DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 31E-146

DIST. 52 REGION

W.P. No. 200-98-04
GWP: 291-97-00(A)

CONT. No.

W. O. No.

STR. SITE No. 44-385

HWY. No. 69

LOCATION Seguin Trail
Muderpan

No. of PAGES - —

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

GEOCRES No
31E-146

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT
FOR
SEGUIN TRAIL UNDERPASS
G.W.P. 291-97-00, SITE 44-385
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

Distribution:

6 cc: Ministry of Transportation
1 cc: Stantec Consulting Ltd.
1 cc: PML Hamilton
1 cc: PML Toronto
1 cc: PML Kitchener

Job No. 98TF010

February, 1999

PetoMacCallum Ltd.
CONSULTING ENGINEERS

TABLE OF CONTENTS

FOUNDATION INVESTIGATION REPORT

For

Seguin Trail Underpass

G.W.P. 291-97-00, Site 44-385

Highway 69, District 52, Huntsville

INTRODUCTION

This report summarizes the results of the foundation investigation carried out for construction of the underpass to carry Seguin Trail over the proposed four-lane Highway 69 (Station 16+567 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, Seguin Trail chainage.

SITE DESCRIPTION

The site is located about 14 km south of Parry Sound on the existing Highway 69. The proposed underpass will carry Seguin Trail traffic over the proposed four lane Highway 69. At the underpass, Seguin Trail will run approximately northeast-southwest (east-west for purposes of this report).

The bridge location is situated partially in a wooded area and partially in a swamp area. The ground surface is gently undulating. Bedrock outcrops are evident to the north of the bridge site. A fill berm exists immediately north of the site. The western portion of the bridge site is located in a swamp.

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 September 3, 4, 5, 22, 23 and December 9, 1998 and comprised 18 testholes (10 boreholes, 6 test pits and 2 rock probes) drilled/excavated at the locations shown on Drawing 1.

Due to the presence of overhead wires, the borehole at the north end of the east abutment was drilled about 2.7 m to the southwest of the planned location.

The boreholes were drilled to refusal on bedrock/inferred bedrock at depths of 0.5 to 2.6 m. Three boreholes were extended 2.9 to 3.3 m into the bedrock using NQ rock coring equipment.

The boreholes were advanced using continuous flight hollow stem augers, powered by a track-mounted CME-75 drillrig, supplied and operated by a specialist drilling contractor. Where an appreciable overburden thickness was encountered, samples were recovered using a conventional split spoon sampler as well as from the auger cuttings. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata.

The test pits were excavated to bedrock/inferred bedrock at depths of 1.3 m to 3.8 m. The test pits were advanced using a track-mounted excavator, supplied and operated by a local contractor.

The rock probes were advanced to refusal at depths of 0.7 and 1.1 m by members of our engineering staff using hand augers.

The fieldwork was carried out under the full-time supervision of a member of our engineering staff.

The groundwater conditions in the boreholes and test pits 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. Samples of the recovered rock core were subjected to unconfined compressive strength tests.

SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Log of Testhole 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 Drawings 2, 3 and 4.

The subsurface stratigraphy revealed at the bridge site comprised a pavement structure, surficial fill and/or peat overlying silt, sand and clay deposits mantling bedrock. The strata encountered are summarized below:

Pavement Structure

A pavement structure was encountered surficially in testholes 385-7 and 385-9. The pavement structure comprised 100 and 130 mm of asphaltic concrete and 100 to 150 mm of brown sand and gravel.

Fill

A 0.3 to 1.3 m thick surficial fill/probable fill layer was encountered in testholes 385-6, 385-8, 385-10, 385-15 and 385-16. A 700 mm thick layer of fill was also encountered below the pavement structure in testhole 385-9 at a depth of 0.2 m. The fill comprised sand to silty sand with gravel. Cobbles and boulders were noted in the fill identified in testhole 385-16.

Topsoil

A local 400 mm thick black silt topsoil layer underlies the fill in testhole 385-15.

Peat

Dark brown, amorphous peat was encountered at ground surface in testholes 385-1 to 385-4 and 385-11 to 385-14 drilled/excavated at the west abutment and approach of the bridge site. The peat was saturated and ranged in thickness from 0.3 to 1.3 m.

A 900 mm thick layer of dark brown amorphous peat was also encountered below the fill at a depth of 0.3 m in testhole 385-6 located on the south side of the centre pier.

Sand

A 0.5 m thick sand layer was contacted surficially in testhole 385-5.

A 0.9 to 1.7 m thick compact to very dense sand unit was encountered below the pavement structure in testhole 385-7, the fill in testhole 385-10, and the topsoil in testhole 385-15. A basal sand layer was also identified below a clay deposit in testholes 385-9, 385-12, 385-13 and 385-14 as well as a silt layer in testhole 385-8 at depths of 0.8 to 1.7 m. It extended to depths of 1.3 to 2.4 m. Moisture contents ranged from 9 to 12%.

Silt

A 300 mm thick layer of loose silt was encountered below the sand in testhole 385-8 at a depth of 0.9 m and extended to a depth of 1.2 m.

A local 0.1 m thick layer of clayey silt underlies the peat in testhole 385-3 at a depth of 1.0 m and extends to a depth of 1.1 m.

Clay

In testholes 385-1 and 385-4, the peat was underlain by a 0.3 m thick layer of very soft clay.

A 0.1 to 1.3 m thick layer of stiff silty clay was contacted below the fill, peat or silt in testholes 385-3, 385-9, and 385-11 to 385-14 at depths of 0.3 to 1.3 m and extended to depths of 0.9 to 1.7 m. Silt layers were noted within the silty clay in testhole 385-9.

Bedrock

Bedrock or inferred bedrock was contacted below the peat, sand and/or clay in all testholes at depths of 0.5 to 3.8 m (elevations 230.4 to 233.6). It is noteworthy, from the information revealed in testholes 385-15 and 385-16, that testhole 385-8 appears to have met refusal on a boulder.

A description of the rock cores recovered from testholes 385-3, 385-6 and 385-7 is provided on Table I. The bedrock consists of granitic gneiss. Core recovery ranged from 93 to 100% (average 99%) and the RQD typically ranged from 62 to 100%, 40% and 29% in the first core in testholes 385-6 and 385-7. The rock was fair to excellent quality, poor in the upper 250 and 360 mm in testholes 385-6 and 385-7.

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

Testhole	Depth (m)	Unconfined Compressive Strength (MPa)
385-3	1.9 – 2.0	81.9
385-3	3.3 – 3.5	31.4
385-6	1.2 – 1.5	64.8
385-7	1.9 – 2.1	58.2

Groundwater

Upon completion of augering, free water was observed in testholes 385-1 to 385-4, and 385-11 to 385-14 at ground surface; it was 1.2 to 2.0 m below ground surface in testholes 385-7 to 385-10, respectively. Free water was not observed in the remaining testholes during the course of the fieldwork. Observed water levels are subject to seasonal fluctuations and rainfall patterns.

CLOSURE

The fieldwork was carried out under the supervision of M. Rapsey. The drillrig and track-mounted excavator were supplied by All-Terrain Drilling Limited and Ronald Renauld Excavating Limited, respectively.

