Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION AND DESIGN REPORT
for

LOVERING CREEK BRIDGE NORTHBOUND
HIGHWAY 69
SITE NO. 46-509/1, W.P. 5262-05-01
DISTRICT 54, SUDBURY, ONTARIO

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

PETO MacCALLUM LTD.
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PML Ref.: 06TF052C
Index No.: 424FIR and 425FDR
GEOCRES No.: 41I-215
November 16, 2007

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for
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District 54, Sudbury, Ontario

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

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1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed Lovering Creek Bridge Northbound on the realigned Highway 69 to be located 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 bridge will carry the new Highway 69 northbound lanes (NBL) between approximate Sta. 14+383.0 and 14+427.0.

This report provides subsurface information pertaining to the foundation of the proposed new Highway 69 northbound overpass and approaches within about 20 m of the abutments, between approximate Sta. 14+363.0 and 14+447.0.

2. SITE DESCRIPTION AND GEOLOGY

The site is located approximately 43.7 km south of Sudbury and to about 0.5 km 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 areas 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.

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BARRIE, HAMILTON, KITCHENER, TORONTO

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 NBL bridge was carried out during the period from March 27 to April 24, 2007.

The scope of the subsurface investigation comprised 8 boreholes that were advanced through the soil cover to depths of 0.0 to 1.8 m at the locations shown on Drawing LCN-1, appended. Four of the boreholes, LCN-3 and LCN-5 to LCN-7, were cored 2.8 to 3.1 m into the bedrock to depths between 2.8 to 4.1 m.

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	200.5
TBM2	Existing ground at South abutment WP	205.6

(*) Geodetic, metric

The boreholes were advanced by using a portable drilling equipment, as required by the very restrictive site accessibility and prevalent weather limitations. Boreholes LCN-1, LCN-2, and LCN-8 were advanced using manual augering methods. The remaining boreholes, LCN-3 to LCN-7 were advanced using manually operated electric drill (HILTI) equipped for NQ diamond rock coring and washboring, 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 rock cores 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 by visual examination of the soil, the sampler and drill rods as the samples were retrieved 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. Due to the practically non-existent or very shallow soil covers over bedrock/probable bedrock surface insufficient quantities of 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 included in Appendix B.

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

The depth of the soil cover revealed in the boreholes varies from 0.0 (at exposed rock outcrops) to 1.8 m. The soil cover at the borehole locations generally comprises topsoil over localized silt (north approach borehole LCN-8) mantling boulders. Scattered layers of cobbles and boulders were encountered at the surface and in boreholes LCN-3 and LCN-4, which were drilled through a shallow talus slope deposit.

4.1 Topsoil

A surficial layer of topsoil is present in boreholes LCN-1 to LCN-4 and LCN-8 and extends to 0.1 to 0.6 m depths from ground surface, elevations 195.6 to 211.3. In boreholes LCN-3 and LCN-4, the topsoil contained cobbles and boulders.

The topsoil in boreholes LCN-1 and LCN-2 extended to the probable bedrock of the rock outcrop located south of the creek. These boreholes were terminated below the topsoil at elevations 208.7 and 211.3, respectively.

4.2 Silt

A localized deposit of cohesionless brown/grey silt content is present below the topsoil in borehole LCN-8, and extends to 0.8 m depth, elevation 194.9, where the borehole was terminated on probable boulders. The silt contains gravel, sand and trace clay.

4.3 Cobbles and Boulders

A localized deposit of cohesionless cobbles and boulders mixed with sand and silt was encountered below the topsoil in boreholes LCN-3 and LCN-4 and extends to 1.0 and 1.8 m depths, respectively, elevations 204.6 and 200.1. The layer was judged to comprise a shallow talus slope at the toe of the rock outcrop south of the creek.

4.4 Bedrock

A detailed description of the rock cores retrieved from boreholes LCN-3 and LCN-5 to LCN-7 is provided in Table A and summarized on the record of borehole logs. A large bedrock outcrop was encountered surficially at the location of boreholes LCN-5 to LCN-7. The bedrock comprises light grey to black migmatite in boreholes LCN-3, LCN-6 and LCN-7. In borehole LCN-5, 0.8 m thick light grey to pink migmatite band was encountered over pink and white pegmatite. The rock is typically unweathered and exhibits high strength.

At the north abutment boreholes, LCN-5 to LCN-7, the bedrock was confirmed by drilling three core holes 2.8 to 3.1 m. The rock surface elevations 198.6 to 200.5, indicating maximum bedrock surface relief of 1.9 m between borehole locations. The slope of the bedrock surface between boreholes is less than 20°. Photographs of the rock cores taken from boreholes LCN-5 to LCN-7 are shown on Plates 1 to 6, Appendix A.

At the south abutment boreholes, LCN-2 and LCN-3, the bedrock was confirmed by drilling one core hole a minimum of 3 m or inferred by refusal at depths of 0.3 to 1.0 m, elevations 204.6 to 208.7.

Rock coring at the originally planned borehole LCN-4 was terminated on boulders and fractured rock at 1.8 m depth, elevation 200.1, due to limitation of the portable equipment. A photograph of the rock core taken at borehole LCN-3 is shown on Photo 7, Appendix A.

In the north and south abutments boreholes, the measured core recovery is 100%, with one isolated value of 98% in borehole LCN-3. The RQD determined from the north abutment rock cores, LCN-5, LCN-6 and LCN-7, is typically greater than 90%, with one isolated value of 72 % in borehole LCN-7, indicating excellent quality rock with local fair quality rock. The range of RQD for borehole LCN-3 is between 0 and 53%, indicating very poor to fair rock quality.

4.5 Groundwater

No groundwater was observed in the boreholes, LCN-1 to LCN-8, during and upon completion of drilling. The water level in the Lovering Creek at the time of the investigation was about elevation 196.0.

