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GEOCRES No. 31E-133

DIST. 52 REGION

W.P. No. 400-97-01
GWP: 290-97-00

CONT. No.

W. O. No.

STR. SITE No. 44-377

HWY. No. 69

LOCATION Healey Lake Rd. Underpass

No of PAGES - 1

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT
FOR
HEALEY LAKE ROAD UNDERPASS
W.P. 400-97-01
G.W.P. 290-97-00, SITE 44-377
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

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August, 1999

PetoMacCallum Ltd.
CONSULTING ENGINEERS

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT

For
Healey Lake Road Underpass
W.P. 400-97-01
G.W.P. 290-97-00, Site 44-377
Highway 69, District 52, Huntsville

INTRODUCTION

This report summarizes the results of the foundation investigation carried out for construction of the Highway 69 underpass at Healey Lake Road (Station 10+302 Highway 69 chainage).

The report pertains to the proposed bridge structure and approaches within about 20 m of the abutments, between approximate stations 9+945 and 10+055, Healey Lake Road chainage.

SITE DESCRIPTION

The site is located about 5 km north of MacTier and about 2 km west of the existing Highway 69 alignment. The proposed structure will carry Healey Lake Road traffic over the new four-lane section of Highway 69. At the underpass, the structure will run roughly east-west.

Healey Lake Road passes through a low-lying swampy area at the proposed structure location. The road grade rises to the east and west. Bedrock outcrops immediately to the east of the northeast corner of the structure. The surrounding lands are generally wooded. The ground surface 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 area 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.

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INVESTIGATION PROCEDURES

The fieldwork was carried out during the period February 24 to March 2, 1998 and comprised 10 boreholes drilled at the locations shown on Drawing 1.

The boreholes were drilled to refusal on bedrock/inferred bedrock at depths of 0.0 to 12.5 m. Three of the boreholes were extended an additional 2.7 to 3.0 m into bedrock using NQ rock coring equipment.

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

Representative samples of the overburden were recovered at frequent depth intervals 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.

All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determinations. Grain size distribution analyses were carried out on selected samples. 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 sand fill overlying bedrock. Peat and bouldery material were revealed in/below the fill in three boreholes, and native silt/sand/clay

were encountered below the fill/peat at the centre pier. The strata encountered are summarized below.

Asphaltic Concrete

A 50 mm thick layer of asphaltic concrete was encountered surficially in boreholes 377-2, 4, 8 and 10 drilled on the existing Healey Lake Road.

Sand Fill

Sand fill was encountered either surficially or below the asphaltic concrete layer in all boreholes except borehole 377-9. The fill was generally loose to very loose indicating it was poorly compacted. The sand was typically fine to medium-grained; the results of grain size distribution tests conducted on the sand are presented on Figure 1. A layer of sand and gravel was revealed above/within the sand in boreholes 377-4, 8 and 10. The lower 1.8 to 2.6 m of fill in boreholes 377-5 and 6 was bouldery.

The thickness of the fill was greatest at the centre pier, extending to depths of 7.0 and 9.3 m in boreholes 377-5 and 6, respectively. It decreased to 2.2 and 2.5 m in boreholes 377-1 and 2 at the west limit of the study area, and 0.6 m in borehole 377-10 at the east limit. The fill mantled bedrock in boreholes 377-1, 2, 3, 7 and 10.

Peat

A 1.2 to 1.8 m thick deposit of fine granular amorphous peat was encountered below the fill in boreholes 377-4, 5 and 8. Moisture contents of 52 and 344% were measured in this material. The peat was penetrated at 8.5 m depth in borehole 377-5 and mantled bedrock at 6.3 and 2.4 m depth in boreholes 377-4 and 8, respectively.

Clay

Silty clay was identified below the peat in borehole 377-5 based on examination of auger cuttings. Standard penetration testing indicates the clay is stiff. The clay was penetrated at 11.4 m depth.

Silt and Sand

Dense grey silt and sand was encountered below the clay in borehole 377-5. A 200 mm thick layer of sand and gravel was encountered between the silt and sand and underlying bedrock.

Silt

Compact grey silt was contacted below the sand fill in borehole 377-6. The silt mantled inferred bedrock.

Bedrock

Bedrock or inferred bedrock was contacted surficially in borehole 377-9 and below the fill, peat and silt/sand in all other boreholes. The depth to bedrock was greatest at the centre pier, ranging from 2.2 m (elevation 244.0) in borehole 377-1 at the west limit of the study area and 0.0 m (elevation 247.3) at borehole 377-9 at the east limit, to 12.5 m (elevation 232.9) in borehole 377-5 at the centre pier.

A description of the rock cores recovered from boreholes 377-4, 5 and 7 is provided on Table I. The bedrock consists of biotite migmatite in boreholes 377-4 and 7, granitic gneiss over pegmatite in borehole 377-5. Core recovery ranged from 75 to 100% and the RQD ranged from 45 to 100%. The rock was described as poor to excellent quality, very poor quality in the upper 300 mm in borehole 377-5.

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

Borehole No.	Depth (m)	Unconfined Compressive Strength (MPa)
377-4	6.7	122.2
377-5	13.1	105.9
377-7	1.6	76.6

Groundwater

Free water was observed in boreholes 377-1 to 6 at depths of 1.0 to 2.1 m (elevations 243.8 to 244.7) during or upon completion of augering. 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, Project Engineer and reviewed by D.W. Kerr, Manager of Geotechnical and Geo-Environmental Services, Hamilton.

Yours very truly

Peto MacCallum Ltd.

