadian Highway Bridge Design Code (CHBDC), 2006 Edition, clause 4.4.6).

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The assessment of the liquefaction potential of the silty sand encountered at the site was considered to be not applicable since the materials will be removed for the establishment of the bridge and approach foundations.

2.2 Footings on Bedrock

The reference founding levels for spread footings placed on the bedrock at the south and north abutments are provided on the following table:

FOUNDATION UNIT	SUBGRADE TYPE	REFERENCE ELEVATIONS*	DEPTHS* (m)
South Abutment	Bedrock	201.9 to 204.2	0.1 to 0.5
North Abutment	Bedrock	<194.4 to 197.6	0.0 to >1.1

*Depth from existing ground surface. The minimum 2.0 m frost protection was not considered.

The founding surfaces of the potentially sloping bedrock will have to be levelled off for the placement of the footings or mass concrete placed on the locally sloping bedrock as further recommended in this section of this report.

The following geotechnical resistances should be used for the design of the spread footings:

FOUNDATION UNIT	SUBGRADE TYPE	FACTORED BEARING RESISTANCES AT ULS (kPa)	GEOTECHNICAL RESISTANCES AT SLS (kPa)
South Abutment	Bedrock	8,000	Not Applicable
North Abutment	Bedrock	8,000	Not Applicable



A reduction was applied to the factored bearing resistance at ULS of 10,000 kPa for sound bedrock to account for zones of the bedrock with medium strength and very poor quality. All fractured rock should be removed from the founding surfaces and/or the open joints in the rock filled by pressure grouting (unconfined compressive strength of 35 MPa).

Considering the bedrock to be non-yielding, the design will not be governed by settlement criteria since the loading required to produce 25 mm deformation is much larger than the factored resistance at ULS. The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The minimum footing width and the groundwater level were not considered applicable for the ULS assessment for spread footings founded on the bedrock.

Construction of the footings should be performed and monitored in accordance with SP 902S01 to verify the competency of the founding surface.

Mass concrete could be placed to provide a level founding surface for the footings and/or be employed to raise the subgrade to the design founding level of the footings. The need to expand the plan area at the base of the mass concrete to provide for stress distribution (2V:1H), place reinforcing steel in the mass concrete and/or use high strength concrete to prevent overstressing of the mass concrete will be dictated by the actual thickness of the mass concrete and structural design considerations.

Subject to these comments, the bearing resistance provided for footings bearing on bedrock is considered to be appropriate for mass concrete with an unconfined compressive strength of at least 30 MPa. If the actual bearing pressure is less than 8,000 kPa, the compressive strength of the concrete could be reduced in direct linear proportion to the actual bearing pressure (minimum value of 25 MPa).

Comments concerning excavation of the bedrock, if required, to found the footings at a level lower than indicated previously, are provided in subsequent sections of the report.



The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the founding bedrock.

An unfactored friction factor of 0.7 is considered to be suitable at this site on the "rough bedrock surfaces" (asperity height of at least 25 mm). The factored horizontal resistance at ULS of the bedrock is considered to be 5,000 kPa.

The lateral resistance of footings founded on bedrock could be increased, if required, by installing shear keys, sockets or anchors into the bedrock (SP 999S26). The increased lateral resistance will be provided by the shear strength of the steel dowels, the horizontal resistance of the bedrock and the horizontal component of tensile forces developed in any inclined anchors. A greater frictional resistance between the footing and rock may be achieved if the anchors are prestressed to increase the vertical pressure.

If dowels are employed, a NSSP should be included in the tender documents to provide specific direction for the contractor during installation and testing of the dowels. Fractured rock should be removed from these areas. A NSSP should also be prepared for the shear keys.

Design, installation and testing of the anchors should be conducted in accordance with SP 999S26 and clause 6.10.4 (CHBDC). If anchors are installed, a factored bond stress at the rock/grout interface of 1.4 MPa at ULS (a resistance factor of 0.4 is applied for a minimum 35 MPa grout) is recommended for design. The total capacity of a group of closely spaced anchors may be less than the summed capacities of the individual anchors; the impact of anchor interaction should be assessed if the spacing is less than one-fifth of the anchor length.