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, carried out an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

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

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

CN/BRG/nr-lnr

TABLE A

ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
LCN-3	1	1.0 – 1.5	100	0	1.0 – 4.0	MIGMATITE: Light grey to light green and pink, fine crystalline, with minor garnet, slight banding, occ. pegmatite layer, coarse crystalline, becoming mainly dark green to black biotite below 3 m depth, with yellow to rust coloured oxidation on partings, high strength, slightly weathered to unweathered, very close to close spaced flat to dipping partings, rough planar, tight to oxidized, also vertical parting, rough planar to undulating, very poor to fair quality.
	2	1.5 – 2.0	100	0		
	3	2.0 – 3.0	100	53		
	4	3.0 – 4.0	98	9		
LCN-5	1	0.0 – 0.8	100	93	0.0 – 0.8	MIGMATITE: Light grey to pink, medium crystalline, with black bands and/or inclusions (biotite), coarse crystalline, with 200 mm thick garnetiferous layers, high strength, unweathered, moderate spaced flat partings, rough planar, tight, excellent quality.
	2	0.8 – 1.7	100	100		
	3	1.7 – 2.8	100	100		
					0.8 – 2.8	PEGMATITE: Pink and white, coarse crystalline, high strength, with layers of pure biotite, low to medium strength, unweathered, moderate spaced flat partings, rough planar, tight, excellent quality.
LCN-6	1	0.0 – 1.1	100	100	0.0 - 3.1	MIGMATITE: Dark grey to black, medium crystalline, with occ. white bands and/or irregular inclusions (leucosome?). coarse crystalline, trace garnet, fine crystalline, high strength, unweathered, close to wide spaced flat partings, rough planar, tight, excellent quality.
	2	1.1 – 1.8	100	100		
	3	1.8 – 3.1	100	100		
LCN-7	1	0.0 – 1.2	100	98	0.0 - 3.1	MIGMATITE: Medium grey, fine to medium crystalline, slight banding, garnetiferous, becoming light grey to pink, with occ. layers of pegmatite, black and white, coarse crystalline, high strength, unweathered, generally moderate to wide (locally very close to close) spaced flat partings, rough planar, tight, fair to excellent quality.
	2	1.2 – 2.0	100	72		
	3	2.0 – 3.1	100	100		

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 (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	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
r_u	1	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	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	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	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1, %	VOID RATIO

n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT

e_{max}	1, %	VOID RATIO IN LOOSEST STATE
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation Ontario

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Foundation Design

RECORD OF BOREHOLE No LCN-1

METRIC

W.P.	5262-05-01	LOCATION	Hwy 69(New), Sta. 14+363, o/s 18.8m Rt 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 (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
211.4	Ground Surface										
0.0 / 0.1	Topsoil										
	End of borehole										
	Refusal on probable bedrock										
	* Borehole dry										

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$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No LCN-2

METRIC

W.P. 5262-05-01 LOCATION Hwy 69(New), Sta. 14+383, o/s 11.8m Rt. 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					DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
209.0	Ground Surface					*					
0.0 / 208.7	Topsoil										
0.3	End of borehole Refusal on probable bedrock										

* Borehole dry

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$+^7, \times^5$: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No LCN-3

METRIC

W.P. 5262-05-01	LOCATION Hwy 69(New), Sta. 14+383, o/s 18.8m Rt. 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.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
205.6	Ground Surface					
0.0	Topsoil, cobbles and boulders					
0.3	Cobbles and boulders with sand, with silt				205	
204.6 / 1.0	Bedrock	1	RC NQ	REC 100%		RQD 0%
	Migmatite: light grey to light green and pink, high strength, slightly weathered to unweathered, very poor to fair quality	2	RC NQ	REC 100%	204	RQD 0%
		3	RC NQ	REC 100%	203	RQD 53%
		4	RC NQ	REC 98%	202	RQD 9%
201.6 / 4.0	End of borehole					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS *; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GR SA SI CL

* Borehole charged with drilling water

ON_MOT VER 3A 06TF052C.GPJ ON_MOT.GDT 7/10/2007 11:46:40 AM

$+^7, \times^5$: Numbers refer to Sensitivity

20 / 15–5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No LCN-4

METRIC

W.P. 5262-05-01 LOCATION Hwy 69(New), Sta. 14+383, o/s 25.8m Rt. 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 March 27, 2007 CHECKED BY C.N.

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, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
201.9	Ground Surface					*					
0.0	Topsoil cobbles and boulders										
201.3 / 0.6	Boulders and fractured rock		1	RC NQ	-		201				
			2	RC NQ	-						
200.1 / 1.8	End of borehole Refusal on probable boulders										

* Borehole charged with drilling water

+⁷, ×⁵ : Numbers refer to Sensitivity   20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No LCN-5

METRIC

W.P. 5262-05-01 LOCATION Hwy 69(New), Sta. 14+427, o/s 11.8m Rt. of CL Med. ORIGINATED BY N.L.B.

DIST 54 HWY 69 BOREHOLE TYPE Portable Rotary Drill COMPILED BY N.R.

DATUM Geodetic DATE April 04, 2007 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
200.2	Ground Surface				*		
0.0	Bedrock Migmatite: light grey to pink high strength unweathered excellent quality	1	RC NQ	REC 100%		200	RQD 93%
	Pegmatite: pink and white high strength unweathered excellent quality	2	RC NQ	REC 100%		199	RQD 100%
		3	RC NQ	REC 100%		198	RQD 100%
197.4 / 2.8	End of borehole						

* Borehole charged with drilling water

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; GR SA SI CL

$+^{7}, \times^{5}$: Numbers refer to Sensitivity 20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No LCN-6

METRIC

W.P. 5262-05-01 LOCATION Hwy 69(New), Sta. 14+427, o/s 18.8m Rt. of CL Med. ORIGINATED BY N.L.B.

DIST 54 HWY 69 BOREHOLE TYPE Portable Rotary Drill COMPILED BY N.R.

DATUM Geodetic DATE April 03, 2007 CHECKED BY C.N.

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 (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
200.5	Ground Surface										
0.0	Bedrock Migmatite: dark grey to black high strength unweathered excellent quality		1	RC NQ	REC 100%		200				RQD 100%
			2	RC NQ	REC 100%		199				RQD 100%
			3	RC NQ	REC 100%		198				RQD 100%
197.4 3.1	End of borehole										
	* Borehole charged with drilling water										

ON_MOT VER 3A 08TF052C.GPJ ON_MOT.GDT 7/9/2007 2:58:37 PM

$+^7$, $\times^5$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No LCN-7

METRIC

W.P. 5262-05-01 | LOCATION Hwy 69(New), Sta. 14+427, o/s 25.8m Rt. of CL Med. | ORIGINATED BY N.L.B.

DIST 54 HWY 69 | BOREHOLE TYPE Portable Rotary Drill | COMPILED BY N.R.

DATUM Geodetic | DATE April 04, 2007 | CHECKED BY C.N.