The report was written by Edward B.H. Wong, P.Eng., Project Engineer, and reviewed by Dennis W. Kerr, P.Eng., 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

EW:mma

ABBREVIATIONS FOR BORING AND TEST DATA

Accep	Acceptable
Agg	Aggregate
Amor	Amorphous
Asph	Asphalt
BR	Bedrock
Blk	Black
Bl	Blue
BH	Borehole
Bld (y)	Boulder (y)
Blds	Boulders
BU	Break Up
Br	Brown
CF	Channel Face
Cl	Clay
Co	Coarse
Cob	Cobbles
Comp	Compact
Conc	Concrete
Contam	Contaminated
Cord	Corduroy
Cr	Crushed
Dk	Dark
Decomp	Decomposed
D	Dense
E	Earth
Fib	Fibrous
w	Field Moisture Content
F	Fine
Fr Wat	Free Water
FB	Frost Boil
FH	Frost Heave
Gran	Granular
Gr	Gravel (ly)
Grn	Green
Gry	Grey
H	Heavy
Hi	Highly
HP	High Plasticity
HM	Hot Mix
Lt	Light
Liq	Liquid
W_L	Liquid Limit
Lo	Loam
L	Loose
Mrl	Marl
Matl	Material
Max	Maximum
MDD	Maximum Dry Density
MWD	Maximum Wet Density
Med	Medium
MP	Medium Plasticity
Mod	Moderate
Mott	Mottled
Mul	Mulch
NFP	No Further Progress
NFP (Blds)	No Further Progress (Boulders)
Num	Numerous
OCC	Occasional
Wopt	Optimum Moisture Content
Ora	Orange
Org	Organic
Org M	Organic Matter
Ob	Overburden
Pavt	Pavement
Pedo	Pedological
Pen Mac	Penetration Macadam
Wp	Plastic Limit
Ip	Plasticity Index
Psty	Polystyrene
Poss	Possible
PST	Prime & Surface Treated
Quant	Quantity
Reinf	Reinforced
RSS	Remoulded Shear Strength
RF	Rock Fill
Sa	Sand
Sat	Saturated
SH	Shale
St	Sensitivity
SSM	Select Subgrade Material
Sh Rk	Shot Rock
Si (y)	Silt (y)
Sl (y)	Slight (ly)
SP	Slight Plasticity
Stn (y)	Stoney
D_R	Relative Density
Stks	Streaks
Surf	Surface
Temp	Temperature
TH	Test Hole
TP	Test Pit
Tps	Topsoil
Tr	Trace
USS	Undisturbed Shear Strength
Unreinf	Unreinforced
Varv	Varved
VF	Very Fine
WT	Water Table
Weath	Weathered
W	With
Wd (y)	Wood (y)
Yel	Yellow

SUSCEPTIBILITY TO FROST HEAVING

HSFH – High
MSFH – Medium
LSFH – Low

ONTARIO PROVINCIAL STANDARD DRAWING

Date	1986 07 18	Rev	

ABBREVIATIONS
GEOTECHNICAL

Date _____

OPSD - 100.06

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.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385–1

N 5 018 458
E 273 607

PROJECT GWP 291–97–00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE 'Seguin Trail Underpass, Site 44–385
LOCATION Station 9+945 (Seguin Trail) 6.5 m Lt. Centreline
BORING METHOD Hand Augers
BORING DATE September 4, 1998

OUR PROJECT 98TF010
ENGINEER E. Wong
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N – VALUES	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
	GROUND ELEVATION 232.32								
0	PEAT : Dark brown, silty amorphous peat		232						Upon completion of augering, free water at surface.
1.20									
1.46	CLAY : Very soft, grey, clay		231						
1.5	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 1.46m. PROBABLE BEDROCK.		230						
3.0									
4.5									
6.0									
7.5									
9.0									
10.5									
12.0									
13.5									
15.0									
16.5									

NOTES:

CHECKED BY: E.W.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385–2

N 5 018 447
E 273 620

PROJECT GWP 291–97–00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Seguin Trail Underpass, Site 44–385
LOCATION Station 9+945 (Seguin Trail) 11.0m Rt. Centreline
BORING METHOD Hand Augers
BORING DATE September 4, 1998

OUR PROJECT 98TF010
ENGINEER E. Wong
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	SAMPLES NUMBER	SAMPLES 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 232.45								
0	PEAT : Dark brown, amorphous peat, saturated		232						Upon completion of augering, free water at surface.
-1.00-	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 1.00m. PROBABLE BEDROCK.		231						
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: E.W.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385–3

N 5 018 476
E 273 619

PROJECT GWP 291–97–00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Seguin Trail Underpass, Site 44–385
LOCATION Station 9+966 (Seguin Trail) 5.5m Lt. Centreline
BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring
BORING DATE September 3, 1998
OUR PROJECT 98TF010
ENGINEER E. Wong
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	SAMPLES NUMBER	SAMPLES 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 232.45								
0	**PEAT** : Dark brown, amorphous peat		232						Upon completion of augering, free water at surface.
1.00									
1.10	**SILT** : Grey, clayey silt, organic								
1.35			231						
1.5	**CLAY** : Grey, clay, high plastic, W.T.P.L.			1	RC		660 100 85 100		
			230						
	BEDROCK : Granitic Gneiss			2	RC		1520 97 62 100		
3.0			229						
				3	RC		360 100 100 100		
				4	RC		400 100 100 100		
4.29							RUN (mm) / RECOVERY (%) / RQD (%) / DRILL WATER RETURN (%)		
4.5	**BOREHOLE TERMINATED AT 4.29m.**		228						
6.0									
7.5									
9.0									
10.5									
12.0									
13.5									
15.0									
16.5									

NOTES:

CHECKED BY: E.W.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385-4

N 5 018 467
E 273 663

PROJECT GWP 291-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Seguin Trail Underpass, Site 44-385
LOCATION Station 9+966 (Seguin Trail) 10.5m Rt. Centreline
BORING METHOD Hand Augers
BORING DATE September 3, 1998

OUR PROJECT 98TF010
ENGINEER E. Wong
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	GROUNDWATER OBSERVATIONS AND REMARKS
	GROUND ELEVATION 232.52						
0	PEAT : Dark brown, amorphous peat		232				Upon completion of augering, free water at surface.
1.20							
1.5 1.50	CLAY : Grey, clay		231				
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 1.50m. PROBABLE BEDROCK.						
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

NOTES:

CHECKED BY: E.W.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385-5

N 5 018 503
E 273 638

PROJECT GWP 291-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Seguin Trail Underpass, Site 44-385
LOCATION Station 10+000 (Seguin Trail) 6.0 m Lt. Centreline
BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring
BORING DATE September 4, 1998
OUR PROJECT 98TF010
ENGINEER E. Wong
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
	GROUND ELEVATION 234.05								
0									
0.45	SAND : Brown, fine to coarse sand, trace of silt, trace of gravel, damp		233						Upon completion of augering, no free water.
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 0.45m. PROBABLE BEDROCK.								
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: E.W.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385–6

N 5 018 494
E 273 652

PROJECT GWP 291–97–00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Seguin Trail Underpass, Site 44–385
LOCATION Station 10+000 (Seguin Trail) 11.0 m Rt. Centreline
BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring
BORING DATE September 4, 1998

OUR PROJECT 98TF010
ENGINEER E. Wong
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N – VALUES	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
	GROUND ELEVATION 233.51								
0									
0.30	FILL : Dark brown, sand with silt		233						
1.22	PEAT : Dark brown, amorphous peat								
1.5	BEDROCK : Granitic Gneiss		232	1	RC		250		
				2	RC		360		
			231	3	RC		1120		
3.0									
			230	4	RC		1270		
4.22									
4.5	BOREHOLE TERMINATED AT 4.22m.		229						
6.0									
7.5									
9.0									
10.5									
12.0									
13.5									
15.0									
16.5									

	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)
	250	100	40	100
	360	100	86	100
	1120	100	78	100
	1270	100	86	100

NOTES:

CHECKED BY: E.W.

PetoMacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385-7

N 5 018 532
E 273 655

PROJECT GWP 291-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Seguin Trail Underpass, Site 44-385
LOCATION Station 10+034 (Seguin Trail) 5.5 m Lt. Centreline
BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring
BORING DATE September 5, 1998
OUR PROJECT 98TF010
ENGINEER E. Wong
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 234.53								
0									
0.28	**PAVEMENT STRUCTURE** : 130mm of asphalt over 150mm of brown sand and gravel	CONCRETE	234						
1.19	**SAND** : Very dense, brown, fine to medium sand with silt, very moist		233	1	RC		360 93 29 100		
1.5									
	BEDROCK : Granitic Gneiss		232	2	RC		1470 98 95 90		Slight drill water lost at 1.85m.
3.0			231	3	RC		1450 100 91 100		
4.47			230						
4.5	**BOREHOLE TERMINATED AT 4.47m.**						RUN (mm) / RECOVERY (%) / RQD (%) / DRILL WATER RETURN (%)		
			229						
6.0									
7.5									
9.0									
10.5									
12.0									
13.5									
15.0									
16.5									

NOTES:

CHECKED BY: E.W.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385-8

N 5 018 524
E 273 669

PROJECT GWP 291-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Seguin Trail Underpass, Site 44-385
LOCATION Station 10+034 (Seguin Trail) 10.5 m Rt. Centreline
BORING METHOD Hand Augers
BORING DATE September 3, 1998
OUR PROJECT 98TF010
ENGINEER E. Wong
TECHNICIAN M. Ropsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	SAMPLES NUMBER	SAMPLES 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 234.20								
0	SAND : Loose, brown, fine sand with silt, saturated (probable fill)		234	1	SS				Upon completion of augering, free water at 1.30m.
0.90				2	SS	9	•		
1.20	SILT : Loose, grey, silt, trace of fine sand, wet		233						
1.5 1.60	SAND : Very dense, dark brown, sand with silt, very moist			3	SS	38/100mm			
			232						
3.0	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 1.60m ON BOULDER OR BEDROCK.								
4.5									
6.0									
7.5									
9.0									
10.5									
12.0									
13.5									
15.0									
16.5									

NOTES:

CHECKED BY: E.W.

PetoMacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385-9

N 5 018 551
E 273 664

PROJECT GWP 291-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Seguin Trail Underpass, Site 44-385
LOCATION Station 10+055 (Seguin Trail) 6.5 m Lt. Centreline
BORING METHOD Cintinuous Flight Solid Stem Augers
BORING DATE September 5, 1998

OUR PROJECT 98TF010
ENGINEER E. Wong
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES
	GROUND ELEVATION 234.83					
0						
0.20	PAVEMENT STRUCTURE : 100mm of asphalt over 100mm of brown sand and gravel with silt					
0.90	FILL : Dark brown, silty fine sand		234	1	SS	13
1.5						
1.70			233	2	SS	71
2.00	CLAY : Stiff, brown, silty clay, trace of sand, occasional silt layers, medium plastic, W.T.P.L.					
2.39				3	SS	16/100mm
3.0	SAND : Very dense, brown, fine to coarse sand with gravel, trace of silt, moist		232			
	becoming saturated					
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 2.39m. PROBABLE BEDROCK.					
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, free water at 2.00m.

NOTES:

CHECKED BY: E.W.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 385–10

N 5 018 542
E 273 679

PROJECT GWP 291–97–00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Seguin Trail Underpass, Site 44–385
LOCATION Station 10+055 (Seguin Trail) 11.0 m Rt. Centreline
BORING METHOD Cintinuous Flight Solid Stem Augers
BORING DATE September 5, 1998
OUR PROJECT 98TF010
ENGINEER E. Wong
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N – VALUES
	GROUND ELEVATION 234.01					
0						
	FILL : Dark brown, fine to medium sand with silt, trace of gravel, moist			1	SS	15
0.90			233			
1.20	SAND : Compact, brown, fine to medium sand with silt and gravel, moist			2	SS	18
1.5	becoming saturated		232			
2.59						
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 2.59m. PROBABLE BEDROCK.		231			
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 × 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, free water at 1.20m.

NOTES:

CHECKED BY: E.W.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

Sequin Trail Underpass
G.W.P. 291-97-00, Site 44-385
Highway 69, District 52
Huntsville, Ontario
Datum Centre Line

385-11	9+963	5.5 m LT C/L	El. 232.45
	0-1.25	Br Amor Peat Sat	
	1.25-1.50	Gry Si(y) Cl Tr Sa	
	1.50	NFP BR	
		Fr Wat @ 0	

385-12	9+969	5.5 m LT C/L	El. 232.45
	0-250	Br Amor Peat Sat	
	250-850	Gry Si(y) Cl Tr Sa	
	850-1.30	Gry Sa Sat	
	1.30	NFP BR	
		Fr Wat @ 0	

385-13	9+963	11.0 m RT C/L	El. 232.52
	0-1.15	Br Amor Peat Sat	
	1.15-1.30	Gry Si(y) Cl Tr Sa	
	1.30-1.40	Gry Sa Sat	
	1.40	NFP BR	
		Fr Wat @ 0	

385-14	9+969	10.5 m RT C/L	El. 232.52
	0-1.15	Br Amor Peat Sat	
	1.15-1.20	Gry Si(y) Cl Tr Sa	
	1.20-1.50	Gry Sa Sat	
	1.50	NFP BR	
		Fr Wat @ 0	

385-15	10+034	13.0 m RT C/L	El. 234.20
	0-400	Br Sa Fill W Gr Damp	
	400-800	Blk Si Tps Moist	
	800-1.70	Gry Sa Moist	
	1.70-3.00	Gry Si(y) Cl	
	3.00	NFP BR	

385-16	10+037	11.0 m RT C/L	El. 234.20
	0-1.25	Br Sa Fill W Gr Tr Cob Tr Blds	
	1.25-3.80	Gry Si(y) Cl	
	3.80	NFP BR	