2.3 Footings on Structural Fill

Construction of the abutment footings on structural fill placed in the approach embankment could also be employed to support the foundation loads. The structural fill should comprise Ontario Provincial Standards Specifications (OPSS) Granular A material placed in maximum 200 mm thick layers, compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density. Since the existing inorganic native soil comprises of a bouldery talus slope (south abutment) or



saturated silty sand (north abutment), the native soils should be removed and the structural fill should extend to the bedrock surface and out to a plane inclined downwards at 45° to the horizontal originating at least 1 m away from the top of the footing. This scheme is illustrated in the appended Figure 1. The extent of the fill should be established by a site specific survey prior to placement of the fill.

Footings should not be constructed on rock fill. However, rock fill may be placed adjacent to the Granular A core shown in Figure 1. It should be ensured that the base of the structure fill is well drained and the rock surface does not contain undrained depressions which may retain groundwater under the fill. To this end, a NSSP should be added to the contract.

The recommended bearing resistance for a 2.5 m wide footing constructed on structural fill (bearing resistance independent of fill thickness at this location because the engineered fill should be placed directly on the bedrock) is as follows:

Factored Bearing Resistance at ULS	900 kPa
Bearing Resistance at SLS	350 kPa

The resistance at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 2.0 m was assumed for computation of the ULS resistance.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the structural fill. An unfactored friction factor of 0.7 is recommended for footings on the granular structural fill.



2.4 Pile Foundation

It is considered feasible to found the north and south abutments on piles driven through a granular fill pad placed on the underlying bedrock. The existing organic deposits and 0.5 m thick saturated sandy silt and bouldery talus slope should be excavated before placing the granular fill pad for ease of constructability. In addition, to found the south abutment on driven piles a trench should be excavated/blasted into the existing rock outcrop where the free pile length will not be structurally adequate. This bedrock removal will be required to provide a minimum of 5 m of free pile length (below the abutment stem) to allow an integral abutment design for the bridge.

The general pile foundation design recommendations are provided on the following paragraphs followed by additional recommendations for integral abutment foundations.

The estimated reference founding levels for piles for the north abutment and the range of highest levels of the rock surface at the south abutment where piles should be driven to refusal on bedrock are provided on the following table:

LOCATION	DEPTH TO ROCK* (m)	PILE FOUNDING ELEVATION	RELEVANT BOREHOLES
South Abutment	0.1 to 0.5	201.9 to 204.2	LCS-2, LCS-3 and LCS-4
North Abutment	0.0 to >1.1	< 194.4 to 197.6	LCS-5, LCS-6 and LCS-7

* Note: A +1.0 m variation of the average depth to rock should be allowed for construction estimation purposes.

At the south abutment, the actual founding levels are dependent on the structural design of the abutment stem.



The recommended factored axial resistance at ultimate limit states (ULS) for the pile sections listed below is considered to be appropriate.

FACTORED AXIAL RESISTANCE AT ULS, kN

HP 310 x 110	2000
HP 310 x 152	2800
HP 360 x 108	2000
HP 360 x 152	2800

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the bedrock to be non-yielding and the relatively short pile length required, the design is not expected to be governed by settlement since the required loads causing appreciable deformation of the pile and/or bedrock are much larger than the ULS factored resistance.

It is considered that negligible down drag force will develop on the piles due to negative skin friction since the embankments will be placed on unyielding bedrock.

Refer to Section 5 for a discussion and recommendations on the treatment of approach embankment settlements.

The presence of numerous cobbles and boulders was identified above bedrock at depth in boreholes LCS-3 and LCS-7. The risk of damage during driving is considered to be high and the boulders should be removed from the intended pile foundation before driving the piles as indicated previously. The excavation should be backfilled with granular materials. Since the fill pad will be incorporated in the structural fill for the RSS walls (refer to Section 4) the granular materials should be compacted to 100% of the standard Proctor maximum dry density. Nevertheless, a NSSP should be prepared to advise the contractor of the presence of boulders at this site. The NSSP is required to ensure that more comprehensive engineering supervision is required than is called for in SP 903S01.