REGISTERED PROFESSIONAL ENGINEER
D. W. KERR
PROVINCE OF ONTARIO

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

REGISTERED PROFESSIONAL ENGINEER
B. R. GRAY
PROVINCE OF ONTARIO

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

MRA:mma

Our Ref: 97TF088A

TABLE I

ROCK CORE DESCRIPTION
W.P. 400-97-01
GWP 290-97-00, Site No. 44-377

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
377-4	5	6.25 - 7.77	100	45	6.25 - 9.30	**BIOTITE MIGMATITE**, light grey to black heterogeneous to banded, high strength, unweathered; with occ. inclusions/layers of pink granitic gneiss, close to moderate spaced discontinuities, with few horizontal to dipping joints, rough planar with reddish oxidation on parting surface; poor to excellent quality
	6	7.77 - 9.30	100	100		
377-5	9	12.50 - 12.80	75	0*	12.50 - 12.80	**GRANITIC GNEISS**, pink fine crystalline, high strength, unweathered; very close to close spaced dipping joints, rough planar, red staining to slightly altered; very poor quality
	10	12.80 - 13.72	86	50		
	11	13.72 - 14.63	100	56		
	12	14.63 - 14.78	100	100	12.80 - 13.72	with zones of black hornblende banding, close to moderate spaced joints; fair quality
	13	14.78 - 15.24	100	100		
					13.72 - 14.60	with scaling/encrustation on joint surfaces
					14.60 - 14.93	5 mm wide dipping parting, infilled with silt; excellent quality
					14.93 - 15.24	**PEGMATITE**, pink, coarse crystalline with black hornblende layers, high strength, unweathered; close to moderate spaced discontinuities; excellent quality

RQD = Rock Quality Designation

* Low RQD due to casing disturbance

Logged by J. Wright

Our Ref: 97TF088A

TABLE I

ROCK CORE DESCRIPTION
W.P. 400-97-01
GWP 290-97-00, Site No. 44-377

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
377-7	1	0.97 - 1.57	100	46	0.97 - 3.86	**BIOTITE MIGMATITE**, black banded, high strength, unweathered; with occ. layers of pink granitic gneiss, close to moderate spaced discontinuities, occ. dipping to near vertical joints, rough planar, tight, with dark brown oxidation stains; poor to good quality
	2	1.57 - 3.10	92	53		
	3	3.10 - 3.86	100	77		

RQD = Rock Quality Designation

* Low RQD due to casing

Logged by J. Wright

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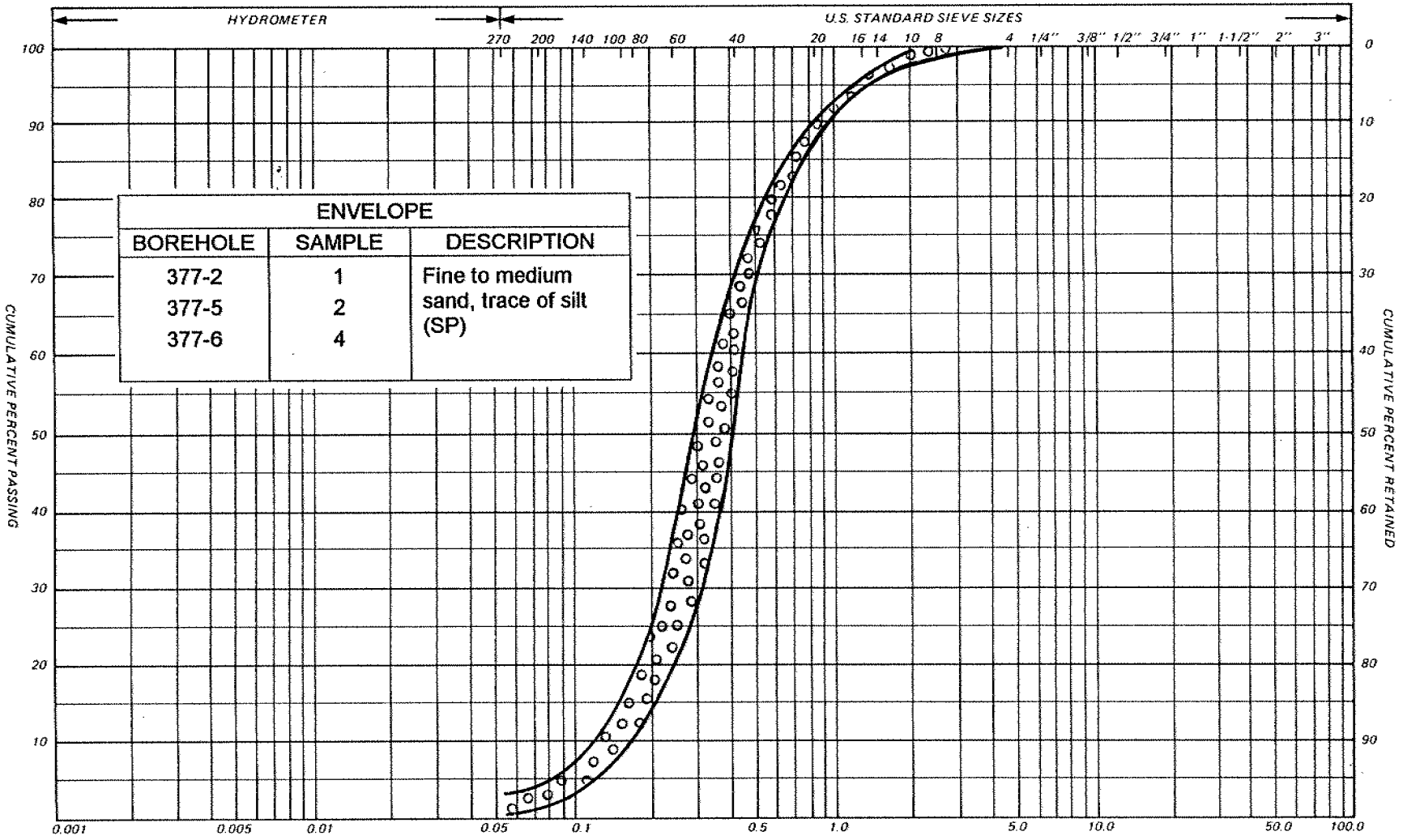
CONSULTING ENGINEERS

PARTICLE SIZE DISTRIBUTION CHART

Figure: 1
OUR PROJECT NO. 97TF088A



ENVELOPE		
BOREHOLE	SAMPLE	DESCRIPTION
377-2 377-5 377-6	1 2 4	Fine to medium sand, trace of silt (SP)

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

REMARKS _____

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 : -

CONSISTENCY	'N' BLOWS/0.3 m	c kPa	DENSENESS	'N' BLOWS/0.3 m
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.