Rock Probes

385-17	10+000	3.0 LT C/L	El. 233.99
	0-650	Overburden	
	650	NFP BR	

385-18	10+000	3.0 RT C/L	El. 234.03
	0-1.10	Overburden	
	1.10	NFP BR	

Our Ref: 98TF010

TABLE I

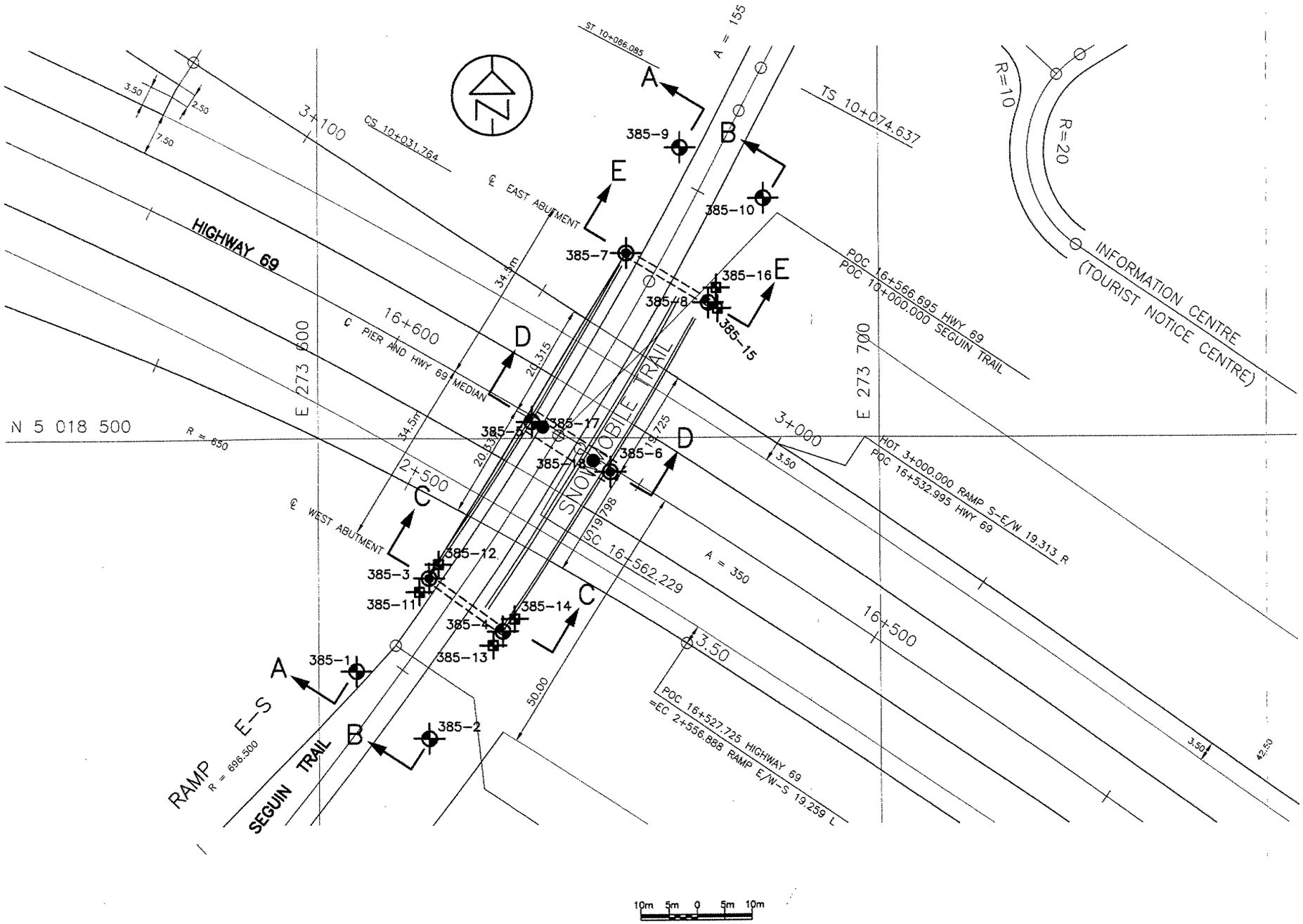
ROCK CORE DESCRIPTION
GWP 291-97-00, Site No. 44-385

CORE RECOVERY					CORE DESCRIPTION	
TESTHOLE	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
385-3	1	1.35 – 2.01	100	85	1.35 – 4.29	**GRANITIC GNEISS,** pink, fine to medium crystalline; medium to high strength; unweathered; close to moderate spaced flat partings, rough planar, tight; fair to excellent quality.
	2	2.01 – 3.53	97	62		
	3	3.53 – 3.89	100	100		
	4	3.89 – 4.29	100	100		
385-6	1	1.22 – 1.47	100	40	1.22 – 4.22	**GRANITIC GNEISS,** pink, fine to medium crystalline, with occasional quartz layers; high strength; unweathered; close to moderate spaced flat to dipping partings, rough to smooth planar, tight to oxidized; poor to good quality.
	2	1.47 – 1.83	100	86		
	3	1.83 – 2.95	100	78		
	4	2.95 – 4.22	100	86		
385-7	1	1.19 – 1.55	93	29	1.19 – 4.47	**GRANITIC GNEISS,** pink, fine to medium crystalline, with occasional layer of grey pegmatite, coarse crystalline and inclusions of black hornblende; high strength; unweathered; very close to close becoming close to moderate spaced flat partings, rough planar, tight; poor to excellent quality.
	2	1.55 – 3.02	98	95		
	3	3.02 – 4.47	100	91		

RQD = Rock Quality Designation

Logged by J. Wright

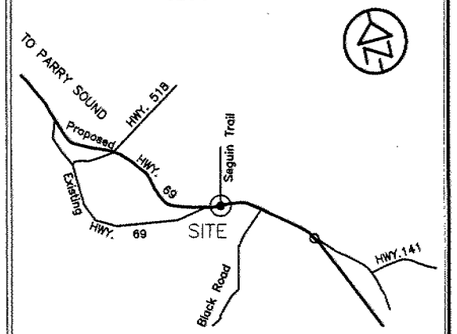
BOREHOLE	LOCATION	ELEVATION	BOREHOLE	LOCATION	ELEVATION	TEST PIT	LOCATION	ELEVATION	ROCK PROBE	LOCATION	ELEVATION
385-1	N 5 018 458 E 273 607	232.32	385-7	N 5 018 532 E 273 655	234.53	385-11	N 5 018 473 E 273 618	232.45	385-17	N 5 018 502 E 273 640	233.99
385-2	N 5 018 447 E 273 620	232.45	385-8	N 5 018 524 E 273 669	234.20	385-12	N 5 018 477 E 273 621	232.45	385-18	N 5 018 496 E 273 649	234.03
385-3	N 5 018 476 E 273 619	232.45	385-9	N 5 018 551 E 273 664	234.83	385-13	N 5 018 463 E 273 631	232.52			
385-4	N 5 018 467 E 273 633	232.52	385-10	N 5 018 542 E 273 679	234.01	385-14	N 5 018 468 E 273 635	232.52			
385-5	N 5 018 503 E 273 638	234.05				385-15	N 5 018 523 E 273 672	234.20			
385-6	N 5 018 494 E 273 652	233.51				385-16	N 5 018 527 E 273 671	234.20			



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DISTRICT MUNICIPALITY OF PARRY SOUND
GEOG TWP FOLEY



KEY PLAN
SCALE: NTS

LEGEND

- BOREHOLE & ROCK CORE
- TEST PIT
- BOREHOLE
- ROCK PROBE
- FOOTING LOCATION

NOTE

1. REFER TO LOG OF TESTHOLE SHEETS FOR DETAILED SUBSURFACE CONDITIONS.
2. REFER TO DRAWINGS 2, 3 AND 4 FOR SOIL PROFILES.