The compacted granular fill pad placed as a working platform for construction equipment during installation of the abutment piles should comprise OPSS Granular A material to allow installation of the piles without damage. Alternative granular materials could be employed provided the maximum particle size does not exceed 75 mm.

The piles will be driven through 4 to 7 m of the compacted granular fill pad. It is considered, based on PML extensive experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed. Since the piles will be driven to bedrock, a specific set is not provided.

The piles will set on or into bedrock and should be equipped with "Rock Points" according to SP 903S01. The Titus H Bearing Pile Points Rock Injector Model (Titus Point) should be used at this site since the slope of the bedrock revealed at the borehole locations ranges from 4 to 19° (south abutment) and the rock surface is covered with cobbles and boulders.

2.5 Integral Abutments on Piles

The bedrock surface level at the south abutment is at elevations 201.9 to 204.2 that is 8.5 to 10.8 m below the proposed top of pavement elevation 212.7.

The need and depth of excavation of a trench into rock to accommodate a free pile length of 5 m for the use of integral abutments will be dictated by structural design details. The excavation width should be at least 1 m wider than the plan area of the piles; side slopes in the soil cover and in the rock should be excavated as indicated in Section 5 of this report. The excavation should be backfilled with Granular A, following the procedures outlined in the section titled "Approach Embankments". Further comments concerning bedrock excavation are provided in the section titled "Excavation and Groundwater Control".



To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

2.6 Lateral Resistance

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile below the annular space referred to previously. The assessed horizontal passive resistance values for the pile sections noted previously is as follows:

Pile Section	GRANULAR BACKFILL	
	HP310	HP360
Factored Lateral Resistance at ULS, kN	120	170
Lateral Resistance at SLS, kN	50	70

The assessed values of lateral resistance assume that the piles are driven through compacted granular materials placed as recommended previously. If greater resistance is required, batter piles should be installed.



To evaluate the point of contraflexure, the coefficient of horizontal subgrade reaction, k_s (MN/m^3) should be computed using the following equation:

a) Cohesionless Soils (Terzaghi, 1955)

$$k_s = n_h z/b$$

where n_h = coefficient related to soil density
 = $10.0 \text{ MN}/\text{m}^3$ for granular backfill
 z = depth, m
 b = pile width, m

The cohesionless soil parameter is applicable to all the granular fill materials to be provided along the piles.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters/widths. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R , as follows:

PILE SPACING IN DIRECTION OF LOADING d = PILE DIAMETER OR WIDTH	SUBGRADE REACTION REDUCTION FACTOR, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

2.7 Comparison of Foundation Alternatives

Caisson foundations were not considered to be feasible due to the presence of numerous boulders above the bedrock. A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives discussed in the preceding paragraphs is presented below.



Footings on Bedrock

Advantages

- Ease of installation
- Lower cost than deep foundations
- May be used with semi-integral abutments

Disadvantages

- Requires removal of all boulders from foundation footprint (south abutment)
- Possible need to remove fractured rock
- Possible need for mass concrete and/or bedrock excavation to provide level founding surfaces

Footings on Structural Fill

Advantages

- Ease of installation
- Lower cost than deep foundations
- Reduced height of abutment
- Fractured rock need not be excavated
- May be used with semi-integral abutments

Disadvantages

- Construction of structural fill pad requires wider area than footings on bedrock
- Requires construction of an engineered fill pad

Piles to Bedrock

Advantages

- High bearing resistance
- Used for integral abutments

Disadvantages

- Requires construction of a fill pad ahead of the approach embankment construction
- Higher installation cost than spread footings
- Requires excavation of trench in rock where free pile length is less than 5.0 m (for integral abutment design)



3. ABUTMENT WALLS

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure, p (kPa), may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation:

$$p = K (\gamma h + q) + C_p + C_s$$

where K = coefficient of lateral earth pressure (dimensionless)

γ = unit weight of free-draining granular material, kN/m^3

h = depth below final grade, m

q = surcharge load, kPa, if present

C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)

C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)

where ϕ = angle of internal friction of retained soil (35° for Granular B Type II)

δ = angle of friction between the soil and wall (23.5° for Granular B Type II)

The seismic site coefficient for the conditions at this site was provided in Section 2.1.