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LOG OF BOREHOLE NO. 377-1

N 5 002 980
E 281 600

PROJECT W.P. 400-97-01 HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 9+946 (Healey Lake Road) 5.7m Lt.
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A
BORING DATE March 2, 1998
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	GROUNDWATER OBSERVATIONS AND REMARKS
	GROUND ELEVATION 246.13						
0	SAND FILL : Compact, brown fine sand, trace of silt, damp to moist		245				
1.5				1	SS	20	
2.18			244				
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 2.18m. BEDROCK ASSUMED.						Upon completion of augering, free water at 1.83m.
3.0			243				
4.5							
6.0							
7.5							
9.0							
10.5							
12.0							
13.5							
15.0							
16.5							

SHEAR STRENGTH C_u ▲
DYNAMIC CONE PENETRATION ×
STANDARD PENETRATION TEST ●
BLOWS/0.3M 20 40 60 80

LIQUID LIMIT ——— W_L
PLASTIC LIMIT ——— W_P
WATER CONTENT ——— W
W_P W W_L
WATER CONTENT % 10 20 30

NOTES:

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LOG OF BOREHOLE NO. 377-2

N 5 002 968
E 281 601

PROJECT W.P. 400-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 9+945 (Healey Lake Road) 5.7m Rt.
BORING METHOD Continuous Flight Hollow Stem Augers
BORING DATE Feb. 28, 1998
OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m N - VALUES	GROUNDWATER OBSERVATIONS AND REMARKS
	GROUND ELEVATION 245.96						
0	0.05 ASPHALTIC CONCRETE : 50mm						
	SAND FILL : Compact, brown fine to medium sand, trace of silt, damp		245				
1.5				1	SS	27	
			244				
	2.51			2	SS	26/25mm & bouncing	
3.0	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 2.51m. BEDROCK ASSUMED.		243				Upon completion of augering, free water at 2.10m.
4.5							
6.0							
7.5							
9.0							
10.5							
12.0							
13.5							
15.0							
16.5							

SHEAR STRENGTH C_u ▲
DYNAMIC CONE PENETRATION x
STANDARD PENETRATION TEST ●
BLOWS/0.3M
20 40 60 80

LIQUID LIMIT ——— W_L
PLASTIC LIMIT ——— W_P
WATER CONTENT ——— W
W_P W W_L
WATER CONTENT %
10 20 30

NOTES:

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CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 377-3

N 5 002 984
E 281 620

PROJECT W.P. 400-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 9+966 (Healey Lake Road) 5.7m Lt.
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A
BORING DATE March 2, 1998 ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m N - VALUES	GROUNDWATER OBSERVATIONS AND REMARKS
	GROUND ELEVATION 245.65						
0	SAND FILL : Compact, brown, fine to medium sand, trace of silt, wet		245				
1.5			244	1	SS	11	
2.55	Very loose		243				
3.0				2	SS	0	
			242				
4.5			241	3	SS	2*	* No Recovery
5.05	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 5.05m. BEDROCK ASSUMED.		240				Upon completion of augering, free water at 1.00m.
6.0							
7.5							
9.0							
10.5							
12.0							
13.5							
15.0							
16.5							

SHEAR STRENGTH C_u ▲
DYNAMIC CONE PENETRATION ×
STANDARD PENETRATION TEST ●
BLOWS/0.3M
20 40 60 80

LIQUID LIMIT ——— W_L
PLASTIC LIMIT ——— W_P
WATER CONTENT ——— W
W_P W W_L
WATER CONTENT %
10 20 30

NOTES:

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 377-4

N 5 002 972
E 281 621

PROJECT W.P. 400-97-01 HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 9+965 (Healey Lake Road) 5.7m Rt.
BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring
BORING DATE Feb. 28, 1998
OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	SHEAR STRENGTH C_u / DYNAMIC CONE PENETRATION x / STANDARD PENETRATION TEST • BLOWS/0.3M (20 40 60 80)	WATER CONTENT % (W_P W W_L) (10 20 30)	GROUNDWATER OBSERVATIONS AND REMARKS
0	GROUND ELEVATION 245.61							
0.05								
0.20	ASPHALTIC CONCRETE : 50mm							
	SAND AND GRAVEL FILL : Brown	245						
	SAND FILL : Loose to compact, fine to medium sand, trace of silt, saturated							
1.5		244	1	SS	11	•	⊙	
		243						
3.0			2	SS	1/450mm*			* No Recovery
		242						Free water at 1.85m after sample 2.
4.50								
4.5	PEAT : Fine amorphous granular peat, occasional lenses of fine sand, decayed woody organics	241	3	SS	3	•		
		240						
6.0			4	SS	1/150mm & bouncing			
6.25	BEDROCK : Biotite Migmatite	239	5	RC		1524 mm RUN; 100% RECOVERY; 45% RQD; 100% DRILL WATER RETURN		
7.5		238						
		237	6	RC		1524 mm RUN; 100% RECOVERY; 100% RQD; 100% DRILL WATER RETURN		
9.0								
9.30	BOREHOLE TERMINATED AT 9.30m.	236						Upon completion of augering, free water at 2.10m.
10.5								
12.0								
13.5								
15.0								
16.5								

