

REMARKS: \_\_\_\_\_

**FOUNDATION INVESTIGATION REPORT  
FOR  
BLACKSTONE/CRAVE LAKE ROAD UNDERPASS  
W.P. 408-97-01  
G.W.P. 290-97-00, SITE 44-383  
HIGHWAY 69, DISTRICT 52  
HUNTSVILLE, ONTARIO**

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Job No. 97TF088A  
Geocres No. 31E-131

August, 1999

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## **FOUNDATION INVESTIGATION REPORT**

For

Blackstone/Crane Lake Road Underpass

W.P. 408-97-01

G.W.P. 290-97-00, Site 44-383

Highway 69, District 52, Huntsville

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### **INTRODUCTION**

This report summarizes the results of the foundation investigation carried out for construction of the Highway 69 underpass at Blackstone/Crane Lake Road (Station 12+447 Highway 69 chainage).

The report pertains to the proposed bridge structure and approaches within about 20 m of the abutments, between approximate stations 9+940 and 10+060, Blackstone/Crane Lake Road chainage.

### **SITE DESCRIPTION**

The site is located about 25 km north of MacTier and about 500 m southwest of the existing Highway 69 alignment. The proposed structure will carry Blackstone/Crane Lake Road traffic over the new four-lane section of Highway 69. At the underpass, the structure will run roughly northeast-southwest.

The lands adjacent to the existing road are generally wooded. The ground surface undulates with a topographic relief of a few metres. It was snow covered at the time of the fieldwork.

The site is located in the Precambrian Laurentian peneplane. The topography is irregular in detail with many small lakes separated by ridges of Precambrian bedrock. The surface in general is relatively flat. The overburden in the region is typically shallow but can vary substantially in thickness over short distances. Swamp environments have developed in areas of poor drainage.

## **INVESTIGATION PROCEDURES**

The fieldwork was carried out on February 23 and 24, 1998 and comprised six boreholes and four test pits drilled/excavated at the locations shown on Drawing 1. The boreholes/test pits are numbered sequentially for ease of reference.

The boreholes were drilled to refusal on bedrock/inferred bedrock at depths of 1.3 to 4.6 m. Three of the boreholes were extended an additional 2.9 to 3.4 m into the bedrock using NQ rock coring equipment.

The boreholes were advanced using continuous flight hollow stem augers, powered by a truck-mounted CME-75 drillrig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff.

Samples of the overburden were recovered using a conventional split spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. The groundwater conditions in the boreholes were closely monitored during the course of the fieldwork.

Test pits were excavated at locations not accessible to the drillrig. The test pits were excavated to depths of 1.5 to 4.1 m using a backhoe supplied and operated by a local excavation contractor. This work was supervised throughout by a member of our engineering staff who directed the excavating, logged the soil and groundwater conditions and collected representative samples of the soils encountered.

All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determinations. A grain size distribution analysis was carried out on a selected sample. Samples of the recovered rock core were subjected to unconfined compressive strength tests.

## **SUMMARIZED SUBSURFACE CONDITIONS**

Reference is made to the appended Log of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration test "N" values, rock core descriptions, groundwater observations and the results of laboratory moisture content determinations. Stratigraphic profiles prepared from the borehole data are presented on Drawing 1.

The stratigraphy revealed in the boreholes generally comprised a surficial layer of topsoil or sand and gravel fill, overlying native sand/silt, mantling bedrock. The fill extended to bedrock at one location. The strata encountered are summarized below.

### **Sand and Gravel Fill**

Sand and/or gravel fill was encountered surficially in all of the boreholes. It was not present at the test pit locations. In general, the fill extended to depths of 0.3 to 1.2 m. In boreholes 381-3 and 7, it extended to depths of 3.5 and 2.2 m, respectively, and contained decayed wood in the lower portions. The fill mantled bedrock in borehole 381-3.

### **Topsoil**

Topsoil was encountered surficially in all of the test pits (Nos. 381-4, 6, 8 and 10). The topsoil layer was 100 to 200 mm thick and was peaty with a high organic content.

### **Silt**

A 0.5 to 1.8 m thick layer of silt was encountered below the fill in boreholes 383-1, 5, 7 and 9. The silt was sandy and compact in boreholes 383-1 and 9; it was clayey and stiff in boreholes 383-5 and 7. It was penetrated at 3.0 m depth in borehole 383-7 and mantled bedrock/inferred in the remaining boreholes.

### Sand

A sand deposit was encountered below the fill/topsoil in all boreholes/test pits along the south side of the bridge (Nos. 383-2, 4, 6, 8 and 10) as well as in borehole 383-7.

The sand was typically silty and fine to coarse grained. The results of a grain size analysis conducted on a recovered sample are presented on Figure 1. The sand was dense in the boreholes and judged to be compact in the test pits. Moisture contents of about 10% were measured in two samples. The sand mantled bedrock/inferred bedrock.

### Bedrock

Bedrock or inferred bedrock was contacted below the fill, silt or sand in all boreholes/test pits at depths of 1.3 to 4.6 m (elevation 231.3 to 236.5).

A description of the rock cores recovered from boreholes 383-3, 5 and 7 is provided on Table I. The bedrock consists of biotite migmatite at boreholes 383-3 and 7, granitic gneiss at borehole 383-5. Core recovery was 100% and the RQD ranged from 70 to 100%. The rock was described as fair to excellent quality. The upper 430 mm in borehole 383-7 was very poor quality rock (RQD of 0%).

The unconfined compressive strength of selected core samples were as follows:

<u>Borehole No.</u>	<u>Depth (m)</u>	<u>Unconfined Compressive Strength (MPa)</u>
383-3	3.4	106.2
383-5	5.3	117.2
383-7	4.9	109.2

Groundwater

During drilling/sampling, free water was observed in boreholes 383-3, 5 and 7 at depths of 1.5 to 2.2 m (elevations 234.3 to 235.5). Free water was also observed in test pit 383-8 at 3.3 m depth (elevation 233.7) upon completion of excavating. Observed water levels are subject to seasonal fluctuations and rainfall patterns.

CLOSURE

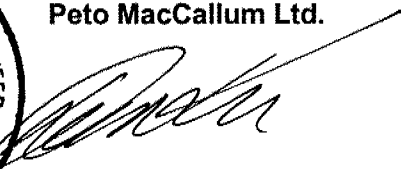
The fieldwork was carried out under the supervision of M. Rapsey, Senior Drillrig Supervisor. The equipment was supplied by All-Terrain Drilling Limited.

The report was written by M.R. Anderson, P.Eng., Project Engineer and reviewed by D.W. Kerr, P.Eng., Manager of Geotechnical and Geo-Environmental Services, Hamilton.

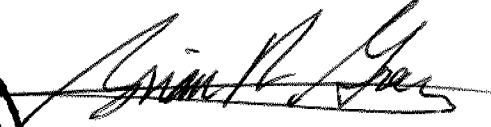
Yours very truly

**Peto MacCallum Ltd.**



  
Dennis W. Kerr, M.Eng., P.Eng.  
Manager Geotechnical and  
Geo-Environmental Services



  
Brian R. Gray, M.Eng., P.Eng.  
Vice-President  
Geotechnical and  
Geo-Environmental Services