Hydrostatic pressures were not included in the equation since free-draining OPSS granular material or rockfill will be used as backfill behind the wall. The following parameters are recommended for design:

PARAMETER	GRANULAR A, GRANULAR B TYPE II or TYPE III	ROCKFILL
Angle of Internal Friction, degrees	35	42
Unit weight, kN/m^3	22.8	18.0
Coefficient of Active Earth Pressure, K_a	0.27	0.20
Coefficient of Earth Pressure At Rest, K_o	0.43	0.33
Coefficient Passive Earth Pressure, K_p	3.69	5.04

Refer to MTO Report SO-96-01 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments. The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained



structures. The earth pressure coefficients should be reviewed if the slope of the backfill exceeds 10° to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load (q in the preceding equation).

The magnitude of the passive resistance and active pressure is dependent on the actual lateral movement of the structure toward and away from the adjacent soil, respectively. We refer to Figure C6.16 (Clause C6.9.1) of the CHBDC for these computations. The backfill should be considered as medium dense sand for this project.

A weeping tile system (SP 405F03, OPSD-3102.100 and OPSD-3190.100) and/or weep holes should be installed to minimise the build-up of hydrostatic pressure behind walls. The weeping tiles should be surrounded by a properly designed granular filter or non-woven Class II geotextile (with an FOS of 75-150 μm according to OPSS 1860) placed to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

Backfilling adjacent to retaining structures should be carried out in conformance with OPSD-3101.150 and 3101.200 for granular or rock backfill at abutments.

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to SP 105S10 for additional information in this regard.

4. RETAINED SOIL SYSTEM WALL

A retained soil system (RSS) could also be employed at the abutments provided the estimated settlements noted in Section 5 Approach Embankments Settlements are accommodated. A high performance, high appearance rated RSS wall should be employed. The design, supply and construction of the RSS wall should conform to SP 599S22.

The topsoil and peat encountered at south and north abutments are highly compressible and not considered adequate to support the RSS wall footings and should be removed from the RSS wall footprint together with the bouldery talus slope material and saturated silty sand.



The RSS walls at the south abutment will be placed on the bedrock or on a thin structural fill layer over the bedrock at various levels ranging from elevations 204.5 to 210.0. The RSS wall footings for the north abutment should be placed on a structural fill pad at varying levels also ranging from elevations 204.5 to 210.0. The structural fill should be compacted to 100% of the standard Proctor maximum dry density. The structural fill pad should be founded at the level of the bedrock encountered at about elevations 194.6 to 201.9. The recommended bearing resistance for the RSS wall footings (maximum 0.6 m wide) constructed as recommended above may be taken as recommended previously for the bridge footings.

A transition treatment between the RSS wall fill and the rockfill should be provided to minimize the movement of granular fill into the voids of the rockfill. For this purpose, the surface of the rockfill at the interface should be chinked or a geotextile should be provided between the two types of fill. The same geotextile used for subdrains (Section 3) is considered adequate.

The earth pressure coefficients provided previously are considered to be appropriate for the RSS wall. The horizontal force at the base of the RSS will be resisted in part by the friction force developed through the granular backfill or along the interface between the granular backfill and the founding soil, subject to site specific design details. An unfactored friction factor of 0.7 is considered to be appropriate for both situations at this site.