MINISTRY OF TRANSPORTATION
ENGINEERING AND RIGHT OF WAY OFFICE
SURVEYS AND PLANS SECTION

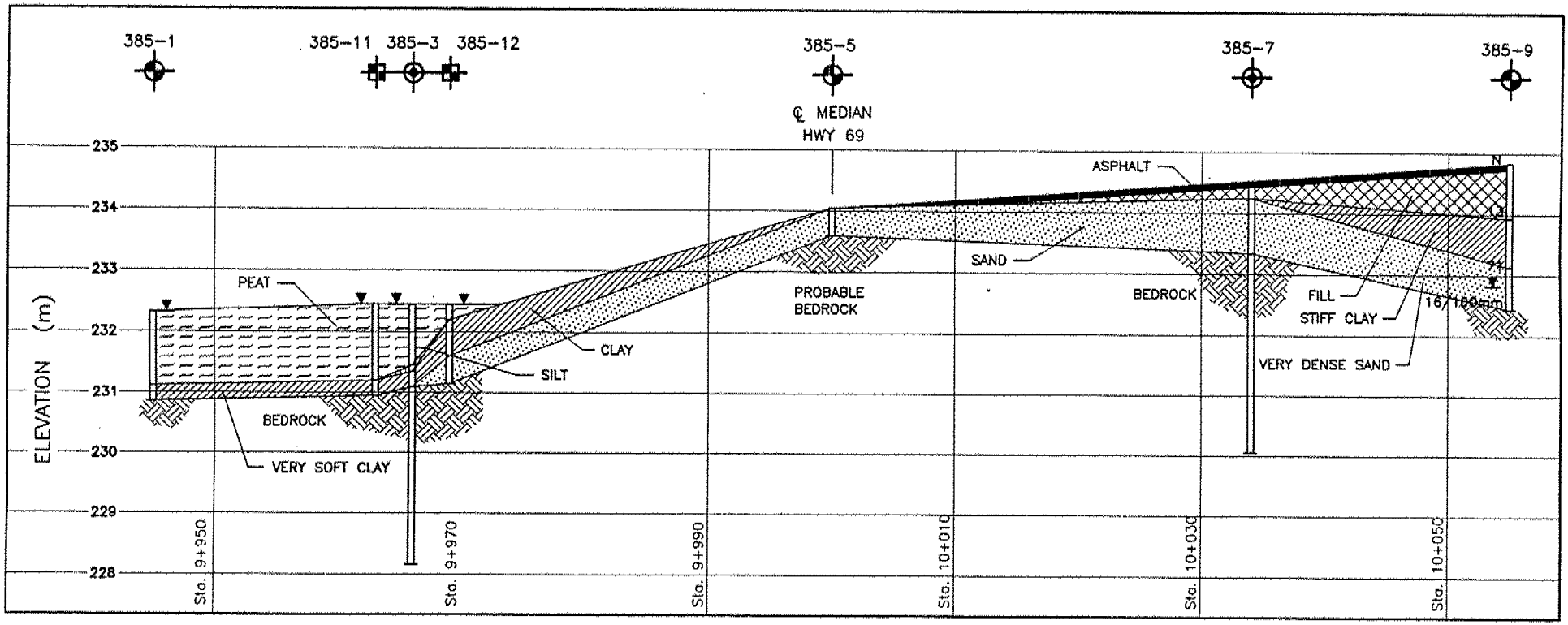
PROPOSED CROSSING
AT
SEGUIN TRAIL
AND
PROPOSED KING'S HIGHWAY 69
DISTRICT MUNICIPALITY OF PARRY SOUND
LOT 121 GEOG TWP FOLEY
CON B TWP OF SEGUIN

SCALE	DISTRICT	REGION
AS SHOWN	52 HUNTSVILLE	NORTHERN
WP/WO 291-97-00	PROFILE	PLAN
SURVEY	PLAN	
SITE 44-385	**PLAN**	

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

DRAWN	CB	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED	EW	FEB. 1999	1:750	98TF010	1
APPROVED	DWK				

TESTHOLE LOCATION PLAN



SECTION A–A

SCALE VERTICAL 1:100
HORIZONTAL 1:500

LEGEND

- BOREHOLE AND ROCK CORE
- BOREHOLE
- ROCK PROBE
- FILL
- SAND
- TEST PIT
- PEAT/TOPSOIL
- SILT
- ASPHALT
- CLAY
- BEDROCK/PROBABLE BEDROCK
- OBSERVED WATER LEVEL (DURING OR UPON COMPLETION OF DRILLING/TEST PITTING, DATED SEPTEMBER 3, 4, 5, 22 AND 23, 1998)

NOTE

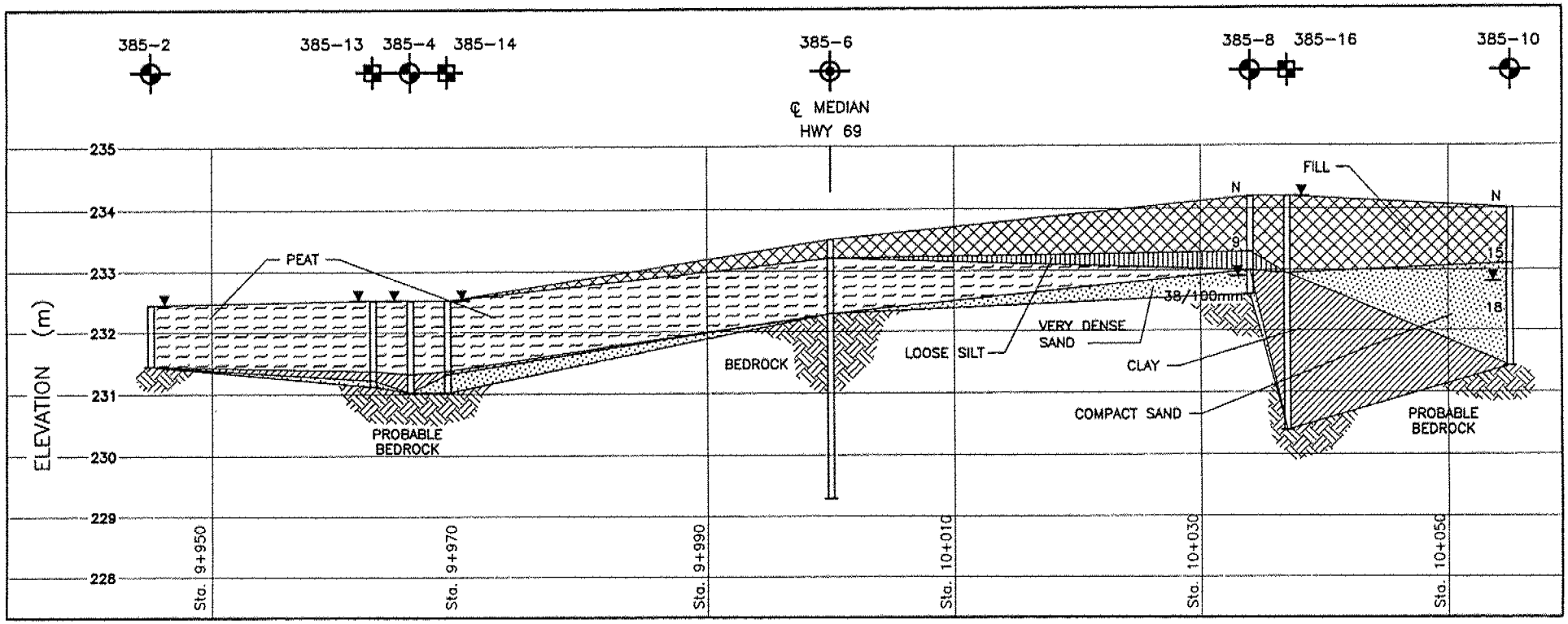
1. REFER TO DRAWING NO. 1 FOR TESTHOLE AND SECTION LOCATIONS
2. REFER TO LOG OF TESTHOLE SHEETS FOR DETAILED SUBSURFACE CONDITIONS.
3. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT TESTHOLE LOCATIONS. BETWEEN TESTHOLES, BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.