MRA:mma



Our Ref: 97TF088A

**TABLE I**

**ROCK CORE DESCRIPTION**  
**WP 408-97-01, Site No. 44-383**

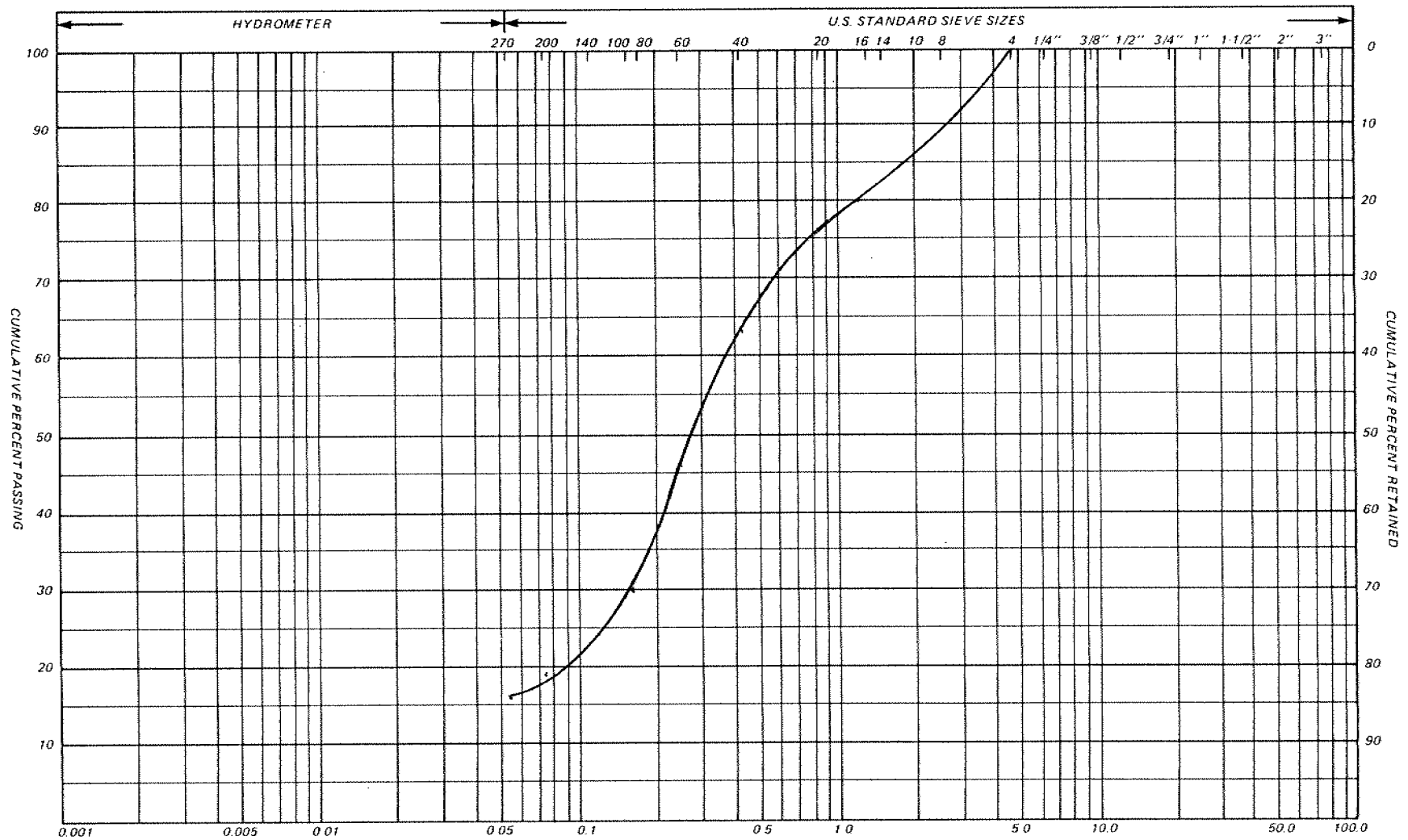
CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
383-3	4	3.20 - 6.35	100	70	3.20 - 4.17	<b>BIOTITE MIGMATITE</b> , black banded, high strength, unweathered; close to moderate spaced discontinuities, with oxidation on parting surface, fair quality
					4.17 - 4.60	5 mm wide dipping joint, rough planar, infilled with clay/silt
					4.60 - 5.80	becoming lighter, separating along dipping schistosity plane
					5.80 - 6.35	with bands/layers of pink granitic gneiss
383-5	2	2.69 - 5.31	100	100	2.69 - 6.00	<b>GRANITIC GNEISS</b> , light grey to pink, fine crystalline, high strength, unweathered; close to wide spaced discontinuities, occ. near vertical banding, good quality
	3	5.31 - 6.02	100	82	6.00 - 6.02	2 mm wide dipping joint, rough planar, infilled with silt
383-7	3	4.57 - 5.00	100	0	4.57 - 5.00	<b>BIOTITE MIGMATITE</b> , black to grey banded to heterogeneous, high strength, unweathered; very close to moderate spaced discontinuities, occ. dipping parting, rough planar, tight with red oxidation on parting surface, very poor quality
	4	5.00 - 7.95	100	100	5.00 - 7.95	excellent quality

RQD = Rock Quality Designation

Logged by J. Wright

# PARTICLE SIZE DISTRIBUTION CHART

Figure: 1  
OUR PROJECT NO. 97TF088A



SILT & CLAY				FINE SAND			MEDIUM SAND		COARSE SAND		GRAVEL		COBBLES	UNIFIED
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL		COBBLES					M.I.T.
CLAY	SILT			V. FINE	FINE	MED.	COARSE	GRAVEL						U.S. BUREAU

REMARKS: **Test Pit 383-8**  
**Sample 1 at 3.0 m depth**  
**Fine to coarse sand, some silt**

## LIST OF ABBREVIATIONS

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N', - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 0.3m INTO THE SUBSOIL, DRIVEN BY MEANS OF A 63.5kg HAMMER FALLING FREELY A DISTANCE OF 0.76m.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 0.3m INTO THE SUBSOIL. THE DRIVING ENERGY BEING 475 J PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3 m</u>	<u>c kPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3 m</u>
VERY SOFT	0 - 2	0 - 12	VERY LOOSE	0 - 4
SOFT	2 - 4	12 - 25	LOOSE	4 - 10
FIRM	4 - 8	25 - 50	COMPACT	10 - 30
STIFF	8 - 15	50 - 100	DENSE	30 - 50
VERY STIFF	15 - 30	100 - 200	VERY DENSE	> 50
HARD	> 30	> 200		

W.T.P.L. WETTER THAN PLASTIC LIMIT

D.T.P.L. DRIER THAN PLASTIC LIMIT

A.P.L. ABOUT PLASTIC LIMIT

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W	THINWALL OPEN
W.S	WASHED SAMPLE	T.P	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S	OESTERBERG SAMPLE
A.S	AUGER SAMPLE	F.S	FOIL SAMPLE
C.S	CHUNK SAMPLE	R.C	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

### SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL		

▲, ▲ - Undisturbed and remoulded shear strength determined from in situ vane test.

■ - Undrained shear strength determined from pocket penetrometer test.

## LOG OF BOREHOLE NO. 383-1

**N 5 016 266**  
**E 276 862**

PROJECT W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE  
SITE Blackstone/Crane Lake Road, Site 44-383  
LOCATION Station 9+946 (Blackstone/Crane Lake Road) 5.7m Lt.  
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A  
ENGINEER M. R. Anderson  
TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		SHEAR STRENGTH $c_u$				LIQUID LIMIT $w_L$			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST •				PLASTIC LIMIT $w_p$			
							BLOWS/0.3M				WATER CONTENT %			
							20	40	60	80	10	20		30
0	GROUND ELEVATION 237.55													
0.30	<u>SAND AND GRAVEL FILL</u> : Dark brown sand and gravel, some silt		237											
1.20	<u>SAND FILL</u> : Loose, brown fine sand, trace of silt													
1.5			236	1	SS	15	•				•			
1.83	<u>SILT</u> : Compact, brown fine sandy silt, trace of clay													
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 1.83m. BEDROCK ASSUMED.		235											
3.0													Upon completion of augering, no free water, no cave.	
4.5														
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

Upon completion of augering, no free water, no cave.