The RSS supplier should be responsible for specifying the type of backfill material employed, taking into consideration the engineering properties of the proprietary product, the design life of the structure, the pullout resistance required and drainage requirements. The RSS wall designer should note that the MTO Northeastern Region requires that all fill to the structures comprises OPSS Granular B Type II for rockfill embankments. The RSS wall should be designed to withstand the estimated settlements of the native soils under the embankment loads indicated in the Approach Embankments section.

The supplier of the RSS should also be responsible for the design of the structure (reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations, performance level, appearance, etc.



5. APPROACH EMBANKMENTS

The existing topsoil, peat and thin talus slope and saturated sandy silt soils within the backfill zone of the abutments and RSS walls should be excavated prior to placement of the backfill.

The level of the approach embankments will be raised about 8.1 m (south) and 17.4 m (north) above the existing surface. It is anticipated that the new embankments will be constructed with rock fill. The field investigation indicates that the existing native soils are non-existent or very shallow and generally comprise heterogeneous mixture of topsoil with localized deposit of silt with sand and variable amounts of gravel, cobbles and boulders. The existing soils, 0.1 to 1.1 m thick, cover bedrock at shallow depths. Construction of the embankment fill remote from the abutments on the existing soils is considered to be feasible.

The embankments should be constructed in accordance with OPSD-201.020, 202.010, 208.010 and SP 206S03. The side slopes of approach embankments should be inclined no steeper than 2 horizontal to 1 vertical (2H : 1V) for earth fill and 1.25 H : 1 V for rockfill. Where the height of the embankment is greater than 8 or 10 m for earth or rock fill embankments, respectively, a 2 m wide mid-height berm will be required.

Where slope flattening is proposed, a drainage gap should be provided in accordance with OPSD-202.020. Where slopes are flattened to eliminate the need for a guide rail, a granular infilled drainage gap should be provided. OPSS Granular B Type II or Type III should be used for the drainage gap.

Although the new embankments will be constructed on bedrock the platform width should be widened by a minimum of 2 m each side in accordance with the Northeastern Region Engineering Directive (NRE 98-200).

It is considered that the approach embankments constructed in accordance with these recommendations will be stable. Settlement of the embankment fill due to consolidation of the underlying bedrock will be negligible.



Settlement of the road surface within the approach embankments during and following completion of construction will result only from the “self weight” consolidation of the embankment fill since the bedrock is considered to be unyielding.

The settlement of new rock fill should be in the order of 40 to 85 mm if constructed in accordance with the requirements of SP 902S01 and OPSS 501 (Method A). It is estimated that about 50% of the total settlement occurs during the first year following the placement of the rockfill and the remaining 50% will be developed at a progressively decreasing rate during the following 5 to 10 year period.

Earth fill slopes should be protected against surface erosion by sodding (OPSS 571) and suitable vegetation. Refer to OPSS 572 for time constraints and type of seed and mulch required.

The face of the approach embankments should be protected with rip rap or equivalent materials to the level of the high water in the creek (SP 511S01).

6. EXCAVATION AND GROUNDWATER CONTROL

Excavation for construction of footings founded on bedrock will extend through about 0.1 to 0.5 m and 0.0 m to greater than 1.1 m thick in-situ soil cover at the south and north abutments, respectively. Cobbles and boulders should be expected in the excavations.

The talus slope containing boulders at the south abutment and the compact silty sand are classified as Type III soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Therefore, temporary cut slopes inclined at 45° to the horizontal should be used.



A large excavator equipped with a tiger-toothed bucket in conjunction with a jackhammer or hoe ram is the preferred method of excavation to shallow depths in rock scaling. Conventional rock excavation techniques such as blasting (OPSS 120) may also be required. The actual equipment required and method of excavation within the bedrock will be dependent upon the geometry of the cut and relative depth of excavation into the bedrock. Mass concrete could be employed to level minor variations in the bedrock surface, as mentioned previously.

It is important that blasting of the rock is controlled to prevent fracturing and/or disturbance of the bedrock surface on which footings will be founded according to SP 299F06. Any overblasting/overexcavation should be made the sole responsibility of the contractor and all loosened rock resulting from blasting operations is to be removed by mechanical means.