NOTES:

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CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 377-5

N 5 002 991
E 281 654

PROJECT W.P. 400-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 10+001 (Healey Lake Road) 5.7m Lt.
BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring
OUR PROJECT 97TF088A
BORING DATE March 1, 1998
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

GROUND ELEVATION 245.37

DEPTH in METRES	DESCRIPTION	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	GROUNDWATER OBSERVATIONS AND REMARKS
0	**SAND FILL** : Loose, brown, fine to medium sand, trace of silt, saturated	245				After sample 1, free water at 1.17m.
		244				
1.5			1	SS	6*	* No Recovery
		243				
3.0		242	2	SS	5	
		241				
4.5			3	SS	1	
	5.20 becoming bouldery	240				
6.0		239	4	SS	60/300mm**	** 50 for last 150mm
	7.00 **PEAT** : Loose, black, fine amorphous granular peat, with decayed wood pieces	238				
7.5			5	SS	7	344%
	8.50 **CLAY** : Stiff, grey silty clay (from cuttings)	237				
9.0		236	6	SS	8*	* No Recovery
		235				
10.5			7	SS	13*	
	11.40	234				
12.0	**SILT AND SAND** : Dense, grey silt and fine sand, trace of gravel, wet					
	12.30	233	8	SS	45	
	12.50 becoming brown sand and gravel		9	RC		* Low RQD due to casing disturbance
	BEDROCK : Granitic Gneiss	232	10	RC		
13.5			11	RC		
		231	12	RC		
	14.95					
15.0	15.24 Pegmatite	230	13	RC		
	BOREHOLE TERMINATED AT 15.24m.					
16.5						

NUMBER	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)
9	305	75	0*	20
10	914	86	50	100
11	914	100	56	100
12	152	100	100	100
13	457	100	100	100

SHEAR STRENGTH c_u ▲
DYNAMIC CONE PENETRATION x
STANDARD PENETRATION TEST ●
BLOWS/0.3M: 20 40 60 80
LIQUID LIMIT ——— W_L
PLASTIC LIMIT ——— W_P
WATER CONTENT ——— W
W_P W W_L
WATER CONTENT %: 10 20 30

NOTES:

CHECKED BY:

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 377-6

N 5 002 979
E 281 654

PROJECT W.P. 400-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 9+999 (Healey Lake Road) 5.7m Rt.
BORING METHOD Continuous Flight Hollow Stem Augers
BORING DATE February 25 & 28, 1998
OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

GROUND ELEVATION 245.41

DEPTH in METRES	DESCRIPTION	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES
0					
0.30	SAND FILL : Dark brown, fine to medium sand, some gravel	245			
	becoming loose, brown fine to medium sand, trace to some silt, saturated	244			
1.5			1	SS	1
		243			
3.0			2	SS	5
		242			
4.5		241			
			3	SS	1
6.0		240			
			4	SS	4
6.70	becoming bouldery	239			
7.5		238			
		237			
9.0					
9.25	SILT : Compact, grey silt, saturated	236	5	SS	18
10.5		235			
		234			
11.89					
12.0	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 11.89m. BEDROCK ASSUMED.	233			
13.5					
15.0					
16.5					

SHEAR STRENGTH C_u ▲
DYNAMIC CONE PENETRATION ×
STANDARD PENETRATION TEST ●
BLOWS/0.3M
20 40 60 80

LIQUID LIMIT ——— W_L
PLASTIC LIMIT ——— W_P
WATER CONTENT ——— W
W_P W W_L
WATER CONTENT %
10 20 30

GROUNDWATER OBSERVATIONS AND REMARKS

After sample 1, free water at 1.20m.

NOTES:

CHECKED BY:

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 377-7

N 5 002 998
E 281 687

PROJECT W.P. 400-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 10+035 (Healey Lake Road) 5.7m Lt.
BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring
BORING DATE February 25, 1998
OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	SHEAR STRENGTH C_u ▲ / DYNAMIC CONE PENETRATION x / STANDARD PENETRATION TEST ● BLOWS/0.3M (20 40 60 80)	LIQUID LIMIT W_L / PLASTIC LIMIT W_P / WATER CONTENT W (W_P W W_L) WATER CONTENT % (10 20 30)	GROUNDWATER OBSERVATIONS AND REMARKS
	GROUND ELEVATION 246.62								
0	SAND FILL : Brown fine sand, some silt		246						
0.97	BEDROCK : Biotite Migmatite		245				610 / 100 / 46 / 100		* Upon completion of augering, no free water, no cave.
1.5			244				1524 / 92 / 53 / 100		
3.0			243				762 / 100 / 77 / 100		
3.86	BOREHOLE TERMINATED AT 3.86m.		242				RUN (mm) / RECOVERY (%) / RQD (%) / DRILL WATER RETURN (%)		
4.5									
6.0									
7.5									
9.0									
10.5									
12.0									
13.5									
15.0									
16.5									

NOTES:

CHECKED BY:

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 377-8

N 5 002 987
E 281 668

PROJECT W.P. 400-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 10+034 (Healey Lake Road) 5.7m Rt.
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A
BORING DATE February 24, 1998
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES
	GROUND ELEVATION 246.57					
0	0.05 ASPHALTIC CONCRETE : 50mm					
	0.60 SAND FILL : Brown, fine sand, trace of silt		246			
	1.20 SAND AND GRAVEL FILL : Brown		245	1	SS	2
1.5	PEAT : Fine amorphous granular peat, occasional lenses of fine sand					
	2.40 BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 2.40m. BEDROCK ASSUMED.		244			
3.0						
4.5						
6.0						
7.5						
9.0						
10.5						
12.0						
13.5						
15.0						
16.5						

SHEAR STRENGTH C_u ▲
DYNAMIC CONE PENETRATION x
STANDARD PENETRATION TEST ●
BLOWS/0.3M
20 40 60 80

LIQUID LIMIT ——— W_L
PLASTIC LIMIT ——— W_P
WATER CONTENT ——— W
W_P W W_L
WATER CONTENT %
10 20 30

52%

GROUNDWATER OBSERVATIONS AND REMARKS

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

NOTES:

CHECKED BY: MMA

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 377-9

N 5 003 002
E 281 707

PROJECT W.P. 400-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 10+055 (Healey Lake Road) 5.7m Lt.
BORING METHOD Continuous Flight Hollow Stem Augers
BORING DATE February 24, 1998
OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m N - VALUES	SHEAR STRENGTH C_u ▲ / DYNAMIC CONE PENETRATION x / STANDARD PENETRATION TEST ● BLOWS/0.3M (20 40 60 80)	LIQUID LIMIT W_L / PLASTIC LIMIT W_P / WATER CONTENT W (W_P W W_L) WATER CONTENT % (10 20 30)	GROUNDWATER OBSERVATIONS AND REMARKS
0	GROUND ELEVATION 247.34								
	BEDROCK AT SURFACE.		247						
1.5									
3.0									
4.5									
6.0									
7.5									
9.0									
10.5									
12.0									
13.5									
15.0									
16.5									

NOTES:

CHECKED BY:

PetoMacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 377-10

N 5 002 991
E 281 708

PROJECT W.P. 400-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Healey Lake Road Underpass, Site 44-377
LOCATION Station 10+054 (Healey Lake Road) 5.7m Rt.
BORING METHOD Continuous Flight Hollow Stem Augers
BORING DATE February 24, 1998
OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES
	GROUND ELEVATION 247.23					
0	0.05		247			
	0.23 ASPHALTIC CONCRETE : 50mm					
	0.64 SAND AND GRAVEL FILL : Brown		246			
1.5	SAND FILL : Compact, brown fine sand, trace of silt, damp					
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 0.64m. BEDROCK ASSUMED.					
3.0						
4.5						
6.0						
7.5						
9.0						
10.5						
12.0						
13.5						
15.0						
16.5						

SHEAR STRENGTH C_u ▲
DYNAMIC CONE PENETRATION x
STANDARD PENETRATION TEST ●
BLOWS/0.3M
20 40 60 80

LIQUID LIMIT ——— W_L
PLASTIC LIMIT ——— W_P
WATER CONTENT ——— W
W_P W W_L
WATER CONTENT %
10 20 30

GROUNDWATER OBSERVATIONS AND REMARKS

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

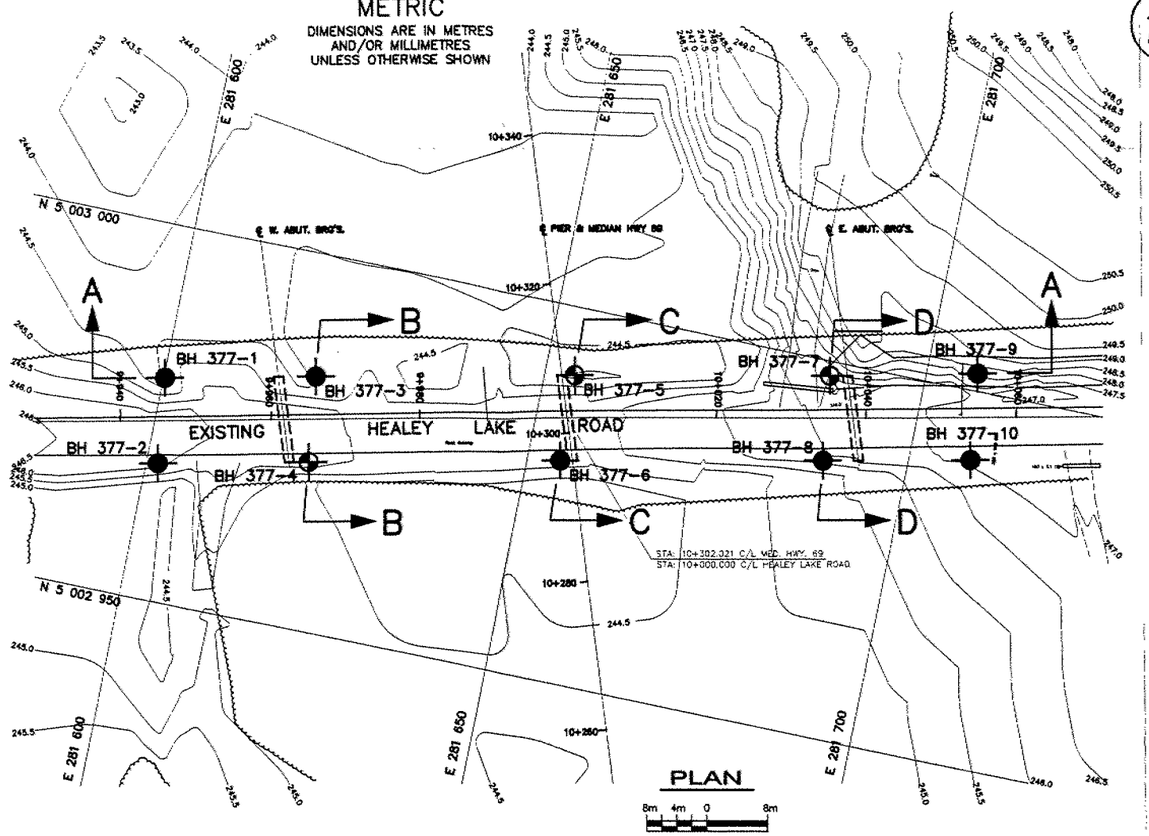
NOTES:

CHECKED BY:

PLATE No 774-69/99-0

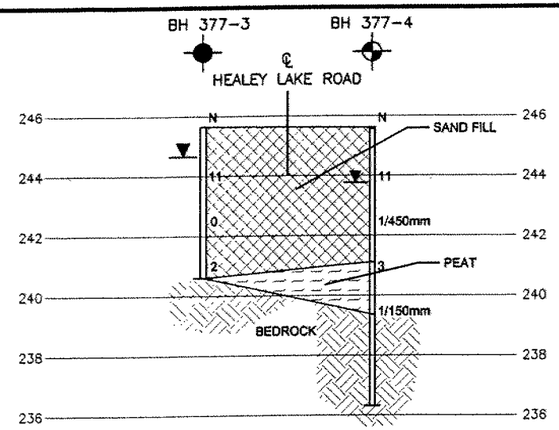
METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

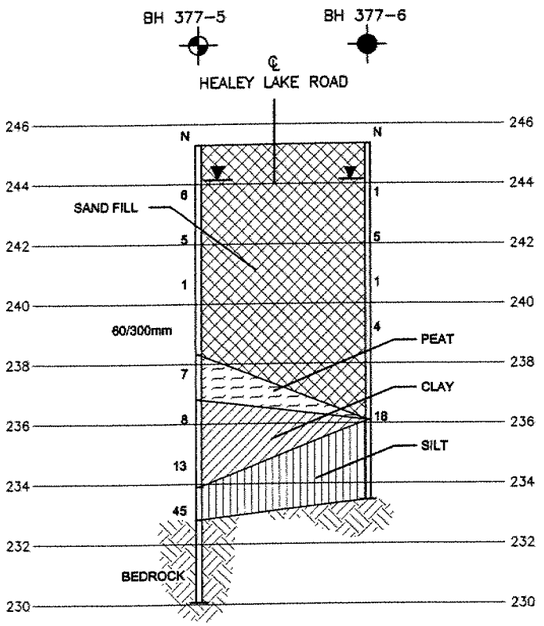


PLAN

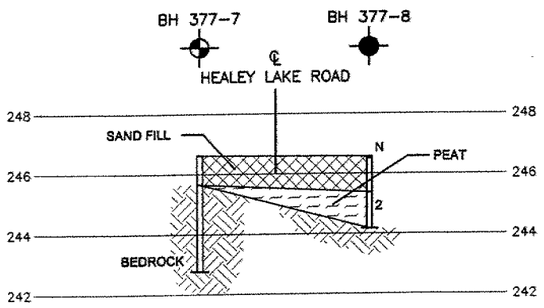
8m 4m 0 8m



SECTION B-B

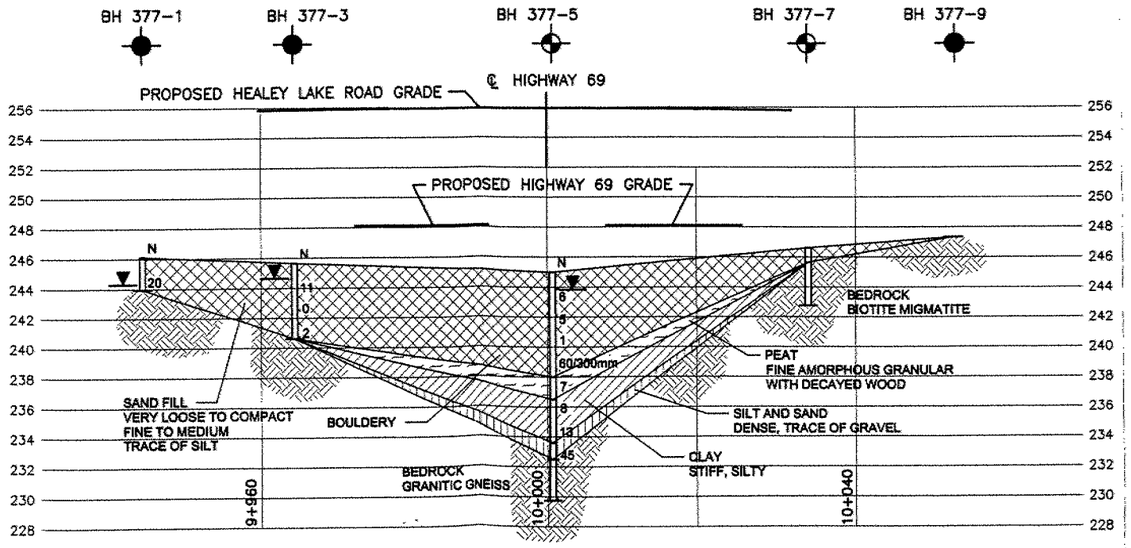


SECTION C-C



SECTION D-D

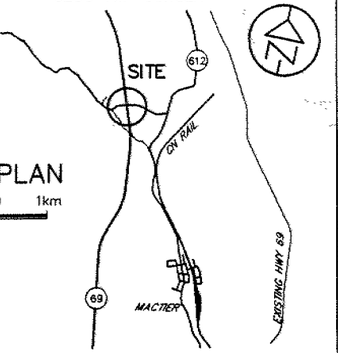
4m 2m 0 4m HOR.
2m 1m 0 2m VERT.



SECTION A-A

8m 4m 0 8m HOR.
4m 2m 0 4m VERT.

DISTRICT MUNICIPALITY OF PARRY SOUND
GEOG TWP CONGER



BOREHOLE	NORTHING	EASTING	ELEVATION
BH 377-1	N 5 002 980	E 281 600	246.13
BH 377-2	N 5 002 988	E 281 601	245.96
BH 377-3	N 5 002 984	E 281 620	245.85
BH 377-4	N 5 002 972	E 281 621	245.81
BH 377-5	N 5 002 991	E 281 654	245.37
BH 377-6	N 5 002 979	E 281 654	245.41
BH 377-7	N 5 002 998	E 281 687	246.62
BH 377-8	N 5 002 987	E 281 688	246.57
BH 377-9	N 5 003 002	E 281 707	247.34
BH 377-10	N 5 002 991	E 281 708	247.23

LEGEND

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

NOTE

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

PROPOSED CROSSING
AT
HEALY LAKE ROAD
AND
PROPOSED KING'S HIGHWAY 69
DISTRICT MUNICIPALITY OF MUSKOKA

LOT 5 GEOG TWP CONGER
CON 1 TWP OF ARCHIPELAGO

SCALE	DISTRICT	REGION
AS SHOWN	52 HUNTSVILLE	NORTHERN
WP/WO 400-97-01	PLAN B-774-69-5	
SURVEY 97 12	PLAN 97 12	
SITE 44-377	PLAN E-774-69-099	

Peto MacCallum Ltd.
CONSULTING ENGINEERS
45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C6

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

BOREHOLE LOCATION PLAN AND SOIL PROFILES

PetoMacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
FOR
HEALEY LAKE ROAD UNDERPASS
W.P. 400-97-01
G.W.P. 290-97-00, SITE 44-377
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
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Job No. 97TF088A
Geocres No. 31E-133

August, 1999

TABLE OF CONTENTS

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
For
Healey Lake Road Underpass
W.P. 400-97-01
G.W.P. 290-97-00, Site 44-377
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 Healey Lake Road.

Construction of a two span underpass structure is planned. At the underpass location, the proposed four-lane Highway 69 will be constructed about 2 m above existing grade (approximate elevation 248). Road grades on Healey Lake Road over the structure will be near elevation 255.6 to 255.9, some 10 m above existing grade (based on General Arrangement drawing dated February 1999) and existing ground surface elevations determined at borehole locations).

The subsurface stratigraphy revealed at the bridge site generally comprised sand fill overlying bedrock. Peat and bouldery material were revealed in/below the fill in three boreholes, and native silt/sand/clay were encountered below the fill/peat at the centre pier.

FOUNDATIONS

Integral Abutments on Piles

The preliminary profile drawings indicate that road grades along Healey Lake Road at the underpass location will be some 10 m above existing grade. Consideration may therefore be

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BARRIE, BRAMPTON, HAMILTON, KITCHENER, TORONTO

given to construction of integral abutments supported on steel H-piles driven through the approach fill. The H-piles should be driven to refusal on bedrock anticipated at the following elevations:

Location	Bedrock/Inferred Bedrock Elevation
West Abutment, North End	240.6
West Abutment, South End	239.4
Centre Pier, North End	232.9
Centre Pier, South End	233.5
East Abutment, North End	245.7
East Abutment, South End	244.2

Factored axial resistances at the ultimate limit state for selected pile sections are presented below (resistance at centre pier reduced due to presence of boulders in fill).

H-Pile Section	Factored Resistance at ULS (kN) Abutments	Centre Pier
HP 310 x 79	1450	1200
HP 310 x 110	2000	1650

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 be driven to rock, a specified set 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 approach fill should be constructed prior to installation of the piles. It is recommended that the peat revealed below the existing fill be excavated prior to placement of the approach fill. To facilitate driving of piles through the fill, the fill should comprise granular material with a particle size not exceeding 75 mm.

The pile tip should be reinforced (OPSD 3301) to minimize the potential for damage when driving through bouldery material and setting into bedrock. Some difficulty in penetrating the bouldery material may be experienced. Rock points should be provided 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 or the existing 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. The gradation of the loose sand should be as specified on Table I. Refer to MTO Report SO-96-01 for further details.

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

$$k_s = n_h z/b$$

where z = depth (m)
b = pile width (m)

The recommended values for n_h in kN/m^3 are as follows:

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

Spread Footings/Caissons

Based on the borehole information, supporting the west abutment and centre pier of the structure on spread footings or augered caissons does not appear practical. The east abutment could be supported on conventional spread footings. We make the following comments in this regard:

- The existing fill is considered an unsuitable bearing stratum for spread footings due to the variable nature of the fill and the potential for compression/consolidation of the peat layers identified in three boreholes.
- Extending the footings down to bedrock or subexcavation/ replacement of the existing fill with structural fill would require excavating approximately 4 and 10 m below the groundwater level at the west abutment and centre pier, respectively.
- Extending caissons to bedrock is likely to be extremely problematic due to the high groundwater level, the pervious soils overlying the bedrock, and the presence of bouldery material in the fill.

If footings/caissons are employed, they should bear on sound bedrock and 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 at the east abutment slopes down to the south at an inclination of about 15° to the horizontal. Mass concrete could be placed to provide a level founding surface. Dowels should be installed through the mass concrete into the underlying bedrock. Further comments regarding dowels are provided in the next section.

Alternatively if the existing fill and peat is to be completely removed/replaced, the footings could be constructed on structural fill placed in the approaches. The engineered 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 factored bearing capacities at ultimate (ULS) and serviceability (SLS) limit states of 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, 3rd Edition, 1991) or employing the following equation, assuming a triangular pressure distribution:

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

where K = coefficient of lateral earth pressure

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

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^3)	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. An unfactored friction factor of 0.6 is recommended for footings on bedrock. A value of 0.7 may be used if the bedrock surface is roughened (asperity height of at least 25 mm) by mechanical means.