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 (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
198.6	Ground Surface					*					
0.0	Bedrock Migmatite: medium grey high strength unweathered fair to excellent quality		1	RC NQ	REC 100%		198				RQD 98%
			2	RC NQ	REC 100%		197				RQD 72%
			3	RC NQ	REC 100%		196				RQD 100%
195.5 / 3.1	End of borehole										
	* Borehole charged with drilling water										

ON_MOT VER 3A 06TF052C.GPJ ON_MOT.GDT 7/9/2007 2:58:37 PM

$+^{7}, \times^{5}$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No LCN-8

METRIC

W.P. 5262-05-01 LOCATION Hwy 69(New), Sta. 14+447 o/s 18.8m Rt, 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.

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 (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
195.7	Ground Surface										
0.0 / 0.1	Topsoil										
	Silt, with sand trace gravel, trace clay										
194.9 / 0.8	Brown/ grey Moist						195				
	End of borehole										
	Refusal on probable boulders										
	* Borehole dry										

ON_MOT VER 3A 06TF052C.GPJ ON_MOT.GDT 7/9/2007 2:58:38 PM

$+^7$, $\times^5$: Numbers refer to Sensitivity

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

METRIC

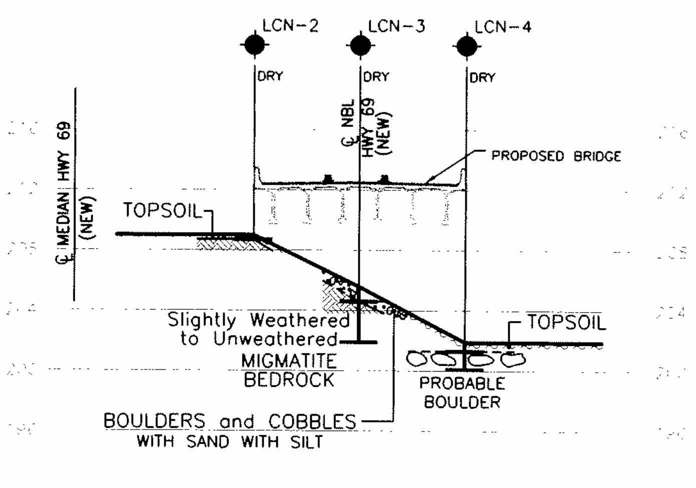
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No
WP No 5262-05-01

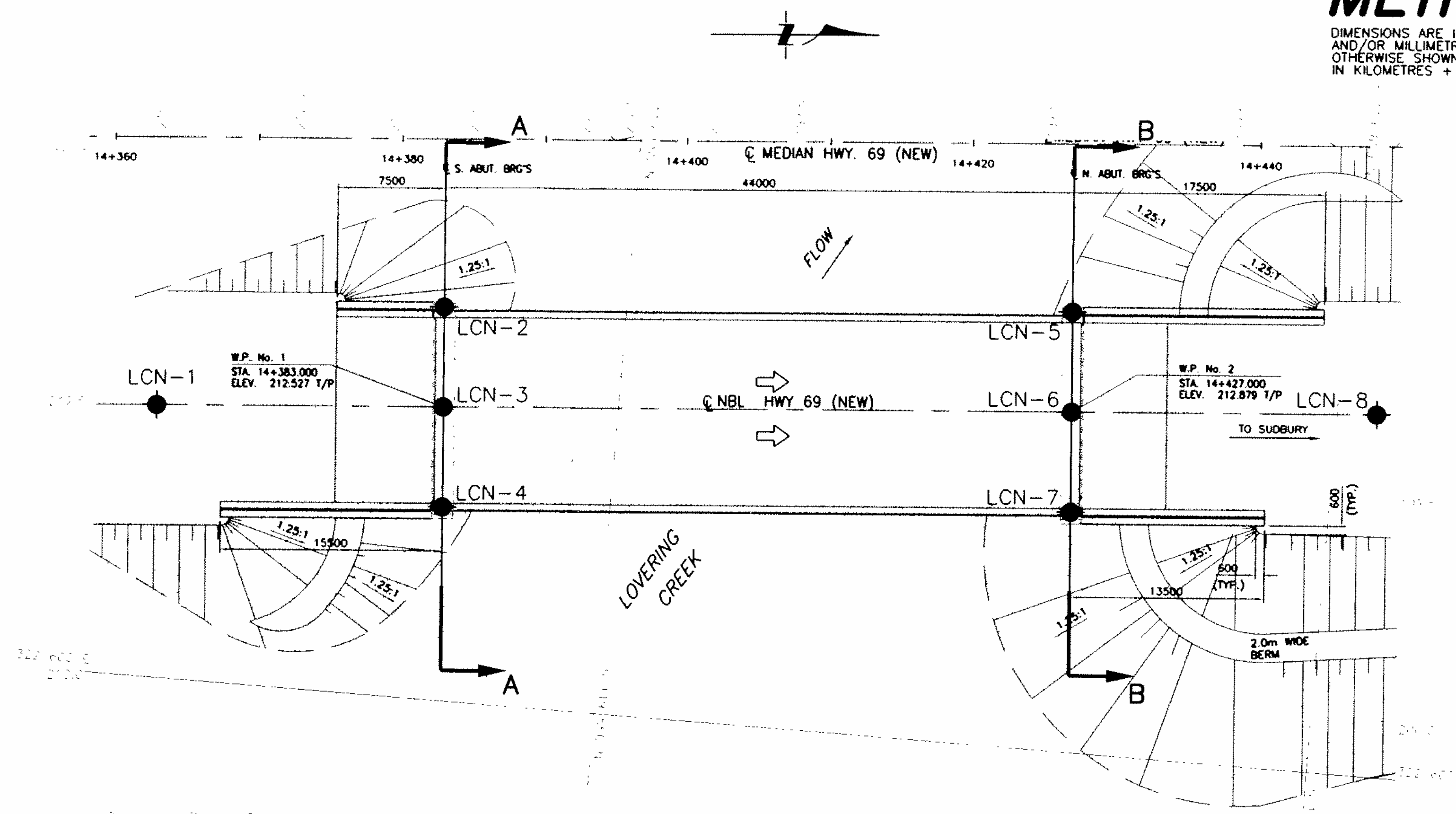
LOVERING CREEK OVERPASS NBL
HIGHWAY 69
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS

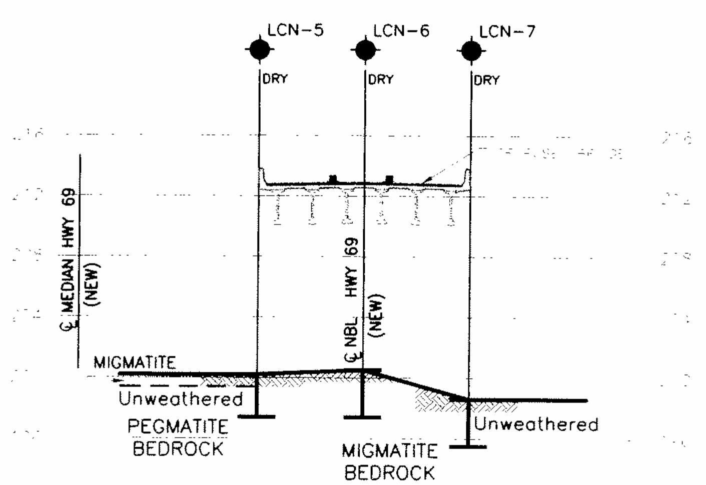


A–A



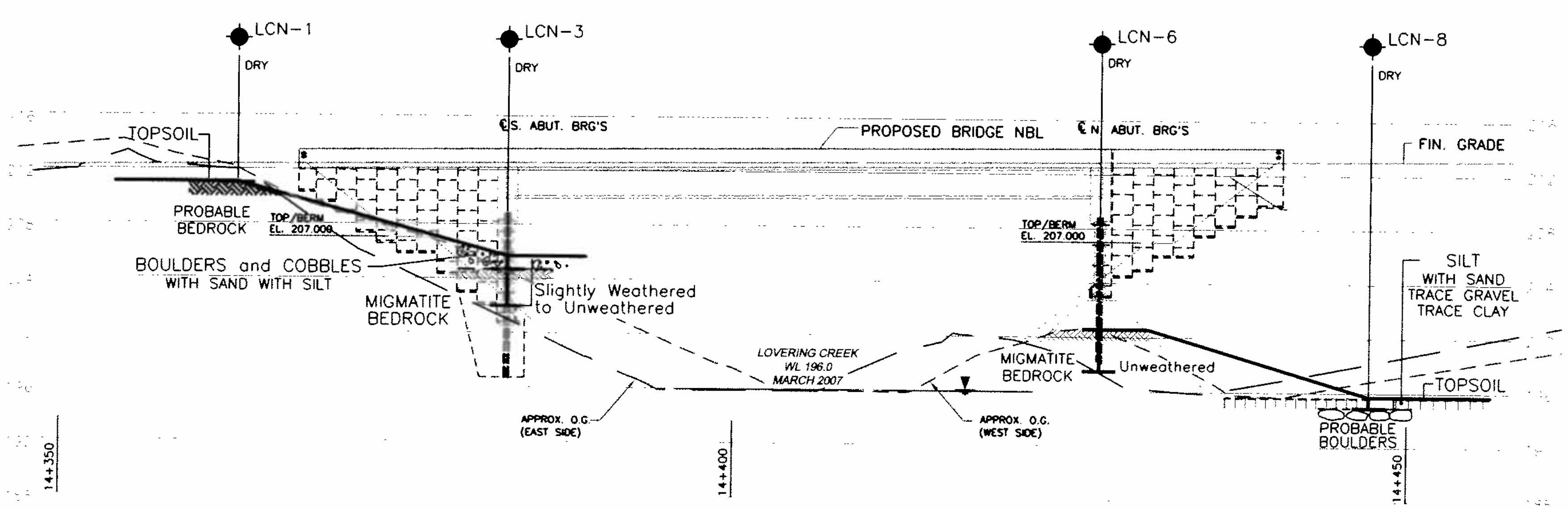
PLAN

SCALE 5 0 5 10m



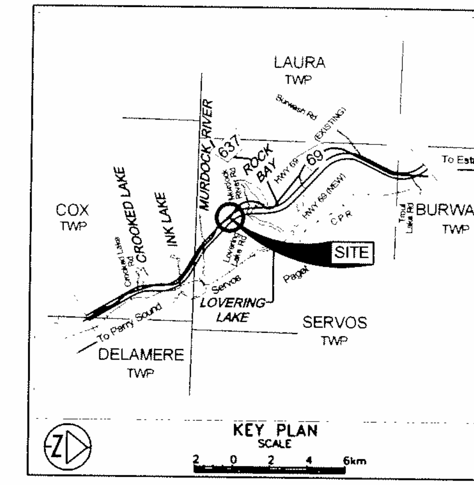
B–B

SCALE 5 0 5 10m



℄ NBL PROFILE

SCALE 5 0 5 10m



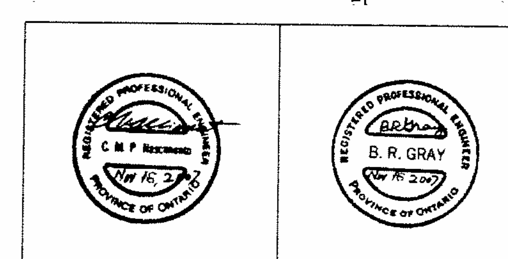
LEGEND

- 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 HWY 69 NBL
LCN-1	211.4	14+363	18.8m Rt.
LCN-2	209.0	14+383	11.8m Rt.
LCN-3	205.6	14+383	18.8m Rt.
LCN-4	201.9	14+383	25.8m Rt.
LCN-5	200.2	14+427	11.8m Rt.
LCN-6	200.5	14+427	18.8m Rt.
LCN-7	198.6	14+427	25.8m Rt.
LCN-8	195.7	14+447	18.8m Rt.

– NOTE –

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



REVISIONS		
DATE	BY	DESCRIPTION

Geocres No. 42I-215

HWY No 69			DIST SUDBURY
SUBM'D FP	CHECKED CN	DATE NOV. 16 2007	SITE 46-509/1
DRAWN NA	CHECKED CN	APPROVED BRG	DWG LCN-1

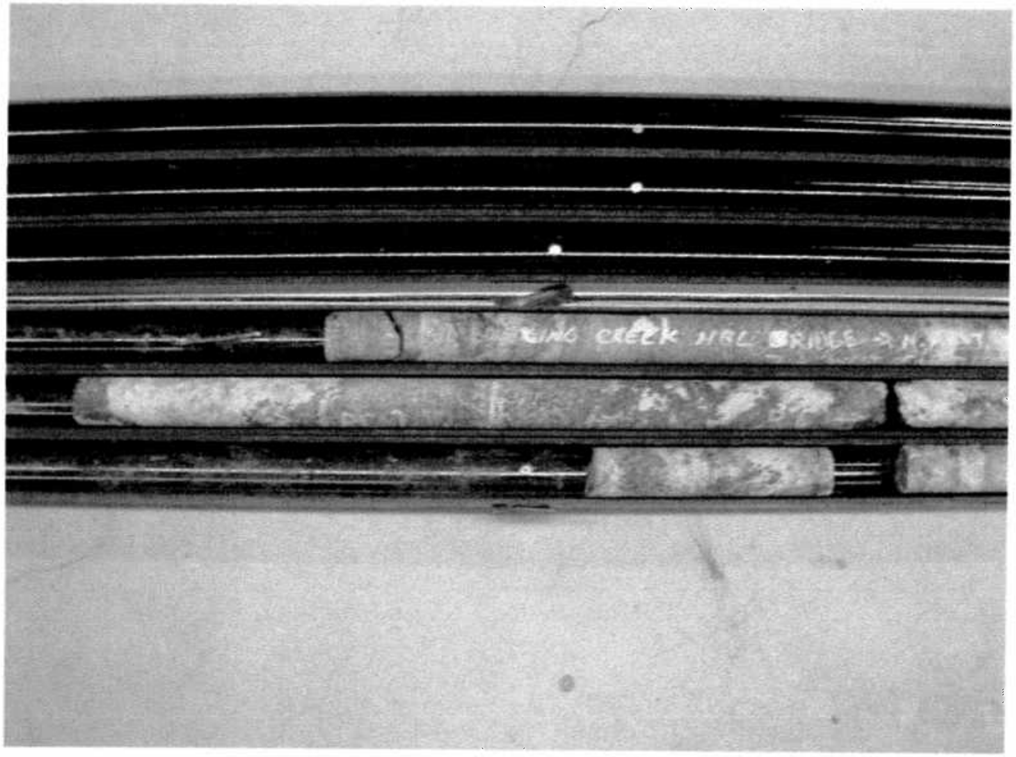
NOTES:

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

REF TSH DRAWINGS: 42-91088-NBL_SBL_LOVERING CREEK.dwg, 470_ph_base_nad83_z12.dwg, Phase1_Contours.dwg