In addition, Department of Fisheries Guideline for the use of explosives in or near Canadian Fisheries water (Canadian Technical report of Fisheries and Aquatic Sciences 2107, dated 1998) should be followed when conventional rock excavation techniques such as blasting are required near the Lovering Creek to protect the water environment and fish habitat.

Near vertical sidewalls may be utilized in excavations in bedrock. Examination of the sidewalls and removal of any loosened rock fragments should be carried out continually for the safety of workers.

Open surface water was encountered in borehole LCS-7 at the north abutment. It is considered that this water is locally perched in a rock surface depression connected to the creek. This water will have to be locally removed by the dam and pump method. In the remaining boreholes at the south and north abutment and approach locations, groundwater was not encountered. Subject to the groundwater level at the time of construction, it is considered feasible to employ sump pumps to control groundwater seepage into the excavations for construction of the south abutment footings and the RSS wall foundations.

Surface water run-off should be diverted away from excavation to ensure that the foundations are constructed in the dry.



All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

7. **CLOSURE**

The report was prepared by Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer. It was reviewed by Mr. B.R. Gray, MEng., P.Eng., MTO Designated Principal Contact carried out an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

A handwritten signature in black ink, appearing to read "Carlos M. P. Nascimento", written in a cursive style.



Carlos M. P. Nascimento, P. Eng.
Senior Project Engineer

A handwritten signature in black ink, appearing to read "Brian R. Gray", written in a cursive style.