G.W.P. 291-97-00, HIGHWAY 69
SEGUIN TRAIL UNDERPASS, SITE 44-385
DISTRICT 52, HUNTSVILLE, ONTARIO

SOIL PROFILES

Peto MacCallum Ltd.
CONSULTING ENGINEERS
45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C6
Tel. (905) 561-2231 Fax (905) 561-5363

DATE	SCALE	JOB NO.	DRAWING NO.
FEB. 1999	AS SHOWN	98TF010	2



SECTION B–B

SCALE VERTICAL 1:100
HORIZONTAL 1:500

LEGEND

- BOREHOLE AND ROCK CORE
- BOREHOLE
- ROCK PROBE
- FILL
- SAND
- TEST PIT
- PEAT/TOPSOIL
- SILT
- ASPHALT
- CLAY
- BEDROCK/PROBABLE BEDROCK
- OBSERVED WATER LEVEL (DURING OR UPON COMPLETION OF DRILLING/TEST PITTING, DATED SEPTEMBER 3, 4, 5, 22 AND 23, 1998)

NOTE

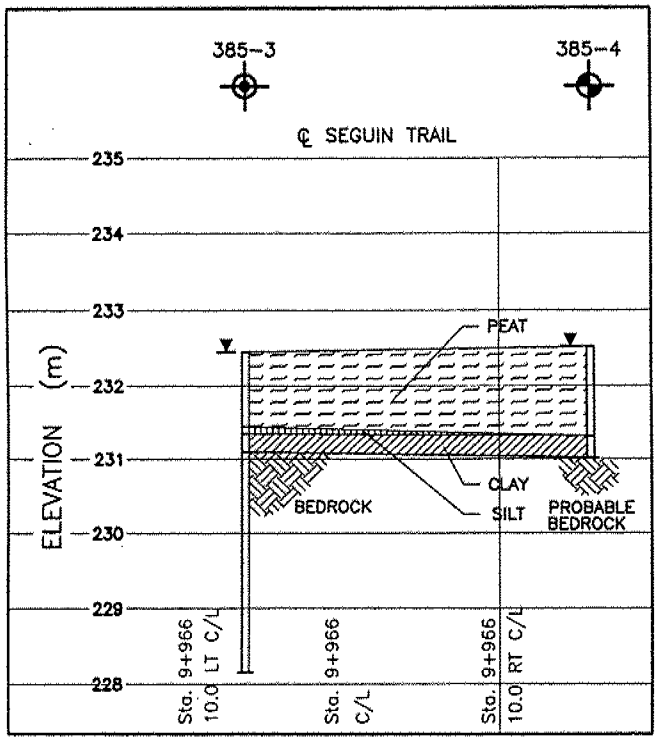
1. REFER TO DRAWING NO. 1 FOR TESTHOLE AND SECTION LOCATIONS
2. REFER TO LOG OF TESTHOLE SHEETS FOR DETAILED SUBSURFACE CONDITIONS.
3. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT TESTHOLE LOCATIONS. BETWEEN TESTHOLES, BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.

G.W.P. 291-97-00, HIGHWAY 69
SEGUIN TRAIL UNDERPASS, SITE 44-385
DISTRICT 52, HUNTSVILLE, ONTARIO

SOIL PROFILES

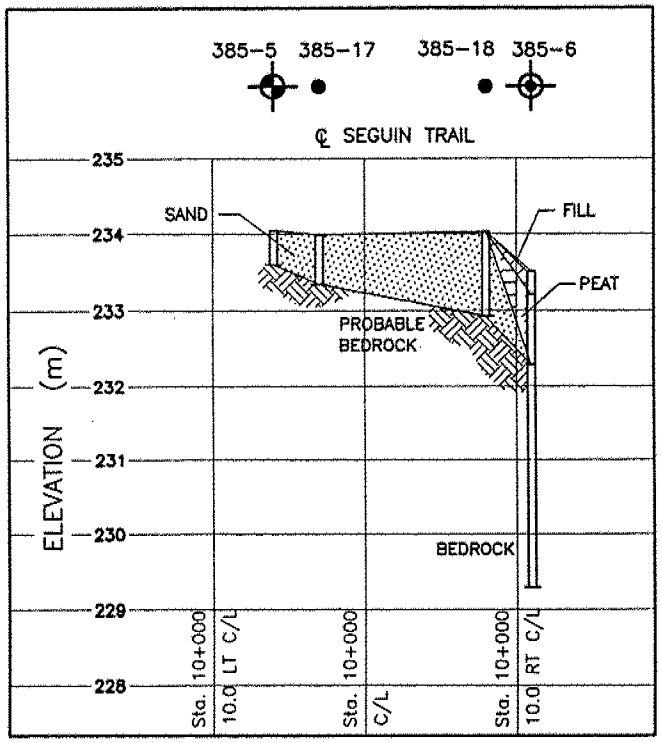
Peto MacCallum Ltd.
CONSULTING ENGINEERS
45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C6
Tel. (905) 561-2231 Fax (905) 561-6363

DATE	SCALE	JOB NO.	DRAWING NO.
FEB. 1999	AS SHOWN	98TF010	3



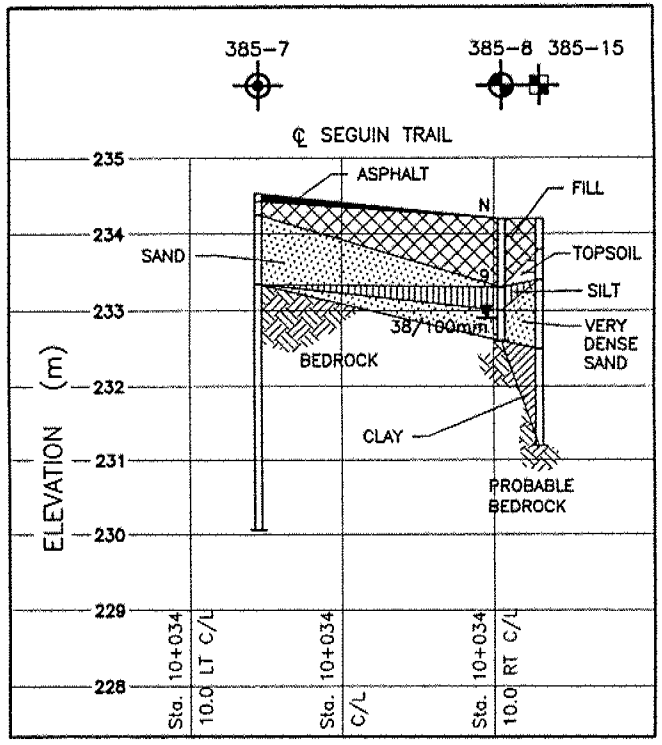
SECTION C–C

SCALE VERTICAL 1:100
HORIZONTAL 1:500



SECTION D–D

SCALE VERTICAL 1:100
HORIZONTAL 1:500



SECTION E–E

SCALE VERTICAL 1:100
HORIZONTAL 1:500

LEGEND

- BOREHOLE AND ROCK CORE
- BOREHOLE
- ROCK PROBE
- FILL
- SAND
- TEST PIT
- PEAT/TOPSOIL
- SILT
- ASPHALT
- CLAY
- BEDROCK/PROBABLE BEDROCK
- OBSERVED WATER LEVEL (DURING OR UPON COMPLETION OF DRILLING/TEST PITTING, DATED SEPTEMBER 3, 4, 5, 22 AND 23, 1998)

NOTE

1. REFER TO DRAWING NO. 1 FOR TESTHOLE AND SECTION LOCATIONS
2. REFER TO LOG OF TESTHOLE SHEETS FOR DETAILED SUBSURFACE CONDITIONS.
3. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT TESTHOLE LOCATIONS. BETWEEN TESTHOLES, BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.

G.W.P. 291-97-00, HIGHWAY 69
SEGUIN TRAIL UNDERPASS, SITE 44-385
DISTRICT 52, HUNTSVILLE, ONTARIO

SOIL PROFILES

Peto MacCallum Ltd.
CONSULTING ENGINEERS
45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C6
Tel: (905) 561-2231 Fax (905) 561-6363

DATE	SCALE	JOB NO.	DRAWING NO.
FEB. 1999	AS SHOWN	98TF010	4

PetoMacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
FOR
SEGUIN TRAIL UNDERPASS
G.W.P. 291-97-00, SITE 44-385
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

Distribution:

6 cc: Ministry of Transportation
1 cc: Stantec Consulting Ltd.
1 cc: PML Hamilton
1 cc: PML Toronto
1 cc: PML Kitchener

Job No. 98TF010

February, 1999

TABLE OF CONTENTS

PetoMacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT

For

Seguin Trail Underpass

G.W.P. 291-97-00, Site 44-385

Highway 69, District 52, Huntsville

INTRODUCTION

This report provides geotechnical comments and recommendations regarding design and construction of foundations, abutments and approaches for the proposed Seguin Trail underpass at Highway 69 (Station 16+567 Highway 69 chainage).

Construction of a two span underpass structure is planned. At the underpass location, the proposed four-lane Highway 69 will be constructed some 1.3 to 2.3 m above the existing ground surface (road grade at elevation 235.4 SBL and 235.3 NBL). Road grades on Seguin Trail over the structure will be near elevation 242, some 8 to 9 m above existing grade (based on preliminary profile drawings provided by Stantec Consulting Ltd., October 4, 1998 titled "Profiles NB/SB Lanes", undated and existing ground surface elevations determined at testhole locations).

The subsurface stratigraphy revealed at the bridge site generally comprised a pavement structure, surficial peat and/or sand fill layer overlying sand and clay/silt deposits mantling bedrock. Bedrock/inferred bedrock was contacted at depths of 0.5 to 1.5 m, locally 3.8 m at the south end of the east abutment.

FOUNDATIONS

Integral Abutments on Piles

Based on preliminary profile drawings and existing ground surface elevations determined at testhole locations, road grades along Seguin Trail at the underpass location will be some 8 to 9 m above existing grade. Construction of integral abutments supported on steel H-piles driven through the

approach fill is therefore considered feasible. The H-piles should be driven to refusal on bedrock anticipated at the following elevations:

Location	Bedrock/Inferred Bedrock Elevation
West Abutment, North End	231.1
West Abutment, South End	231.0
Centre Pier, North End	233.6
Centre Pier, South End	232.3
East Abutment, North End	233.3
East Abutment, South End	230.4-231.2*

* refer to testholes 385-15 and 385-16

Factored pile capacities at the ultimate limit state for selected pile sections computed in accordance with the MTO structural office policy memo 98-01 are presented below. The capacities were obtained by applying a geotechnical resistance factor of 0.6 and a yield strength of 300 MPa for the steel.

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

The capacity 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 driving through the overburden and setting into bedrock. The elevation of the bedrock surface at the test locations for the east abutment ranged from 230.4 to 233.3. Rock points should be employed to minimize the potential for sliding of the pile tip along sloping bedrock surface, identified at the east abutment.

Pile caps should be provided with the normal 1.6 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 sand. 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.

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
b = pile width

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

Based on the borehole information, it is considered that the structure may be supported on conventional spread footings founded on bedrock. Excavation of about 1.5, 1.0 and 1.2 to 3.8 or 1.2 m of overburden soils at the west abutment, centre pier and east abutment respectively, will be required for the construction of footings on bedrock. It must be noted however that excavation for footing construction in the swamp environment that exists at the west abutment may be problematic due to groundwater conditions.

Foundations bearing on the sound bedrock at elevations 230.4 to 233.6 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 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 below the footings should be benched or socketed to provide a level founding surface.

Alternatively, spread 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 Capacity (kPa) ULS	SLS
1	735	250
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 1.6 m was assumed for computation of the ULS capacities.

In general, where founding levels of adjacent footings vary, the founding elevation between footings should be stepped in maximum 600 mm steps at a maximum inclination of 10 horizontal to 7 vertical.

All footings subject to frost action should be provided with the normal 1.6 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

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

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.

Installation of dowels into the bedrock could be employed to increase the lateral resistance to the horizontal force if required; a factored rock-grout bond stress of 1.4 MPa at the ultimate limit state (resistance factor of 0.4 applied) is recommended for design. The anchors should extend a minimum 30 bar diameters into sound bedrock. The increased lateral resistance will be provided by the increased sliding resistance developed at the interface between the footing and founding medium due to the increased vertical pressure created by the stress in the anchor.

APPROACH FILL

It is anticipated that the embankment will be constructed with rockfill materials. Rockfill embankments should be constructed in accordance with OPSS 206.07.08 and OPSD 202.010.

The results of the investigation indicate that the section of embankment between station 9+945 and 9+965 will be constructed over swamp. Swamp treatment should be in accordance with OPSD 203.010 (MOD) attached, and the excavation extended to bedrock (anticipated depth 1.5 to 2.0 m).

The shoulder width on top of the rockfill embankment should be widened by 1 m on each side per MTO practice.

The remaining sections of embankments will be constructed over existing pavement structure and/or inorganic soils where no settlement or bearing capacity problems are anticipated. The shoulder width of the embankment constructed on a firm base should be widened by 2 m on each side per MTO practice. A 2 m wide mid-height berm should be constructed if the height of the embankment exceeds 10 m in accordance with MTO practice.

EXCAVATION AND GROUNDWATER CONTROL

Excavation for construction of footings is expected to be carried out within the surficial fill, peat and overburden deposits.

Excavation of the fill and overburden should be relatively straightforward using conventional equipment. The presence of boulders within the overburden should not be overlooked.

The fill and in situ materials are classified as Type 3 soils according to Occupational Health and Safety Act criteria. If open cut procedures and used, temporary cut slopes inclined at 1 horizontal to 1 vertical should be stable above the groundwater level. The peat is classified as a Type 4 soil; temporary cut slopes inclined at 3 horizontal to 1 vertical, or flatter, will be required.

Control of groundwater in the swamp environment is expected to be difficult.