NOTES:

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## LOG OF BOREHOLE NO. 383-2

**N 5 016 258**  
**E 276 871**

PROJECT W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE  
SITE Blackstone/Crane Lake Road, Site 44-383  
LOCATION Station 9+942 (Blackstone/Crane Lake Road) 5.7m Rt.  
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A  
ENGINEER M. R. Anderson  
TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		SHEAR STRENGTH $C_u$ ▲				LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST •				WATER CONTENT %			
							BLOWS/0.3M				WATER CONTENT %			
							20	40	60	80	10	20		30
0	GROUND ELEVATION 237.52													
0.30	<u>SAND AND GRAVEL FILL</u> : Dark brown sand and gravel, some silt	XXXX	237											
	<u>SAND</u> : Dense, brown fine to medium sand, some silt and gravel, wet													
1.47			236	1	SS	35		•		•				
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 1.47m. BEDROCK ASUMED.												Upon completion of augering, no free water, no cave.	
3.0														
4.5														
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

NOTES:

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# Peto MacCallum Ltd.



CONSULTING ENGINEERS

## LOG OF BOREHOLE NO. 383-3

N 5 016 284  
E 278 870

PROJECT W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE  
 SITE Blackstone/Crane Lake Road, Site 44-383  
 LOCATION Station 9+966 (Blackstone/Crane Lake Road) 5.7m Lt.  
 BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring

OUR PROJECT 97TF088A  
 BORING DATE Feb. 23, 1998 ENGINEER M. R. Anderson  
 TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES				SHEAR STRENGTH $C_u$ ▲				LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$				GROUNDWATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N 1 VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST •				WATER CONTENT %					
							BLOWS/0.3M				WATER CONTENT %					
	GROUND ELEVATION 237.55						20	40	60	80	10	20	30			
0	<u>SAND AND GRAVEL FILL</u> : Compact dark brown, fine to medium sand and gravel, occasional cobbles		237											• No Recovery		
1.5			236	1	SS	14*	•									
2.50			235													
3.0	becoming silty, fine to medium sand, trace of gravel, occasional decayed wood		234	2	SS	65/300mm**									** 50 for last 150mm	
3.48	<u>BEDROCK</u> : Biotite Migmatite		233				762	100	0	0				After augering to 3.48m, free water at 2.05m.  Encountered a possible joint at 4.24m. Lost drill water and could not advance core. Moved 1.0m west and reaugered to 3.20m to core.		
4.5			232				RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)	3150	100	70		100	
6.0																
6.35	BOREHOLE TERMINATED AT 6.35m.		231								End of Run					
7.5											RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)		
9.0																
10.5																
12.0																
13.5																
15.0																
16.5																

NOTES:



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## LOG OF TEST PIT NO. 383-4

**N 5 016 276**  
**E 276 879**

**PROJECT** W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE  
**SITE** Blackstone/Crane Lake Road, Site 44-383  
**LOCATION** Station 9+962 (Blackstone/Crane Lake Road) 5.7m Rt.  
**DATE** February 24, 1998

**OUR PROJECT NO.** 97TF088A  
**ENGINEER** M. R. Anderson  
**TECHNICIAN** M. Rapsey

DEPTH	DESCRIPTION	LEGEND	SAMPLE NUMBER	PENETRATION T S F				WATER CONTENT %			GROUNDWATER OBSERVATIONS AND REMARKS
ELEV	GROUND ELEVATION			1	2	3	4	10	20	30	
0	237.55										
0.20	<u>TOPSOIL</u> : High organic peaty topsoil with numerous roots										
237.35											
	<u>SAND</u> : Compact, rusty brown, fine to coarse sand, some silt and gravel, occasional cobbles and boulders to 450mm diameter										
0.90											
236.65											
1.50											
236.05	TEST PIT TERMINATED UPON BEDROCK AT 1.50m.										Upon completion of excavating, no free water, sidewalls standing near vertical.
1.5											
3.0											

NOTES

CHECKED BY *MRP*

# PetoMacCallum Ltd.

CONSULTING ENGINEERS

## LOG OF BOREHOLE NO. 383-5

N 5 016 318  
E 276 883

PROJECT W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE

OUR PROJECT 97TF088A

SITE Blackstone/Crane Lake Road, Site 44-383

LOCATION Station 10+002 (Blackstone/Crane Lake Road) 5.7m Lt.

BORING DATE Feb. 23, 1998 ENGINEER M. R. Anderson

BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		SHEAR STRENGTH $C_u$				LIQUID LIMIT $W_L$			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST •				WATER CONTENT %			
							BLOWS/0.3M				WATER CONTENT %			
							20	40	60	80	10	20		30
0	GROUND ELEVATION 237.09												After augering to 2.7m, free water at 2.20m.	
0.30	GRAVEL FILL : Crushed stone													
0.90	SAND FILL : Brown, fine sand, some silt		236											
1.5	SILT : Stiff, brown clayey silt, some sand, slightly plastic, A.P.L.		235	1	SS	12	•					⊙		
2.69			234											
3.0	BEDROCK : Granitic Gneiss		233			2616	100	100	100					
4.5			232											
6.0			231			711	100	82	100					
6.02	BOREHOLE TERMINATED AT 6.02m.					RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)					
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

NOTES:

CHECKED BY: *Amor*



## LOG OF TEST PIT NO. 383-6

**N 5 016 310**  
**E 276 892**

**PROJECT** W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE  
**SITE** Blackstone/Crane Lake Road, Site 44-383  
**LOCATION** Station 9+998 (Blackstone/Crane Lake Road) 5.7m Rt.  
**DATE** February 24, 1998

**OUR PROJECT NO.** 97TF088A  
**ENGINEER** M. R. Anderson  
**TECHNICIAN** M. Rapsey

DEPTH	DESCRIPTION	LEGEND	SAMPLE NUMBER	PENETRATION T S F				WATER CONTENT %			GROUNDWATER OBSERVATIONS AND REMARKS
ELEV	GROUND ELEVATION			1	2	3	4	10	20	30	
0	237.94										
0.10	237.94	TOPSOIL : High organic peaty topsoil with numerous roots									
1.20	236.84	SAND : Compact, rusty brown, silty fine to coarse sand, some gravel, occasional cobbles and boulders, damp									
1.50	236.54	becoming brownish grey									
1.5	236.54	TEST PIT TERMINATED UPON BEDROCK AT 1.50m.									Upon completion of excavating, no free water, sidewalls standing near vertical.
3.0											

**NOTES**

CHECKED BY *mt*

## LOG OF BOREHOLE NO. 383-7

**N 5 016 351**  
**E 276 896**

PROJECT W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE

OUR PROJECT 97TF088A

SITE Blackstone/Crane Lake Road, Site 44-383

LOCATION Station 10+038 (Blackstone/Crane Lake Road) 5.7m Lt.

BORING DATE Feb. 23, 1998 ENGINEER M. R. Anderson

BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		SHEAR STRENGTH $C_u$				LIQUID LIMIT $W_L$			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST •				PLASTIC LIMIT $W_p$			
							BLOWS/0.3M				WATER CONTENT $W$			
											WATER CONTENT %			
0	GROUND ELEVATION 235.86						20	40	60	80	10	20	30	
0.30	GRAVEL FILL : Crushed stone													
	SAND AND GRAVEL FILL : Dark brown, pit run sand and gravel, some silt		235											
1.50	becoming loose fine sand, some silt, with decayed wood		234	1	SS	5								
2.15	SILT : Stiff, brown clayey silt, some sand, wet		233											
3.00	SAND : Dense, brown, silty fine sand, trace of gravel, saturated		232	2	SS	34								
3.95	becoming bouldery		231				432	100	0	100				
4.57	BEDROCK : Biotite Migmatite		230											
			229				2946	100	100	100				
			228											
7.95	BOREHOLE TERMINATED AT 7.95m.		227											

NOTES:

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## LOG OF TEST PIT NO. 383-8

N 5 016 344  
E 276 905

**PROJECT** W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE  
**SITE** Blackstone/Crane Lake Road, Site 44-383  
**LOCATION** Station 10+034 (Blackstone/Crane Lake Road) 5.7m Rt.  
**DATE** February 24, 1998

**OUR PROJECT NO.** 97TF088A  
**ENGINEER** M. R. Anderson  
**TECHNICIAN** M. Rapsey

DEPTH	DESCRIPTION	LEGEND	SAMPLE NUMBER	PENETRATION T S F				WATER CONTENT %			GROUNDWATER OBSERVATIONS AND REMARKS
ELEV	GROUND ELEVATION			1	2	3	4	10	20	30	
0	237.01										
0.15	236.86	TOPSOIL : High organic peaty topsoil with numerous roots									
		SAND : Compact, rusty brown, silty fine to coarse sand, some gravel, occasional cobbles and boulders, damp									
1.5	1.50 235.51	becoming brownish grey									
3.0	3.00 234.01	becoming wet	1								
4.10	232.91	TEST PIT TERMINATED UPON BEDROCK AT 4.10m.									Upon completion of excavating, free water at 3.35m, sidewalls standing near vertical.

NOTES





CHECKED BY *[Signature]*

## LOG OF BOREHOLE NO. 383-9

**N 5 015 370**  
**E 276 903**

PROJECT W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE  
SITE Blackstone/Crane Lake Road, Site 44-383  
LOCATION Station 10+058 (Blackstone/Crane Lake Road) 5.7m Lt.  
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A  
ENGINEER M. R. Anderson  
TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES			SHEAR STRENGTH $C_u$ ▲				LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_p$ WATER CONTENT $W$			GROUNDWATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N = VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST ●				WATER CONTENT %			
							BLOWS/0.3M				WATER CONTENT %			
0	GROUND ELEVATION 235.55						20	40	60	80	10	20	30	Upon completion of augering, no free water, no cave.
0.30	<b>SAND AND GRAVEL FILL</b> : Dark brown sand and gravel		235											
0.75														
1.27	<b>SAND FILL</b> : Brown, fine to coarse sand, some silt		234											
1.5	<b>SILT</b> : Brown, fine sandy silt, trace of clay													
3.0	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 1.27m. BEDROCK ASSUMED.													
4.5														
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

NOTES:



CHECKED BY: *MAA*

## LOG OF TEST PIT NO. 383-10

N 5 016 382  
E 276 912

**PROJECT** W.P. 408-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE  
**SITE** Blackstone/Crane Lake Road, Site 44-383  
**LOCATION** Station 10+054 (Blackstone/Crane Lake Road) 5.7m Rt.  
**DATE** February 24, 1998

**OUR PROJECT NO.** 97TF088A  
**ENGINEER** M. R. Anderson  
**TECHNICIAN** M. Rapsey

DEPTH	DESCRIPTION	LEGEND	SAMPLE NUMBER	PENETRATION T S F				WATER CONTENT %			GROUNDWATER OBSERVATIONS AND REMARKS
ELEV	GROUND ELEVATION			1	2	3	4	10	20	30	
0	236.50										
0.20	<u>TOPSOIL</u> : High organic peaty topsoil										
236.30	<u>SAND</u> : Compact, rusty brown, fine to coarse sand, some silt and gravel, occasional cobbles and boulders to 900mm diameter, damp										
1.5											
2.15											
234.35	TEST PIT TERMINATED UPON BEDROCK AT 2.15m.										Upon completion of excavating, no free water, sidewalls standing near vertical.
3.0											

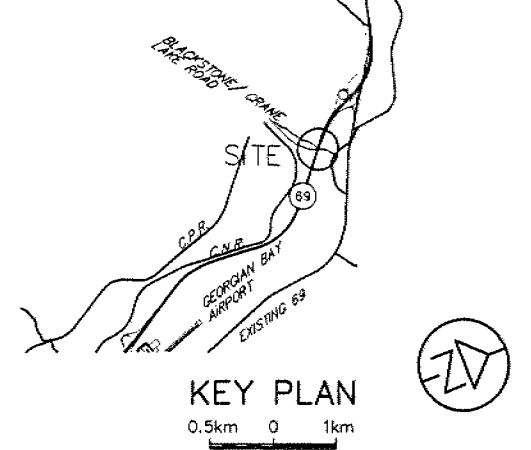
NOTES

CHECKED BY *hmt*

# METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

DISTRICT MUNICIPALITY OF PARRY SOUND  
GEOG TWP FOLEY



BOREHOLE	NORTHING	EASTING	ELEVATION
BH 383-1	N 5 016 266	E 276 862	237.55
BH 383-2	N 5 016 258	E 276 871	237.52
BH 383-3	N 5 016 284	E 276 870	237.55
BH 383-4	N 5 016 276	E 276 879	237.55
BH 383-5	N 5 016 318	E 276 883	237.09
BH 383-6	N 5 016 310	E 276 892	238.04
BH 383-7	N 5 016 351	E 276 896	235.86
BH 383-8	N 5 016 344	E 276 905	237.01
BH 383-9	N 5 016 370	E 276 903	235.55
BH 383-10	N 5 016 362	E 276 912	236.50

## LEGEND

- BOREHOLE
- BOREHOLE & ROCK CORE
- TEST PIT
- OBSERVED WATER LEVEL (DURING OR UPON COMPLETION OF DRILLING)

## NOTE

- REFER TO LOG OF BOREHOLE SHEETS FOR DETAILED SUBSURFACE CONDITIONS.
- THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES, THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.

PROPOSED CROSSING  
AT  
BLACKSTONE/ CRANE LAKE ROAD  
AND  
PROPOSED KING'S HIGHWAY 69

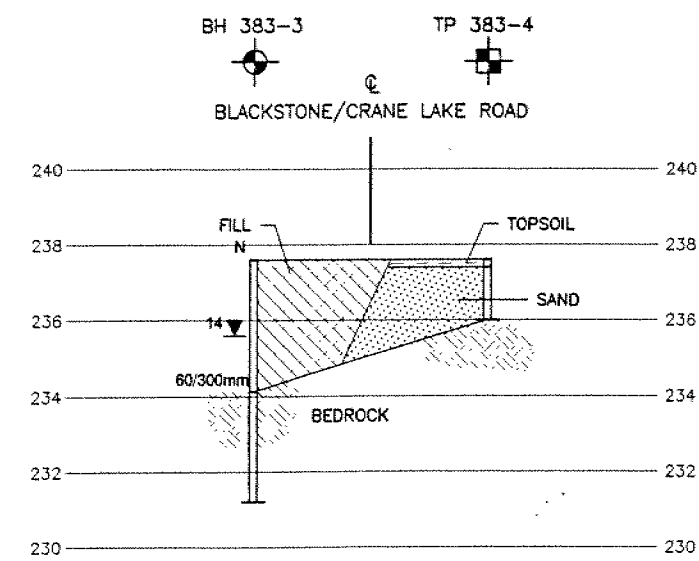
LOT 15 DISTRICT MUNICIPALITY OF PARRY SOUND CON 2  
GEOG TWP FOLEY TWP OF GEORGIAN BAY

SCALE	DISTRICT	REGION
AS SHOWN	52 HUNTSVILLE	NORTHERN
WP/WO 408-97-01	PLAN B-451-69-017	
SURVEY 97 12	PLAN 97 12	
SITE 44-383	PLAN E-451-69-054	

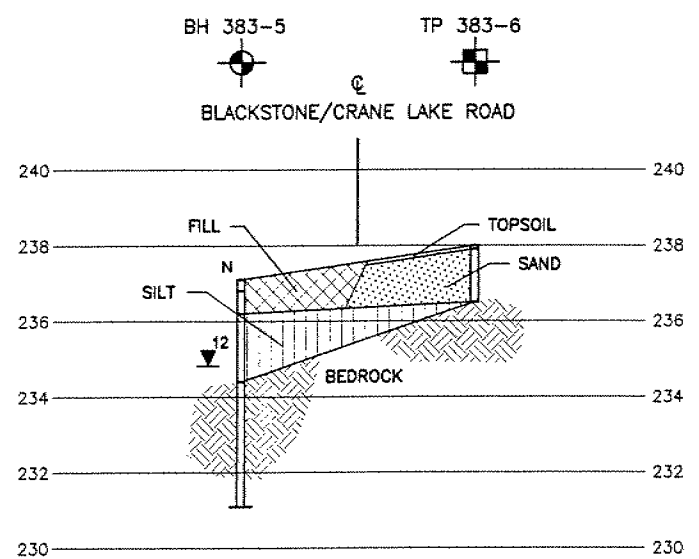
**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS  
45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C8

DRAWN	CB	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED	MRA	AUGUST 1999	AS SHOWN	97TF088B	1
APPROVED	DWK				

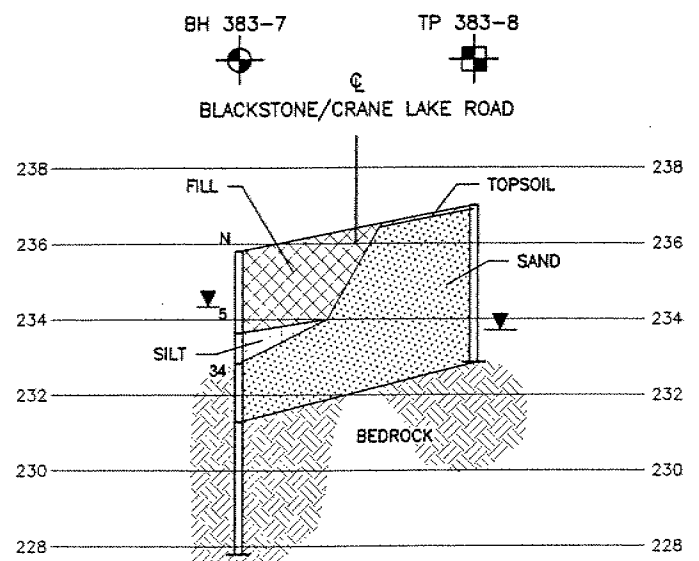
BOREHOLE LOCATION PLAN  
AND SOIL PROFILES



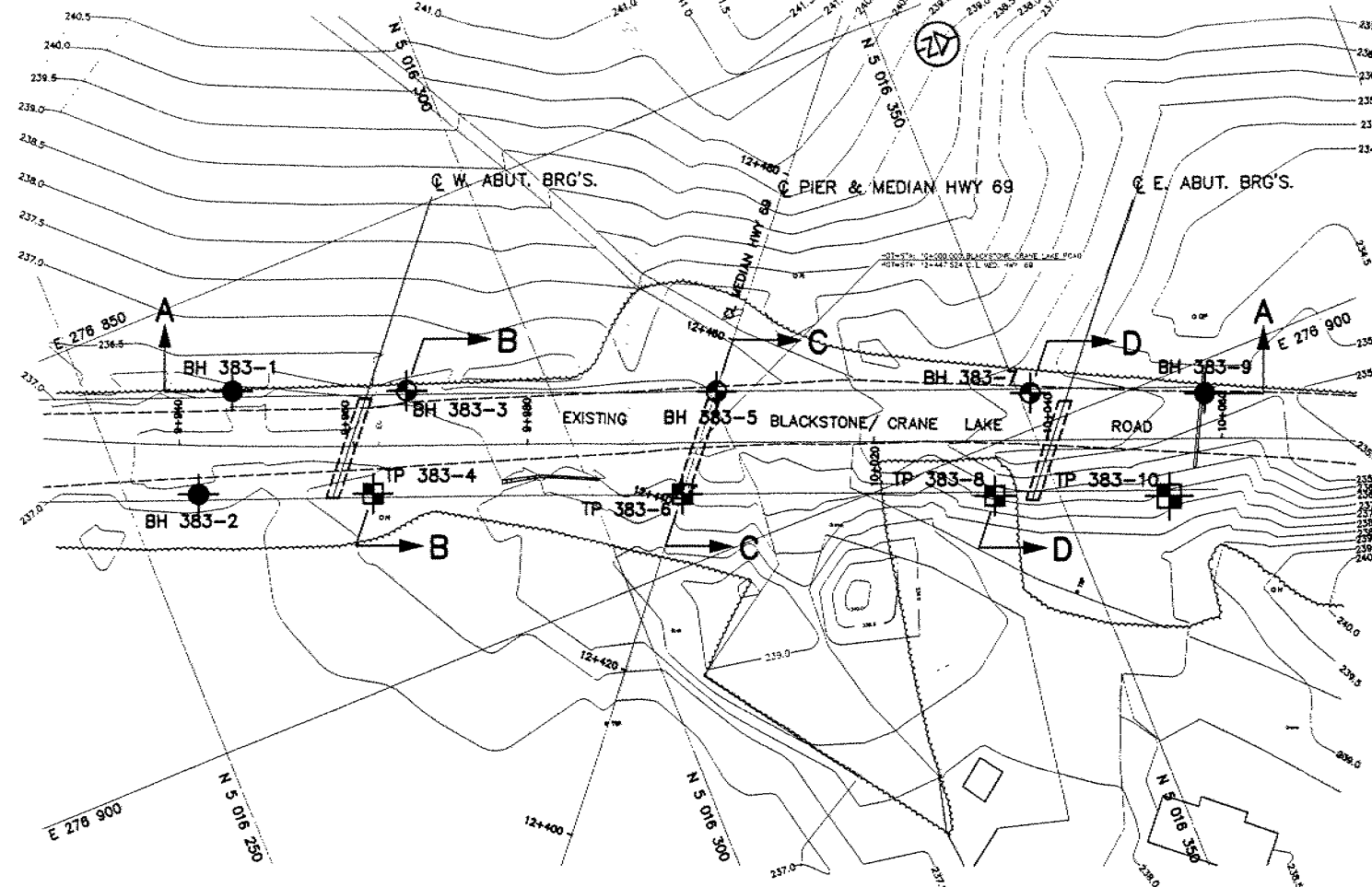
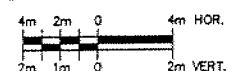
SECTION B-B



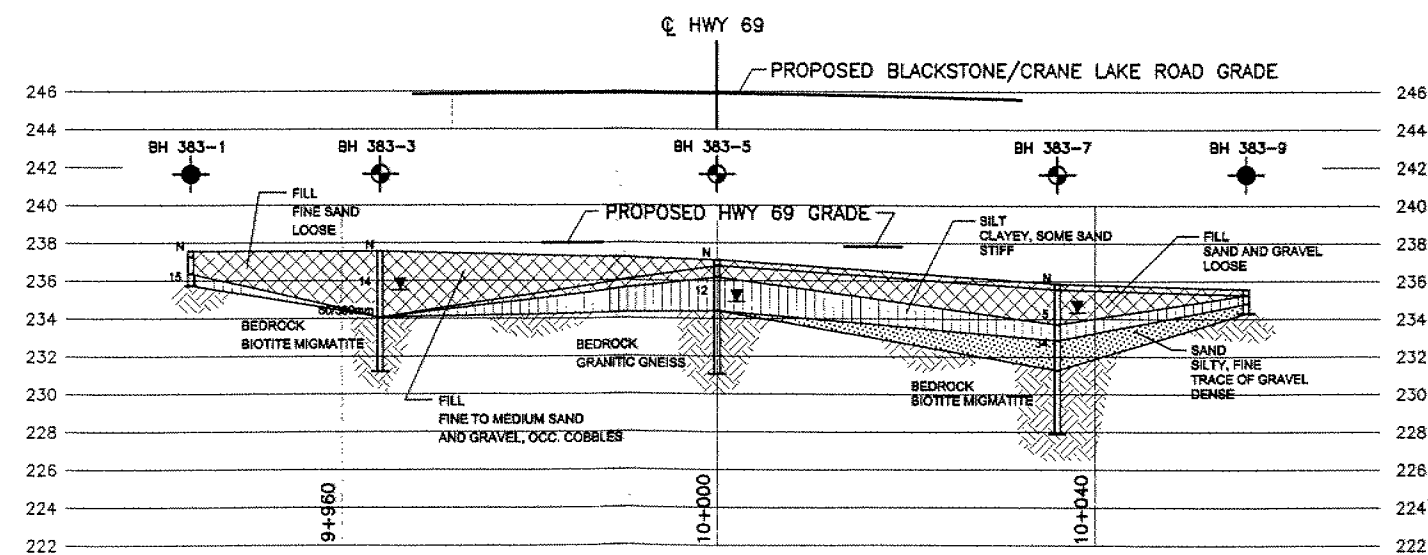
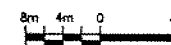
SECTION C-C



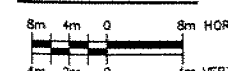
SECTION D-D



PLAN



SECTION A-A



**FOUNDATION DESIGN REPORT  
FOR  
BLACKSTONE/CRANE LAKE ROAD UNDERPASS  
W.P. 408-97-01  
G.W.P. 290-97-00, SITE 44-383  
HIGHWAY 69, DISTRICT 52  
HUNTSVILLE, ONTARIO**

Distribution:

13 cc: Highway 69 Joint Venture c/o McCormick Rankin Corporation for distribution to MTO  
2 cc: Highway 69 Joint Venture c/o McCormick Rankin Corporation  
1 cc: PML Hamilton  
1 cc: PML Toronto  
1 cc: PML Barrie

Job No. 97TF088A  
Geocres No. 31E-131

August, 1999

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**FOUNDATION DESIGN REPORT**  
For  
Blackstone/Crane Lake Road Underpass  
W.P. 408-97-01  
G.W.P. 290-97-00, Site 44-383  
Highway 69, District 52, Huntsville

---

**INTRODUCTION**

This report provides geotechnical comments and recommendations regarding design and construction of foundations, abutments and approaches at the proposed Highway 69 underpass at Blackstone/Crane Lake Road.

Construction of a two span underpass structure is planned. At the underpass location, the proposed four-lane Highway 69 will be constructed near existing grade (approximate elevation 238). Road grades on Blackstone/Crane Lake Road over the structure will be near elevation 245.2 to 245.8, some 8 m above existing grade (based on General Arrangement drawing dated November 1998 and existing ground surface elevations determined at borehole locations).

The subsurface stratigraphy revealed at the bridge site generally comprised a surficial layer of fill or topsoil overlying sand/silt, mantling bedrock at depths of 1.3 to 4.6 m. The fill mantled bedrock at one location.

**FOUNDATIONS**

**Integral Abutments on Piles**

The preliminary profile drawings indicate that road grades along Blackstone/Crane Lake Road at the underpass location will be some 8 m above existing grade. Consideration may therefore be given to construction of integral abutments supported on steel H-piles at the east and west abutments. A pile foundation system is not considered feasible at the centre pier due to the shallow depth to bedrock.

The H-piles should be driven to refusal on bedrock anticipated at the following elevations:

Location	Bedrock/Inferred Bedrock Elevation
West Abutment, North End	234.1
West Abutment, South End	236.0
East Abutment, North End	231.3
East Abutment, South End	232.9

Factored axial resistances at the ultimate limit state for selected pile sections are presented below.

H-Pile Section	Factored Capacity at ULS (kN)
HP 310 x 79	1450
HP 310 x 110	2000

The resistance at serviceability limit states 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 loading required to produce deformation of the pile will be much larger than the factored capacity at ULS.

The type of equipment required to drive the piles will be somewhat dictated by the design capacity. In general, the piles should be driven to practical refusal using a hammer which transfers at least 40 KJ of energy to the pile. Since the piles will set on hard rock, a specific set for this project is not provided.

The installation operations should be inspected on a full-time basis by qualified geotechnical personnel to confirm the toe elevation, driving resistance, alignment, plumbness, uniformity of set, and quality of splices.

The pile tip should be reinforced (OPSD 3301) to minimize the potential for damage when setting into bedrock. Rock points should be used to minimize the potential for sliding of the pile tip along sloping bedrock surfaces.

Pile caps should be provided with the normal 1.8 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

The soil adjacent to the upper portion of the piles is expected to comprise well compacted approach fill placed directly on bedrock, native sand/silt or sand and gravel fill. To accommodate movement of the integral abutments, it is recommended that pre-augered holes filled with loose sand be provided around the piles. The pre-augered holes should be 600 mm diameter and extend 3.0 m below the bottom of the abutment. Refer to MTO Report SO-96-01 for further details. The gradation of the loose sand should be as specified on Table I.

The coefficient of horizontal subgrade reaction,  $k_s$ , for rock fill and Granular "B" backfill may be computed using the following equation:

$$k_s = n_h z/b$$

$$\begin{aligned} \text{where } z &= \text{depth (m)} \\ b &= \text{pile width (m)} \end{aligned}$$

The recommended values for  $n_h$  in  $\text{kN/m}^3$  are as follows:

	Above Groundwater	Below Groundwater
Granular "B"	12,000	8,000
Rock Fill	15,000	9,000

### Spread Footings

Based on the borehole information, it is considered that the structure may be supported on conventional spread footings founded on bedrock. Foundations bearing on the sound bedrock at elevations 231.3 to 236.5 may be designed using a factored bearing resistance of 10,000 kPa at the ultimate limit state.

The capacity at serviceability limit states normally allows for 25 mm of compression of the foundation and founding medium. Considering the bedrock to be non-yielding, the design is not expected to be governed by settlement since the loading required to produce deformation will be much larger than the factored capacity at ULS.

The bedrock surface along the foundations slopes up from north to south at an inclination of 8 to 10 degrees. Mass concrete should be placed to provide a level founding surface.

Alternatively, spread footings could be constructed on structural fill placed in the approaches. The structural fill should comprise OPSS Granular "A" material placed in maximum 200 mm thick lifts, compacted to 100% standard Proctor maximum dry density, and extended laterally to a line inclined outwards at 1:1 (H:V) originating at least 1 m from the top of footing. This scheme is illustrated on Figure 1. The topsoil and existing fill should be removed prior to placement of the structural fill.

The recommended bearing resistances for footings constructed on structural fill are as follows:

Assumed Footing Width (m)	Factored Bearing Resistance at ULS (kPa)	Bearing Resistance at SLS (kPa)
2	920	250
3	1110	250

The recommended capacity at SLS allows for 25 mm of total settlement; differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.8 m was assumed for computation of the ULS capacities.

All footings subject to frost action should be provided with the normal 1.8 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover. Footings bearing on sound bedrock should not require protection from frost.

Prior to placement of structural concrete, all foundation excavations should be examined by qualified geotechnical personnel to verify the competency of the founding surface.

## **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,  $p$ , may be computed using the equivalent fluid pressures presented in Section 6-7.4 of the Ontario Highway Bridge Design Code (OHBDC, 3<sup>rd</sup> Edition, 1991) or employing the following equation, assuming a triangular pressure distribution:

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

where       $K$  = coefficient of lateral earth pressure

$\gamma$  = unit weight of free-draining  
granular material (kN/m<sup>3</sup>)

$h$  = depth below final grade (m)

$q$  = surcharge load (kPa), if present

Free-draining granular material or rock fill should be used as backfill behind the wall. The following parameters are recommended for design:

	Granular "A"	Granular "B"	Rock Fill
Angle of Internal Friction (degrees)	35	32	35
Unit Weight (kN/m <sup>3</sup> )	22.8	21.2	18.0
Active Earth Pressure Coefficient ( $K_a$ )	0.27	0.31	0.27
At Rest Earth Pressure Coefficient ( $K_o$ )	0.43	0.47	0.43
Passive Earth Pressure Coefficient ( $K_p$ )	3.69	3.25	3.69

Refer to MTO Report SO-96-01 for procedures to determine the earth pressure coefficient to be employed to design 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.

A weeping tile system and/or weeping holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile 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.

If spread footings are employed, the horizontal force will be resisted in part by the friction force developed between the underside of footing and the bedrock/structural fill. Unfactored friction factors of 0.6 and 0.45 are recommended for footings on bedrock and granular fill, respectively. A value of 0.7 may be used if the bedrock surface is roughened (asperity height of at least 25 mm).

The lateral resistance of footings founded on bedrock could be increased by installing anchors into the bedrock. The increased lateral resistance will be provided by the shear strength of the steel dowels, the horizontal component of tensile forces developed in any inclined anchors, and/or increased frictional resistance between the footing and rock if the anchors are prestressed to increase the vertical pressure.

A factored rock-grout bond stress of 1.4 MPa at the ultimate limit state (resistance factor of 0.4 applied, minimum 35 MPa grout) is recommended for design. The anchors should extend a minimum 30 bar diameters into sound bedrock and be spaced a distance of at least four times the diameter of the anchor. The total capacity of a group of closely spaced anchors may be less than the summed capacities of the individual anchors; the impact of dowel interaction should be assessed if the spacing is less than one-fifth of the anchor length.

#### **APPROACH FILL**

Backfilling adjacent to the structure should be carried out in conformance with Ontario Provincial Standards specifications for granular or rock backfill.

The embankments should be constructed in accordance with OPSD 200.01, 200.02, 201.01, 201.02 and 202.010. The side slopes of approach fills should be inclined no steeper than 2:1 (H:V) for earth fill and 1.25:1 for rock fill. For high rock fill embankments, provide 2.0 m wide berms so that no

uninterrupted rock slope is greater than 6 m high in accordance with the Northern Region Pavement Design Practices and Guidelines.

No settlement or bearing capacity problems due to placing fill on the bedrock or inorganic native sand/silt are anticipated. If footings are constructed on structural fill, it is recommended that the existing fill, particularly in the vicinity of boreholes 383-3 and 7, be removed prior to placement of the approach fill. Topsoil and other deleterious material should also be stripped.

### **EXCAVATION AND GROUNDWATER CONTROL**

Excavation for construction of footings, if employed, is expected to be carried out primarily within the existing fill, native sand/silt and approach fill. Excavation is expected to be relatively straightforward using open cut procedures.

The in situ materials are classified as Type 3 soils according to Occupational Health and Safety Act criteria. Temporary cut slopes inclined at 1 horizontal to 1 vertical should generally be stable. Flatter sideslopes may be required if excessively soft/wet materials or concentrated seepage zones are encountered.

Footing construction on bedrock will require excavating some 1.5 m below the observed water levels, locally 3.0 m at borehole 383-7. Positive groundwater control measures such as sumps or wellpoints will be required to handle groundwater entering the excavations.

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

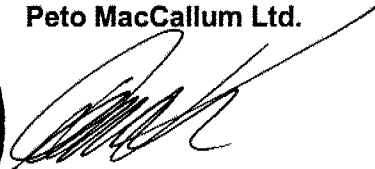
**CLOSURE**

This report was written by M.R. Anderson, P.Eng., Project Engineer and reviewed by D.W. Kerr, P.Eng., Manager of Geotechnical and Geo-Environmental Services, Hamilton.

Yours very truly

**Peto MacCallum Ltd.**



  
Dennis W. Kerr, M.Eng., P.Eng.  
Manager Geotechnical and  
Geo-Environmental Services

MRA:mma



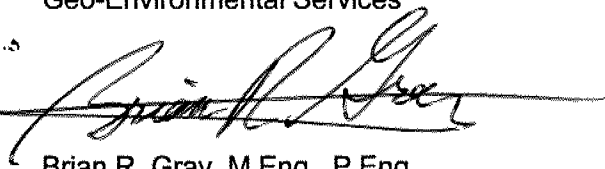
  
Brian R. Gray, M.Eng., P.Eng.  
Vice-President  
Geotechnical and  
Geo-Environmental Services



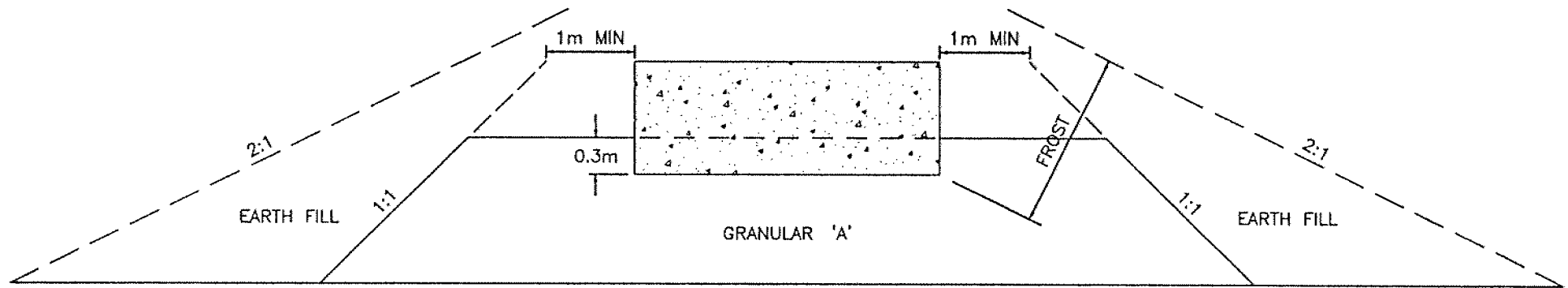
TABLE I

**Gradation Specification for Sand Fill in  
Pre-Augered Holes at Integral Abutments**

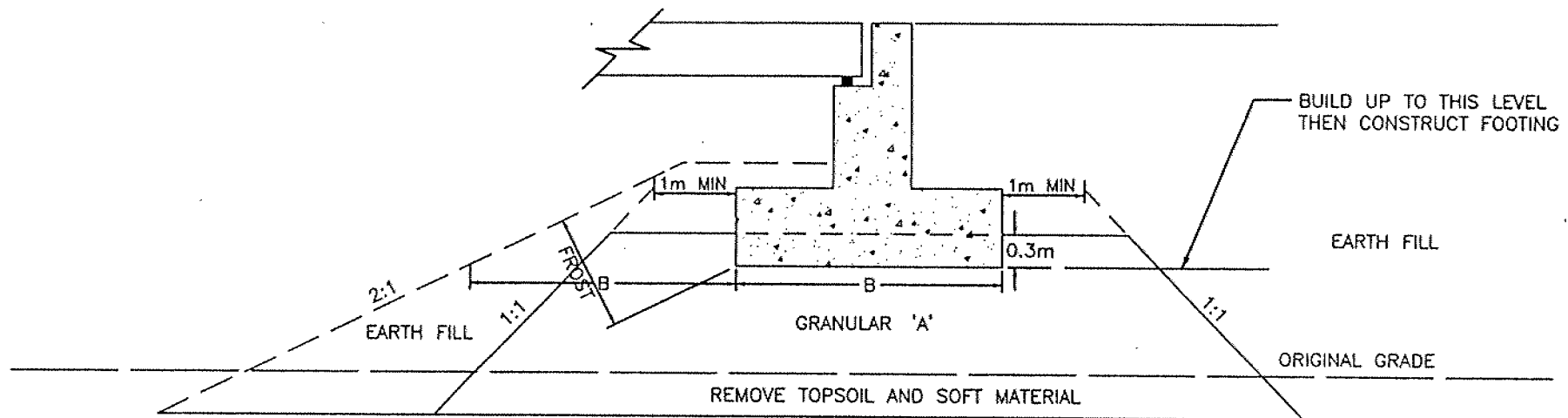
MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100
600 µm	#30	80 - 100
425 µm	#40	40 - 80
250 µm	#60	5 - 25
150 µm	#100	0 - 6

From MTO Report S0-96-01, Revision 1 - July, 1996.

## ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



### CROSS SECTION



### LONGITUDINAL SECTION

#### NOTES

1. REMOVE TOPSOIL AND/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
3. CONSTRUCT CONCRETE FOOTING
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED
5. REFER TO TEXT OF REPORT FOR FROST DEPTH

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS

45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C8  
Tel: (905) 561-2231 Fax (905) 561-8383

DATE	SCALE	JOB NO.	FIGURE NO.
MAR. 1998	NTS	-	1



# memorandum

---

To: Mike Pearsall, P. Eng.  
Senior Project Manager  
Planning & Design Section  
Northern Region

1999 10 20

From: Pavements and Foundation Section  
Room 232, Central Building  
Downsview, Ontario

Re: Final Foundation Investigation Reports  
Hwy 69 - Four Laning From Tower Rd. Northerly 26.5 km to North of Hwy 141  
Blackstone/Crane Lake Underpass, W.P. 408-97-01, Site 44-383  
CNR Overhead, W.P. 405/406-97-01, Site 44-381 N&S  
Healey Lake Road Underpass, W.P. 400-97-01, Site 44-377  
Tower Road Underpass, W.P. 399-97-00, Site 44-321  
Airport Road Underpass, W.P. 407-97-01, Site 44-382  
G.W.P. 290-97-00, Hwy 69, District 52, Huntsville

We have conceptually reviewed the final Foundation reports for the above projects, dated August 1999 produced by Peto MacCallum Ltd. Consulting Engineers for McCormick Rankin Corporation to determine the consultant's performance in providing the deliverables as would be required by MTO for similar consultant assignments. The accuracy of the subsurface information and the adequacy and technical aspects of the recommendations remain the responsibility of the consultant. The Ministry assumes no responsibility or liability for these aspects of the reports. These aspects will be reviewed in order to assess the consultant's performance in this assignment upon implementation of the recommendation in the design and upon review of the performance of the foundations for the completed project.

Most of the comments made in the preliminary foundation report review are incorporated in the final report. However, following are our comments:

Healey Lake Road Underpass, Site 44-377; Section II, Page 8, Second Paragraph: The phrase "earth rock" should be changed to "earth fill".

Blackstone/Crane Lake Underpass, Site 44-383: It should be noted in the report that for excavation below water table, an NSSP for dewatering should be included in the contract.

CNR Overhead, Site 44-381 N&S: The following comments were made in our previous memo dated May 26, 1999, but not incorporated in the final report:

“at this location piles will be driven through the engineered fill. It should be specified in the Foundation report that the engineered fill will be constructed prior to pile driving. In order to drive the piles through the engineered fill, the fill should be constructed of granular material. It should also be specified that the particle size of the granular fill should not be larger than 75 mm for H-piles and 50 mm for pipe piles driving.”

If you have any other questions, please advise.

A handwritten signature in black ink, appearing to read 'K. Ahmad', with a large, stylized loop at the beginning.

K. Ahmad, P. Eng.  
Foundation Engineer  
For  
T.C. Kim, P. Eng.  
Senior Foundation Engineer

cc: T. Kazmierowski

file: c:\ken\2909700.mik.doc



# memorandum

---

To: Bruce Sedgwick, P. Eng.  
Senior Project Engineer  
Planning and Design Section  
Northern Region

1999 05 26

From: Pavements and Foundations Section  
Room 232, Central Building  
Downsview, Ontario

Re: Draft Foundation Investigation Reports  
Highway 69 - Four Laning  
From Tower Road Northerly 26.5 km to 2 km North of Hwy 141  
Blackstone/Crane Lake Underpass, W.P. 408-97-01, Site 44-383  
CNR Overhead, W.P. 405/406-97-01, Site 44-381 N&S  
Healey Lake Road Underpass, W.P. 400-97-01, Site 44-377  
Tower Road Underpass, W.P. 399-97-00, Site 44-321  
Airport Road Underpass, W.P. 407-97-01, Site 44-382  
G.W.P. 290-97-00, Hwy 69, District 52, Huntsville

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### **General Comments for all projects**

1. MTO has established the frost depth for the Huntsville District as 1.8m. The frost depth in all the Foundation reports for this project should be specified as 1.8 m.
2. Recommendation should be given for the side slopes of the approach fills.
3. The Key Plan, northing and easting and the stations shown on the plan are very small. After the drawings are reproduced to include in the contract package, they would not be legible.
4. The cross sections are very small (some of them are thumb size) and should be enlarged. These cross sections will be included in the contract package. When they are reproduced for the contract package, they would not be legible. The plan is produced in 1:500 scale. The cross sections are normally 100 percent larger than the plans to show the details. But in the report the cross sections are 50 percent reduced. Ideally the cross sections should be in true scale, i.e. same horizontal and vertical scales. If the true scale is not feasible, then the ratio of horizontal and vertical scales should be 2. The ratio of the horizontal and vertical scale in the foundation reports are 5. The cross sections, therefore, are very distorted. All the cross sections do not have to fit on one drawing. Cross sections can be produced on more than one drawings. A sample copy of the standard drawing can be obtained from the Pavements and Foundations office.
5. A bar scale, similar to the one provided on the plan should also be provided on the cross sections
6. The Pavements and Foundations Section has assigned Geocres Numbers for these projects. The Consultant should provide the Geocres numbers on the Final Reports. The Geocres number shall be shown on the lower left corner of the Title Page of the Foundation reports.

### **Blackstone/Crane Lake Road, Site 44-383**

1. Page 2 (Section II): The recommended pile resistance on bedrock is conservative and should be revised. Due to the high grade steel of the H-Piles, The pile resistance on sound bedrock has been increased. For example the pile resistance at ULS for HP 310X110 piles is 2000 kN. The term "Pile Capacity" is not used any more in OHBDC. The Consultant should refer to the OHBDC 91, 3<sup>rd</sup> Edition.
2. The Geocres Number for this project is 31E-131.

### **CNR Overhead, Site 44-381, N&S**

- 1 Page 2 (Section II): We understand that at this location piles will be driven through the engineered fill. It should be specified in the Foundation report that the engineered fill will be constructed prior to pile driving. In order to drive the piles through the engineered fill, the fill should be constructed of granular material. It should also be specified that the particle size of the granular fill should not be larger than 75 mm for H-piles and 50 mm for pipe piles.
- 2 Page 2 (Section II): The recommended pile resistance on bedrock is conservative and should be revised. Due to the high grade steel of the H-Piles, The pile resistance on sound bedrock has been increased. For example the pile resistance at ULS for HP 310X110 piles is 2000 kN. The term "Pile Capacity" is not used any more in OHBDC. The Consultant should refer to the OHBDC 91, 3<sup>rd</sup> Edition.
- 3 The Geocres Number for this project is 31E-132.

### **Healey Lake Rd. Underpass, Site 44-377**

- 1 Page 1 (Section II): The proposed abutment and pier locations are underlain by peat. The report did recommend removing peat from these locations. It should also be mentioned in the report that the engineered fill should be constructed prior to pile driving. In order to drive the piles through the engineered fill, the fill should be constructed of granular material. It should be specified that the particle size of the granular fill should not be larger than 75 mm for H-piles and 50 mm for pipe piles.
- 2 Page 2 (Section II): The recommended pile resistance on bedrock is conservative and should be revised. Due to the high grade steel of the H-Piles, The pile resistance on sound bedrock has been increased. For example the pile resistance at ULS for HP 310X110 piles is 2000 kN. The term "Pile Capacity" is not used any more in OHBDC. The Consultant should refer to the OHBDC 91, 3<sup>rd</sup> Edition.
- 3 The Geocres Number for this project is 31E-133.

### **Tower Road Underpass, Site 44-321**

- 1 The Geocres Number for this project is 31E-134.

**Airport Road Underpass, Site 44-382**

1 The Geocres Number for this project is 31E-135.

If you have any questions, please advise.

A handwritten signature in black ink, appearing to read 'K. Ahmad', with a large, stylized initial 'K'.

K. Ahmad, P. Eng  
Foundation Engineer

For

T.C. Kim, P. Eng.  
Senior Foundation Engineer

cc: P. Furst  
W. Roy  
D. Yeo  
I. Hussain  
T. Kazmierowski

file: c:\data\wpwin60\2909700.brc3.wpd