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 hole. 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 anchor 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 embankment subgrade is expected to comprise sand fill, locally bedrock at the east end. A layer of peat exists below the fill at the south end of the east and west abutments. The embankment should be constructed in accordance with OPSD 201.01, 202.010 and 203.010 appended. Excavate all peat from below the approach fill to bedrock.

The side slopes of approach fill should be inclined no steeper than 2:1 (H:V) for earth rock and 1.25:1 for rock fill. For high rock fill embankments, provide 2.0 m wide berms so that no uninterrupted rock fill slope is greater than 6 m high in accordance with the Northern Region Pavement Design Practices and Guidelines.

EXCAVATION AND GROUNDWATER CONTROL

Excavation for construction of footings at the east abutment, if employed, is expected to be carried out primarily within the existing fill and into peat above the groundwater level. The in situ materials are classified as Type 4 soils according to Occupational Health and Safety Act criteria and temporary cut slopes inclined at 3 horizontal to 1 vertical should generally be stable. Sump pumping should be adequate to handle any seepage at this location.

If caissons bearing on bedrock are employed, difficulty is likely to be experienced while augering through bouldery fill material and/or below the groundwater level. A steel liner or other means will be required to control groundwater and support the sidewalls of the excavation during installation.

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, Project Engineer and reviewed by D.W. Kerr, 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

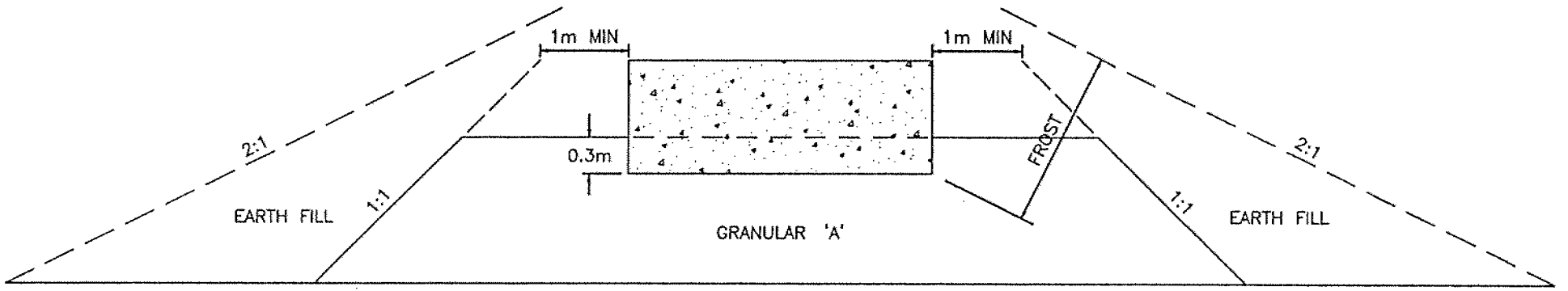
TABLE I

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

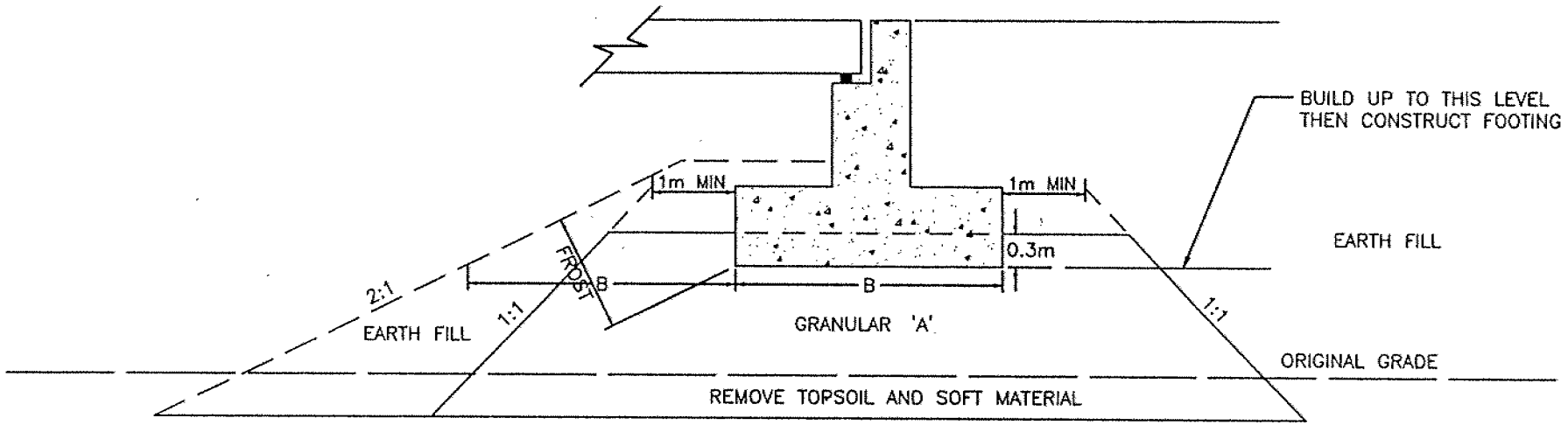
MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100
600 µm	#30	80 - 100
425 µm	#40	40 - 80
250 µm	#60	5 - 25
150 µm	#100	0 - 6

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 3C6
Tel: (905) 561-2231 Fax (905) 561-6363

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

Ontario

memorandum

To: Mike Pearsall, P. Eng. 1999 10 20
Senior Project Manager
Planning & Design Section
Northern Region

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.

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

cc: T. Kazmierowski

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Ontario

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

We have conceptually reviewed the Foundation reports for the above projects 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. Following are our comments:

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 as1.8 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, 3rd 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, 3rd 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, 3rd 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.

K. Ahmad, P. Eng
Foundation Engineer

For

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

cc: P. Furst
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