Brian R. Gray, MEng., P.Eng.
MTO Designated Principal Contact

CN/DWK/BRG/cn:lnr



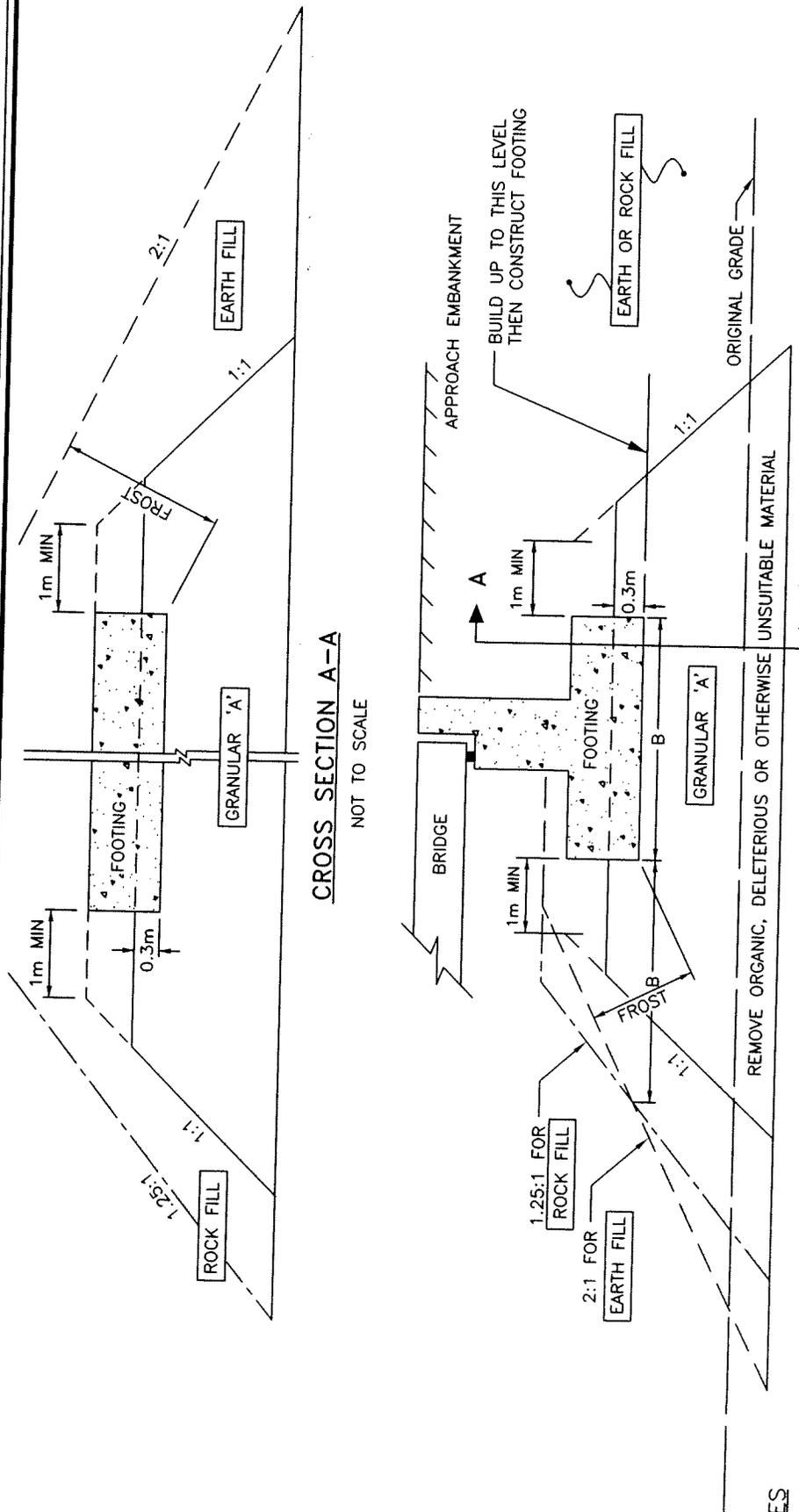
TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE	DATE
OPSS 120	General Specification for the Use of Explosives	November 2003
OPSS 501	Construction Specification for Compacting	November 2005
OPSS 571	Construction Specification for Sodding	November 2001
OPSS 572	Construction Specification for Seed and Cover	November 2003
OPSS 1860	Material Specification for Geotextiles	November 2004
SP 105S10	Construction Specification for Compaction	November 2004
SP 206S03	Construction Specification for Grading	November 2006
SP 299F06	Rock Excavation (Controlled Blasting)	December 2005
SP 405F03	Construction Specification for Pipe Subdrains	November 2006
SP 511S01	Placement of Rip-Rap, Rock Protection and Gravel Sheeting	January 2001
SP 599S22	Requirements for The Design, Supply and Construction of Retaining Soil Systems (RSS)	March 2001
SP 902S01	Excavation and Backfilling of Structures	June 2006
SP 903S01	Construction Specification for Piling	November 2006
SP 999S26	Requirements for Design, Installation and Testing of Temporary and Permanent Pre-Stressed Anchors in Soil and Rock	November 2006
OPSD-201.020	Rock Grading-Divided Rural	November 2005
OPSD-202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment	November 2005
OPSD-202.020	Drainage Gap for Slope Flattening on Rock or Granular Embankment	November 2005
OPSD-208.010	Benching of Earth Slopes	November 2003
OPSD-3101.150	Minimum Granular Backfill Requirements - Abutments	November 2005
OPSD-3101.200	Rock Backfill Requirements - Abutments	November 2005
OPSD-3102.100	Wall Abutment Backfill Drain	November 2005
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail	November 2005
NRE 98-200	Northeastern Region Directive - Platform Widening	October 28, 1998
NSSP	Dowels Into Concrete	December 2002
NSSP	Shear Keys	N/A
NSSP	Presence of Boulders	N/A
NSSP	Drainage of Base of Structural Fill Placed on Bedrock	N/A



TABLE 2
GRADATION SPECIFICATION FOR SAND FILL IN
PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS

MTO SIEVE DESIGNATION		PERCENTAGE PASSING BY MASS
2 mm	#10	100
600 µm	#30	80 – 100
425 µm	#40	40 – 80
250 µm	#60	5 – 25
150 µm	#100	0 – 6



CROSS SECTION A-A
NOT TO SCALE

LONGITUDINAL SECTION
NOT TO SCALE

NOTES

1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE