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GEOCRES No. ____

DIST. 52 REGION ____

G W.P. No. 290-97-00(E)

CONT. No. ____

W. O. No. ____

STR. SITE No. 44-379

HWY. No. ~~52~~ 69

LOCATION Hwy 69 & CPR Subway

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. ____

REMARKS: ____

PetoMacCallum Ltd.
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FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
CP RAIL GRADE SEPARATION
W.P. 402-97-01
G.W.P. 290-97-00, SITE 44-379
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

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July, 2000

PetoMacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT
FOR
CP RAIL GRADE SEPARATION
W.P. 402-97-01
G.W.P. 290-97-00, SITE 44-379
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

Job No. 97TF088G
Geocres No. Not Assigned

July, 2000

PetoMacCallum Ltd.
CONSULTING ENGINEERS

TABLE OF CONTENTS

FOUNDATION INVESTIGATION REPORT
For
CP Rail Grade Separation
W.P. 402-97-01
G. W.P. 290-97-00, Site 44-379
Highway 69, District 52, Huntsville

INTRODUCTION

This report summarizes the results of the foundation investigation carried out for construction of the proposed CP Rail grade separation structure at Highway 69 (Station 15+677 Highway 69 chainage).

The report pertains to the proposed bridge structure and approaches within 20 m of the abutments, between approximate stations 9+940 and 10+060, CPR chainage.

SITE DESCRIPTION

The site is located about 9 km north of MacTier and about 1.4 km west of the existing Highway 69 alignment. The proposed structure will carry CP Rail traffic over the proposed new four-lane section of Highway 69. At the underpass, the CP railway will run east-west along a diversion alignment situated approximately 16.6 m north of the existing CP Rail alignment.

The proposed bridge location comprises a bedrock outcrop. The existing CPR alignment to the south is located in a rock cut. The ground surface rises slightly to the east parallel to the railway. The area north of/adjacent to the site is heavily wooded.

The area is part of the Precambrian Laurentian peneplane. In general, the topography is relatively flat but quite irregular in detail with many small lakes separated by rocky ridges. 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.

The bedrock formations are of Precambrian age and are largely composed of veined, banded, and homogeneous pink and grey gneisses produced by injection and granitization of metamorphic gneisses of various types.

INVESTIGATION PROCEDURES

The fieldwork was carried out during the period August 11 to 18, 1998 and comprised three rock cores put down at the locations shown on Drawing 1. The core locations were situated along a preliminary alignment of the rail diversion located approximately 7 m south of the presently proposed alignment.

The cores were extended to depths of 15.4 to 17.9 m at an inclination of 16.0 to 17.5° from the vertical, oriented 120 to 130° apart. Coring commenced at the rock surface at depths of 0 to 230 mm.

The cores were advanced using NQ rock coring equipment, powered by a track-mounted CME-75 drillrig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. The core recovery and Rock Quality Designation (RQD) were documented in the field as the cores were recovered. The composition of the thin overburden layers was also recorded.

Franklin Geotechnical Ltd. (FGL) was retained to provide a detailed description of the rock characteristics as well as comments/recommendations regarding the rock engineering aspects of the Highway 69/CPR grade separation structure (Report No. G819.2 dated September 4, 1998, revised March 22, 1999). The recovered rock core samples were provided to FGL for detailed visual examination and classification.

SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Log of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, rock core descriptions and groundwater observations. A stratigraphic profile prepared from the borehole data is presented on Drawing 2. The ground surface and probable bedrock profiles along the proposed CP Rail centreline (derived from borehole information obtained during the geotechnical investigation for the rail diversion (refer to Table II) and the profile of original ground provided) are also shown on Drawing 2.

The stratigraphy revealed in the boreholes comprised a discontinuous veneer of topsoil or sand overlying bedrock. The strata encountered are summarized below.

Topsoil

A 220 mm thick layer of topsoil was encountered surficially in borehole 379S-3. The topsoil comprised dark brown silty sand.

Sand

Silty sand with cobbles was encountered surficially in borehole 379S-2. The sand mantled bedrock at a depth of 230 mm.

Bedrock

Bedrock was exposed surficially at borehole 379S-1 and contacted below the topsoil/sand at depths of 230 and 220 mm in boreholes 379S-2 and 3, respectively. The bedrock surface at the test locations undulates slightly ranging from elevation 250.8 to 253.1.