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 Edward B.H. Wong, P.Eng., Project Engineer and reviewed by Dennis W. Kerr, P.Eng., 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

EW:mma

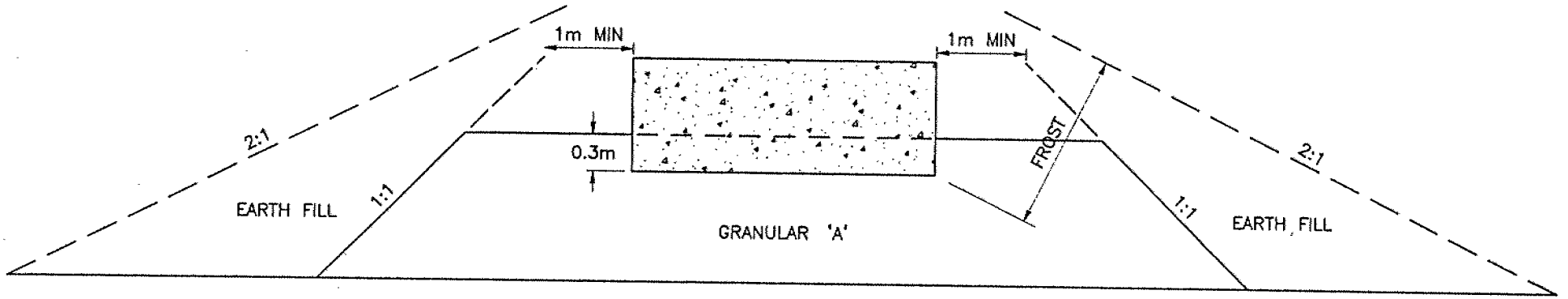
TABLE I

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

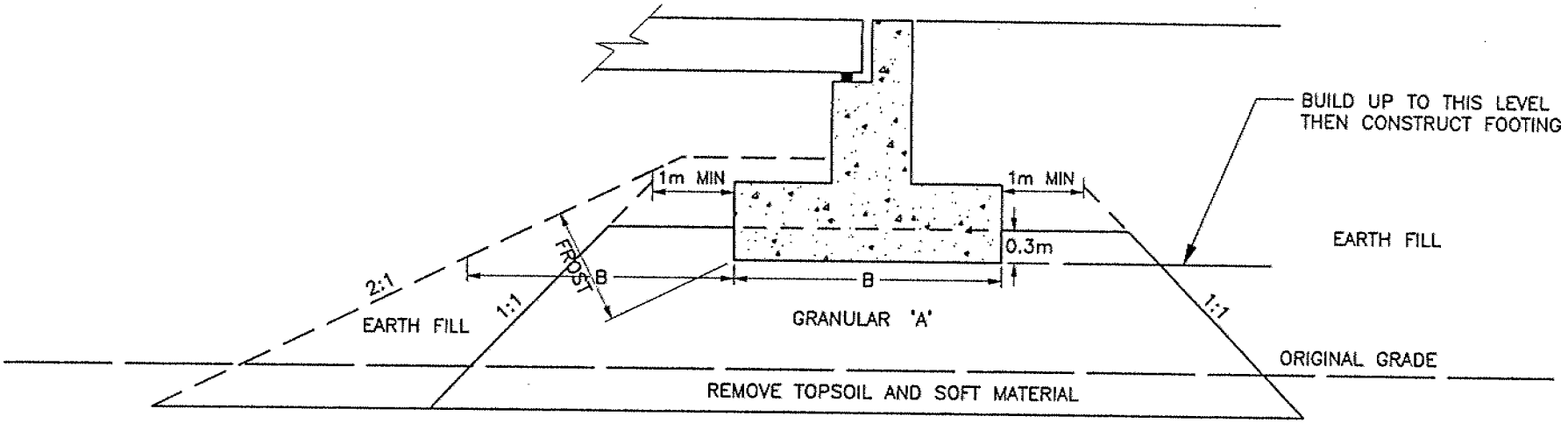
MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100
600 μm	#30	80 - 100
425 μm	#40	40 - 80
250 μm	#60	5 - 25
150 μm	#100	0 - 6

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

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



CROSS SECTION



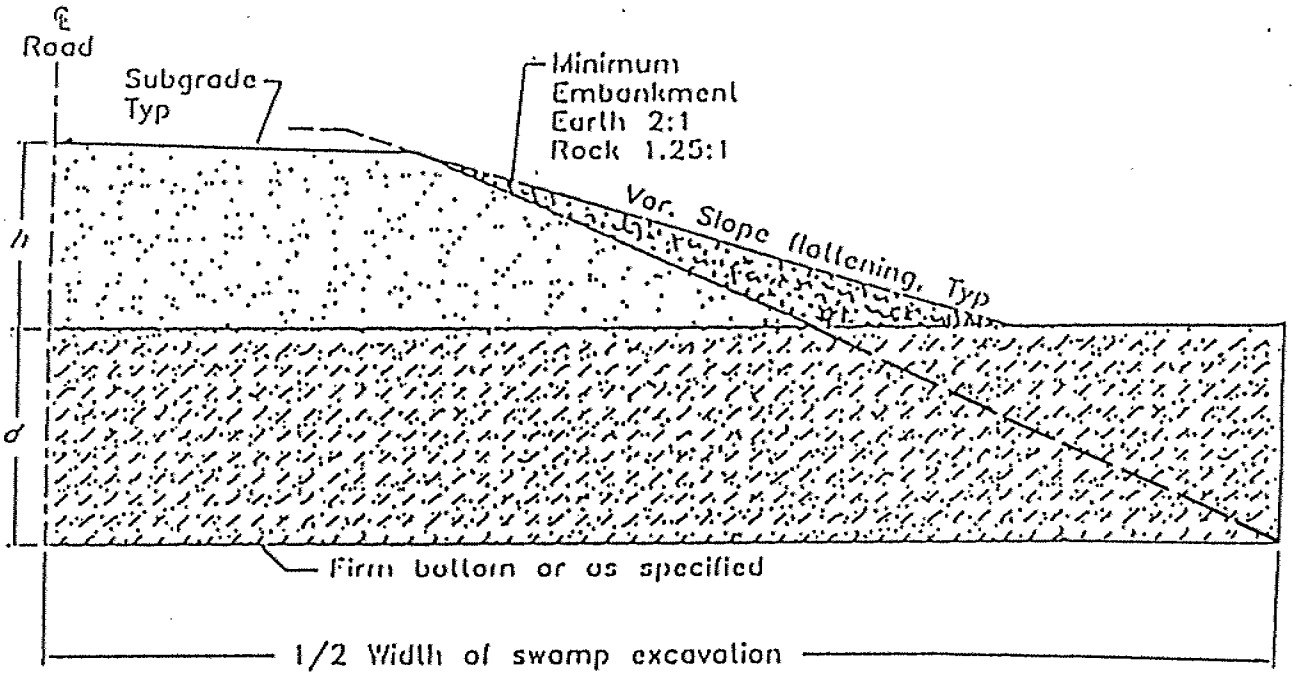
LONGITUDINAL SECTION

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.
OCT. 1998	NTS	98TF010	1



NOTES:

A Height of fill is the vertical difference between top of subgrade and top of swamp elevation measured at new road centreline.

B For divided roads with median < 10 metres, excavate swamp material full width.

C For divided roads with median ≥ 10 metres, excavate swamp material to limits shown.

D All dimensions are in millimetres or metres unless otherwise shown.

LEGEND:

- Embankment materials as specified
- Excavated swamp material
- Excavate and backfill

h — Height of fill
d — Depth of swamp

ONTARIO PROVINCIAL STANDARD DRAWING

EMBANKMENTS OVER SWAMP NEW CONSTRUCTION

1998 03 01 | Rev

OPSD - 203.010 (MOD)