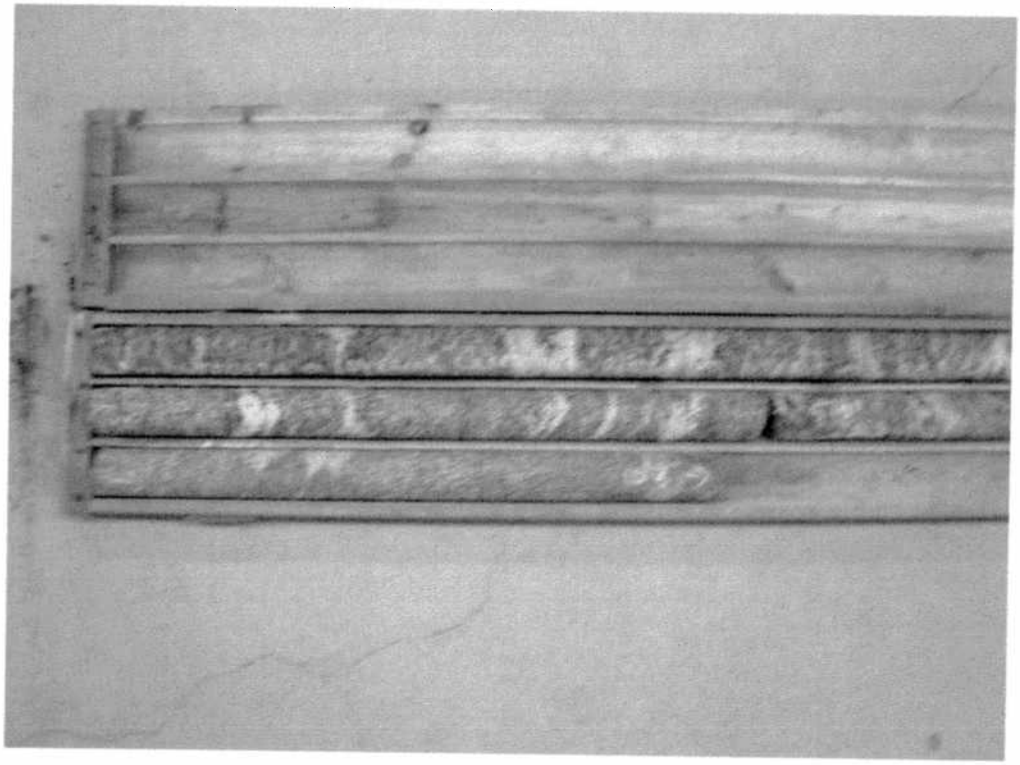
APPENDIX A

Rock Core Photographs

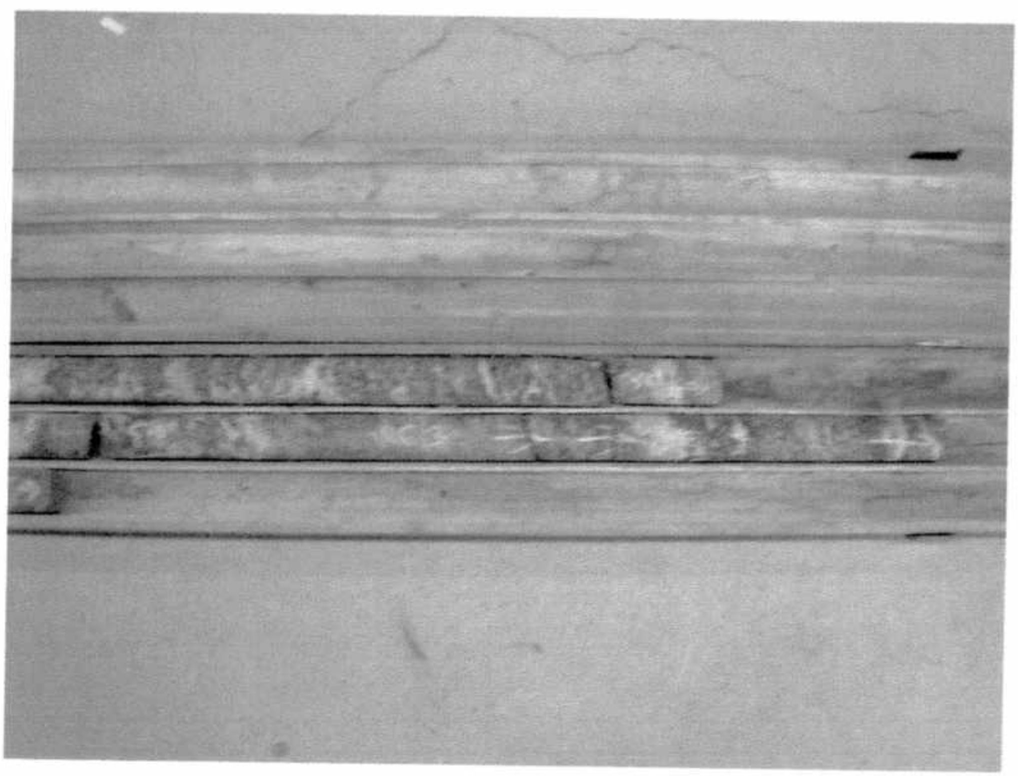


Photograph 1: Rock core from LCN-5

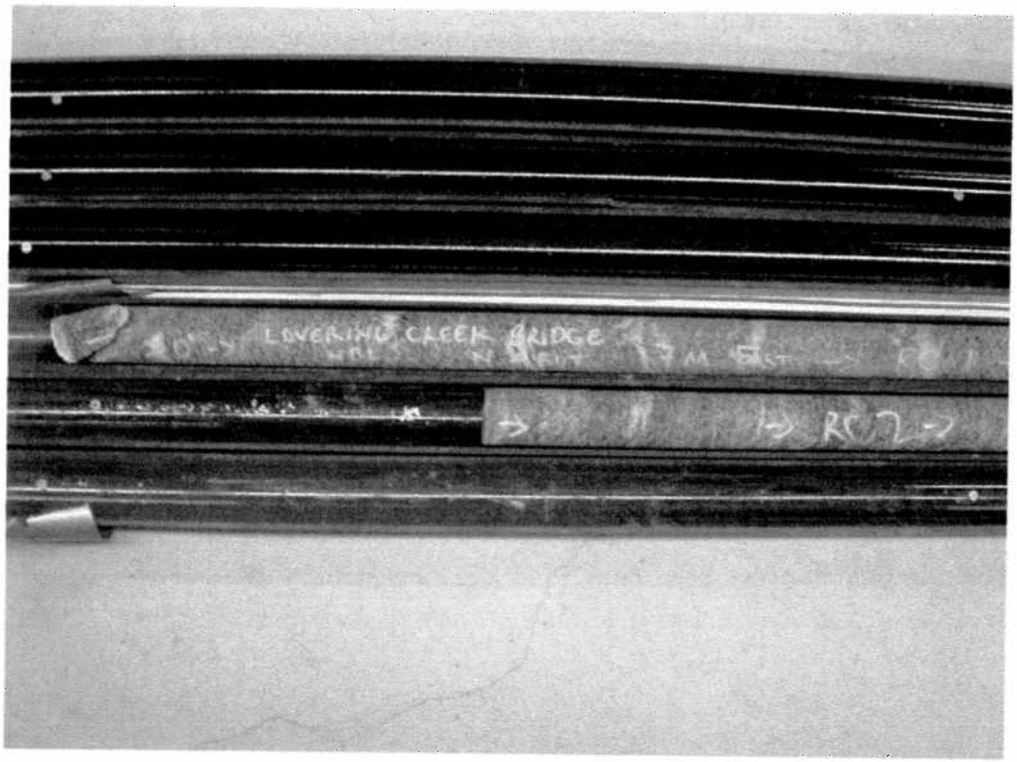
Photograph 2: Rock core from LCN-5



Photograph 3: Rock core from LCN-6

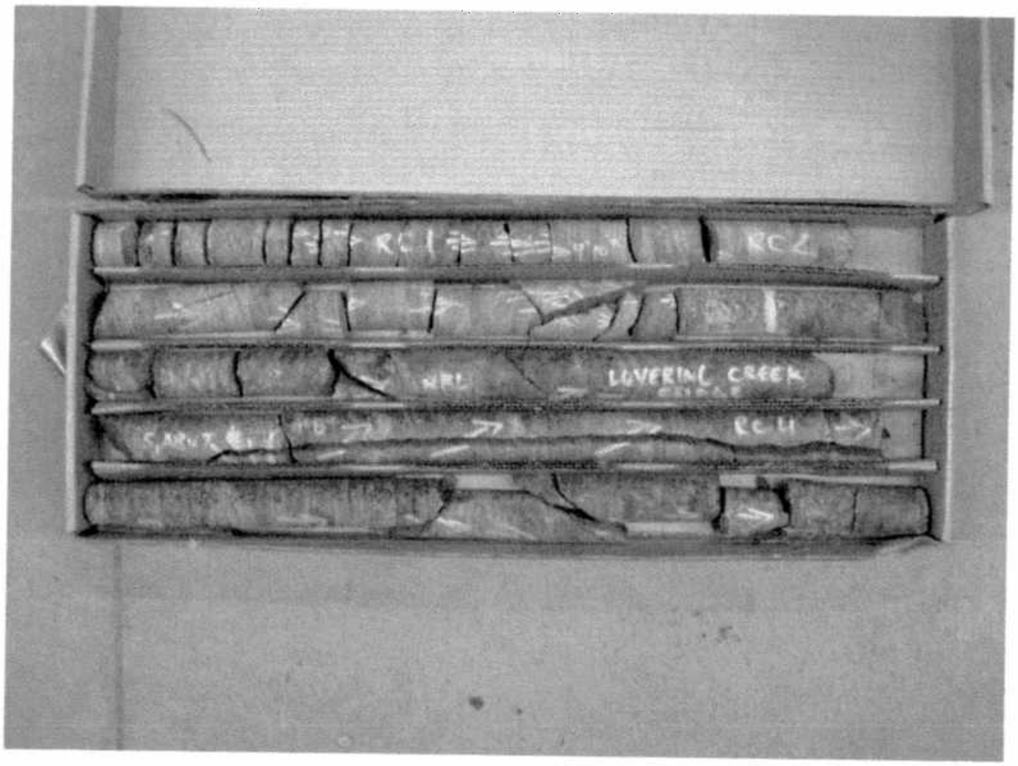


Photograph 4: Rock core from LCN-6



Photograph 5: Rock core from LCN-7

Photograph 6: Rock core from LCN-7



Photograph 7: Rock core from LCN-3

APPENDIX B

Site Photographs

Photograph 1: North abutment of NBL facing east. Close-up view of outcrop (April 2, 2007).

Photograph 2: North abutment of NBL facing south. Note Lovering Creek in front of treed slope. (April 2, 2007).

Photograph 3: South abutment of NBL facing west. Note talus slope (April 4, 2007).

Photograph 4: North abutment of NBL facing west. Note outcrop (April 4, 2007)

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
for
LOVERING CREEK BRIDGE NORTHBOUND
HIGHWAY 69
SITE NO. 46-509/1, W.P. 5262-05-01
DISTRICT 54, SUDBURY, ONTARIO

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

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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.: 06TF052C
Index No.: 425FDR
GEOCRES No.: 41I-215
November 16, 2007

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
for
Lovering Creek Bridge Northbound
Highway 69
Site No. 46-509/1, W.P. 5262-05-01
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 Northbound to be located about 43.7 km south of Sudbury, Ontario. The investigation was conducted for Totten Sims Hubicki Associates on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed bridge will carry the new Highway 69 northbound lanes (NBL) over the existing Lovering Creek between approximate stations 14+383.0 and 14+427.0.

The proposed Lovering Creek Bridge northbound consists of a 44.0 m long, single span structure. The approach embankments to the south and north abutments will be raised about 6.9 to 12.4 m above existing grade, respectively.

The subsurface soil stratigraphy revealed in the boreholes generally comprised a non-existent to very shallow soil cover comprising surficial topsoil over a discontinuous layer of scattered cobbles and boulders (south abutment and approach) and rock outcrop or local deposit of surficial topsoil over silt (north abutment and approach). Numerous cobbles and boulders were encountered in the soil cover.

The soil cover mantled bedrock and bedrock outcrops were noted. The depth to the bedrock ranges from 0.3 to 1.8 m below grade at the south abutment and bedrock at surface was found at the north abutment. The quality of the typical migmatite (with a local pegmatite band) bedrock is generally excellent at the north abutment. At the south abutment, the rock is a very poor to fair quality migmatite bedrock. The rock was assessed as high strength.

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BARRIE, HAMILTON, KITCHENER, TORONTO

Based on the encountered subsurface conditions and the height of the embankments over the existing ground surface, the overpass foundations may be founded on spread footing placed on the bedrock or engineered fill. Alternatively, deep foundations are feasible provided that the rock is excavated in a trench 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 presence of numerous boulders within the native soils, in particular within the talus slope deposit found at the south abutment and shallow soil cover, 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 alternative. The trench through the 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 engineered/structure 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.

The elevations referred in this report are expressed 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 talus slope containing numerous boulders at the south abutment may contain loose zones and it is not considered adequate founding subsoil.

It is considered feasible to support the foundations of the proposed abutment on spread footings constructed on the exposed bedrock at the north abutment and on the underling bedrock at the south abutment. The bridge may also be founded on spread footings constructed on engineered fill.

The bedrock surface at the centreline boreholes drilled for the abutments of the proposed Highway 69 NBL bridge was found at or within 7.9 and 12.4 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, 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, the 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 liquefaction potential of the shallow and localized silt at the site was assessed using the procedure suggested by Seed and Idriss (1971) and, on this basis it is considered that liquefaction of the native soil is unlikely (clause 4.6.2 of the CHBDC).

2.2 Footings on Bedrock

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

FOUNDATION UNIT	SUBGRADE TYPE	REFERENCE ELEVATIONS	DEPTHS* (m)
South Abutment	Bedrock	208.7 (LCN-2) and 204.6 (LCN-3)	0.3 and 1.0
North Abutment	Bedrock	198.6 to 200.5 (LCN-5, 6 and 7)	0.0

*Depth from existing ground surface. The minimum 2.0 m frost protection was not considered required for footings on the bedrock.

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	5,000	Not applicable
North Abutment	Bedrock	10,000	Not applicable

A reduction of 50% was applied to the geotechnical resistance at the south abutment in view of the very poor rock quality. The fractured rock within the footprint of the footings should be excavated 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 for the ULS assessment for footings on 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 35 MPa. If the actual bearing pressure is less than 10,000 kPa, the compressive strength of the concrete could be reduced in direct linear proportion to the actual bearing pressure (minimum value of 2500 kPa).

Comments concerning excavation of the bedrock, if required to found the footings on sound bedrock at a level lower than indicated in the 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 native soil comprises of a bouldery talus slope (south 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.

The recommended bearing resistance for a minimum 2.0 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 have the north abutment founded on piles driven through a granular fill pad placed on the existing bedrock. To found the south abutment on piles, a trench should be excavated/blasted into the existing rock outcrop where the pile length will be less than 5 m. This bedrock removal will be required 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 range of reference founding levels for piles for the north abutment and the maximum levels of the rock excavation 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
North Abutment	0.0	198.6 to 200.5	LCN-5, LCN-6 and LCN-7
South Abutment	0.3 and 1.8	Maximum 204.5	LCN-2 and LCN-3

* 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 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 consolidation of the shallow native clayey silt/silty clay and overlying cohesionless soils under the embankment loading will occur during construction.

Refer to Section 4 for a discussion and recommendations on the treatment of approach embankment settlements.

The presence of numerous cobbles/boulders was identified above bedrock at depth in boreholes LCN-3 and LCN-4. 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. 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 6 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 the north abutment since the slope of the bedrock revealed in the north abutment boreholes ranges from 2 to 15°. For uniformity purposes, the piles for the south abutment, which will be installed in a rock trench, should also be provided with the same type of rock points.

2.5 Integral Abutments on Piles

The bedrock surface level at the south abutment is at elevations 208.7 and 204.6, boreholes LCN-2 and LCN-3, respectively that is 3.8 and 7.9 m below the proposed top of pavement elevation 212.5.

The depth of excavation of a trench into rock to accommodate 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:

	GRANULAR BACKFILL	
Pile Section	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 the native undisturbed soils or through compacted granular materials placed as recommended previously. If greater resistance is required, batter piles should be installed.

To evaluate the point of contraflexture, 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 MN/m^3 for granular backfill
z = depth, m
b = pile width, m

The cohesionless soil parameter is applicable to all 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 Native Soil or 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 engineered 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:

$$p = K(\gamma h + q) + Cp + Cs$$

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
Cp = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
Cs = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where $\varnothing$ = 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 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 WALLS

A retained soil system (RSS) could also be employed at the abutments provided the estimated settlements noted in Section 5 Approach Embankments 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 talus slope deposit encountered at south abutment are highly compressible or heterogeneous and not considered adequate to support the RSS wall footings and should be removed from the RSS wall footprint together with all soil containing organic materials.

The RSS walls at the south abutment will be placed on the bedrock or on a structural fill layer over the bedrock at various levels ranging from elevations 204.5 to 210.0. The structural fill should be compacted to 100% of the standard Proctor maximum dry density. The bearing resistances recommended previously for spread footings founded on bedrock or structural fill constructed on bedrock are considered to be suitable for the RSS wall footings. The anticipated width of the RSS footing is 600 mm.

The RSS wall footings for the north abutment should be placed on a structural fill pad at varying levels ranging from elevation 204.5 to 210.0. The engineered fill pad should be founded at the level of the bedrock or native silty sand encountered below the existing topsoil, elevations 195.6 to 200.5. The recommended bearing resistance for the north abutment 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 and compressible clayey soils within the backfill zone of the abutments and retaining walls should be excavated prior to placement of the backfill.

The level of the approach embankments will be raised about 6.9 m (south) and 12.4 m (north) above the existing ground 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 0.0 to 1.0 m thick 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 cover bedrock at shallow depths. Construction of the embankment fill remote from the abutments on the existing soils 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 1.25H:1V 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 width 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.

Since the new embankments will be constructed on a 0.2 to 0.8 m thick soil layer overlying 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 during and following completion of construction will result from two mechanisms – consolidation of the existing native soils below the embankment fill and "self weight" consolidation of the embankment fill.

The settlement of new rock fill should be in the order of 35 to 60 mm if constructed in accordance with the requirements of SP 902S01 and OPSS 501 (Method A).

Consolidation of the underlying soil is expected to be in the order of 5 to 10 mm. Hence, the total consolidation settlement could be 40 to 70 mm. 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 and suitable vegetation. Refer to OPSS 571 and 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 0.3 and 1.0 m thick in-situ soil cover at the south abutment and from the rock surface at the north abutment, respectively. Cobbles and boulders should be expected in the excavations.

The talus slope containing boulders at the south abutment and the compact silt 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. 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 workmen.

Groundwater at the south and north abutments 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 retaining 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

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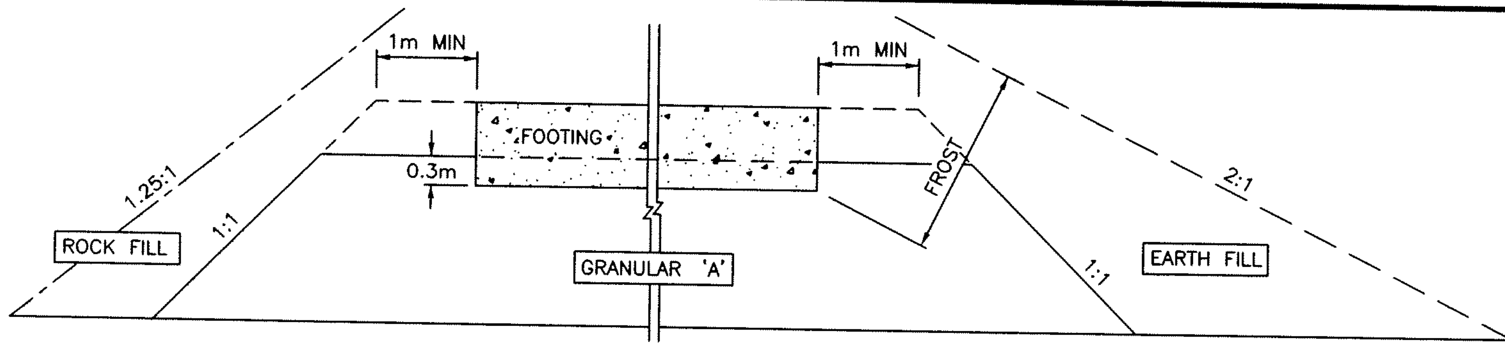
TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

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 2001
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	Walls 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

TABLE 2

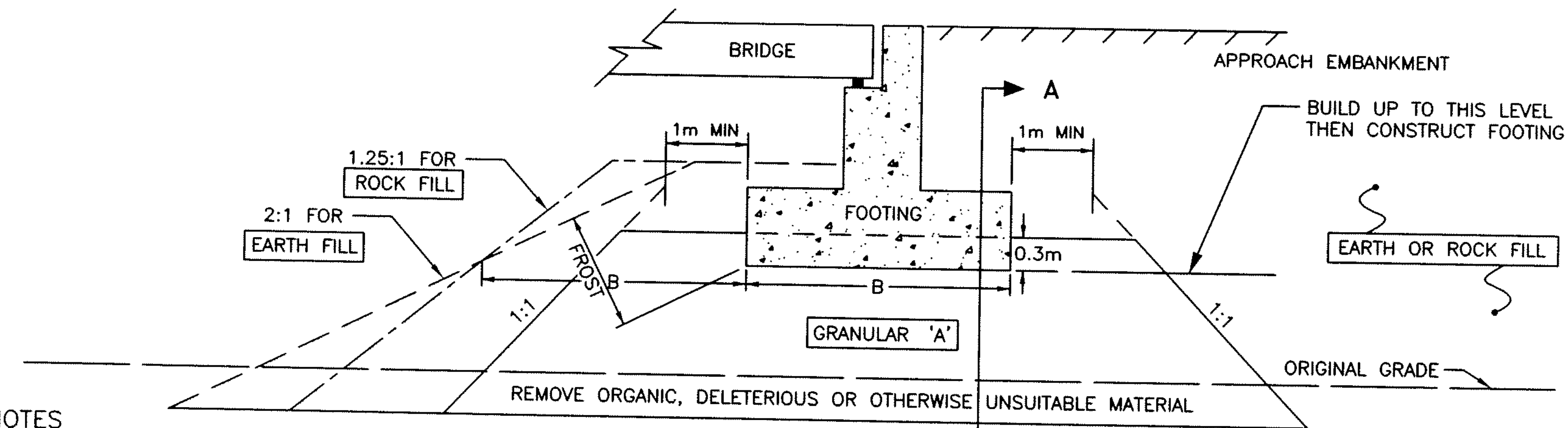
GRADATION SPECIFICATION FOR SAND FILL IN PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS

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