The bedrock was cored at each location to total depths of 15.4 to 17.9 m. Core recovery was generally greater than 90%. The RQD of the rock determined in the field as the core runs were recovered typically ranged from 50 to 100%. RQD values in the 0 to 25% range were measured in three cores recovered from Boreholes 379S-1 and 2.

A description of the recovered rock core is provided on Table I. In general, the bedrock consists of black, pink and grey gneiss. It is considered to be a high strength rock (Canadian Foundation Engineering Manual classification) that tends to split along mica-rich planes of gneissocity.

The upper 1 to 2 m of rock has deteriorated due to weathering/frost effects. Block size in the upper 4 m ranges from 50 mm to 400 mm, typically increasing with depth. The rock is essentially excellent quality with a very occasional layer (240 to 1100 mm thick) of very poor quality material, fair quality between 1.1 to 3.3 m depth in borehole 379S-2.

The bedrock is intersected by three main discontinuities: the gneissocity which dips towards the west-southwest at angles of 30 to 50°, and two near vertical joint sets. The spacing between discontinuities varies widely ranging from "close" (50 to 300 cm) to "wide" (1 to 3 m) (Canadian Foundation Engineering Manual classification). The discontinuity sets are rough, planar and tend to be continuous over a long distance. Little evidence of softening due to weathering was detected. The shear strength is considered to be high (peak friction angle estimated to be at least 60°); lower values (20 to 30°) may exist locally due to pre-sheared joints or micaceous surfaces.

The vertical rock stress is expected to be proportional to depth, increasing at a rate of about 30 kPa per meter. Horizontal stresses as high as 14 MPa are typical.

Groundwater

Approximately three weeks after coring (September 4, 1998), water was measured in core holes 379S-1 and 379S-3 at depths of 7.1 and 1.6 m (elevation 243.9 and 251.7). Water is used during coring; it is not clear whether the measured water levels reflect residual drillwater or groundwater levels. Groundwater levels are subject to seasonal fluctuations and rainfall patterns. A water level could not be obtained from core hole 379S-2 due to blockage.

Flow through the rock mass occurs only along joints. The joints are typically spaced at 300 to 400 mm intervals and appear to be moderately open. The hydraulic conductivity of the rock mass is estimated to be about 10^{-7} cm/sec, 10^{-5} cm/sec in the upper 2 m of weathered material.

CLOSURE

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

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

REGISTERED PROFESSIONAL ENGINEER
D. W. KERR
PROVINCE OF ONTARIO

Yours very truly

Peto MacCallum Ltd.

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

REGISTERED PROFESSIONAL ENGINEER
B. R. GRAY
PROVINCE OF ONTARIO

MRA:mma

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

PetoMacCallum Ltd.
CONSULTING ENGINEERS

PML Ref: 97TF088G

TABLE I

ROCK CORE DESCRIPTION
WP 402-97-01
GWP 290-97-01, Site No. 44-379

CORE RECOVERY					CORE DESCRIPTION			
BOREHOLE	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION	DISCONTINUITIES	BLOCK SIZE
379S-1	1	0.0 – 1.12	100	100	0.0 – 2.57	**GNEISS,** pink, black and grey, banded with concentrations of pink feldspar and black hornblende, some biotite mica, moderately coarse grained, (1-4) 2 mm, strong with no signs of penetrative weathering or alteration. Mod. easy to break with hammer along well-developed gneissocity.	3 sets, one formed by breaks along the gneissocity typically 50° to the core axis, and two steeply dipping sets at 5-15° to the core axis. All sets rough, planar. The steeply dipping sets are in most places iron-stained (brown) with slight penetrative weathering; however, the joint surfaces remain hard.	32 cm
	2	1.12 – 1.68	95	77				
	3	1.68 – 2.64	100	71				
	4	2.64 – 4.17	100	65				
	5	4.17 – 4.68	100	25				
	6	4.68 – 5.69	98	55				6 cm
	7	5.69 – 7.22	100	100	2.57 – 2.82	As above but more micaceous and closely jointed	Long axes of joint ellipses: acute angle to core cross-section circle (= dip for vertical drilling)	
	8	7.22 – 8.74	100	78				
	9	8.74 – 10.27	100	100			350 mm; 82° 110 mm; 65° 200 mm; 77° 54 mm; 29° 115 mm; 66° 50 mm; 20°	
	10	10.27 – 11.79	100	100				
	11	11.79 – 13.32	100	93				33 cm
	12	13.32 – 14.84	100	85	2.82 – 3.82	Pink, massive	Core dia. = 47 mm	9 cm
	13	14.84 – 15.42	100	96	3.82 – 4.92	As above but closely jointed, containing one 8 mm thick clay-filled joint at an angle of 10° to core axis at depth 3.85 (may be wash from topsoil above, but could be geological infill or gouge)		
					4.92 – 10.34 10.34 – 12.84 12.84 – 15.42	Alternating pink and grey as above		26 cm 62 cm 24 cm

RQD = Rock Quality Designation

Core Recovery data provided by Peto MacCallum Ltd.
Core Description by Franklin Geotechnical Ltd.

PetoMacCallum Ltd.
CONSULTING ENGINEERS

PML Ref: 97TF088G

TABLE I Cont'd

ROCK CORE DESCRIPTION
WP 402-97-01
GWP 290-97-01, Site No. 44-379

CORE RECOVERY

BOREHOLE	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)
379S-2	1	0.23 – 1.27	56	20
	2	1.27 – 2.82	97	50
	3	2.82 – 4.34	83*	54
	4	4.34 – 5.87	**	-
	5	5.87 – 7.39	**	-
	6	7.39 – 8.00	88	0
	7	8.00 – 8.92	100	100
	8	8.92 – 10.44	100	97
	9	10.44 – 10.64	100	100
	10	10.64 – 12.04	91	58
	11	12.04 – 13.56	100	88
	12	13.56 – 15.09	100	100
	13	15.09 – 16.38	100	100

CORE DESCRIPTION

DEPTH (m)	DESCRIPTION	DISCONTINUITIES	BLOCK SIZE
0.23 – 1.05	**GNEISS,** black, highly weathered and friable, stained brown along joints with some penetration into the rock material	3 sets, one formed by breaks along the gneissocity typically 50° to the core axis, and two steeply dipping sets at 5-15° to the core axis. All sets rough, planar, the steeply dipping sets are in most places iron-stained (brown) with slight penetrative weathering; however, the joint surfaces remain hard.	3 cm (2-12)
1.05 – 3.25	Black, pink and grey, bands of 0.2 to 0.8 m with concentrations of pink and white feldspar and black hornblende, some biotite mica. Moderately coarse-grained, (1-4) 2 mm, strong with no signs of penetrative weathering or alteration, but mod. easy to break with hammer along well-developed gneissocity.	Long axes of joint ellipses: 410 mm; 84° 116 mm; 66° 200 mm; 77° 110 mm; 65° 160 mm; 73° 52 mm; 25° core dia. = 47 mm	15 cm (4-30)
3.25 – 4.89	Dark grey to black, micaceous, moderately fissile.		18 cm
4.89 – 6.01	Pink granitic		37 cm
6.01 – 7.51	Dark grey micaceous		21 cm
7.51 – 16.38	Pink granitic		48 cm (15-128)

* Lost 250 mm length of core down hole
** Equipment malfunction

RQD = Rock Quality Designation

Core Recovery data provided by Peto MacCallum Ltd.
Core Description by Franklin Geotechnical Ltd.

PML Ref: 97TF088G

TABLE I Cont'd

ROCK CORE DESCRIPTION
WP 402-97-01
GWP 290-97-01, Site No. 44-379

CORE RECOVERY					CORE DESCRIPTION			
BOREHOLE	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION	DISCONTINUITIES	BLOCK SIZE
379S-3	1	0.22 – 1.22	95	85	0.22 – 2.78	**GNEISS,** black and white micaceous, weathered brown in places, banded with concentrations of pink feldspar and black hornblende, some biotite mica, moderately coarse grained, (1-4) 2 mm, strong with no signs of penetrative weathering or alteration. Mod. easy to break with hammer along well-developed gneissocity.	3 sets, one formed by breaks along the gneissocity typically 50° to the core axis, and two steeply dipping sets at 5-15° to the core axis. All sets rough, planar. The steeply dipping sets are in most places iron-stained (brown) with slight penetrative weathering; however, the joint surfaces remain hard.	4 cm (2-12)
	2	1.22 – 2.74	98	75				
	3	2.74 – 4.45	100	100				
	4	4.45 – 4.75	100	58				
	5	4.75 – 5.72	95	87				
	6	5.72 – 7.24	100	100				
	7	7.24 – 8.76	100	100	2.78 – 5.53	black micaceous, homogeneous, weathered, fissile.	Long axes of joint ellipses; angle between joint and core cross-section = dip	23 cm
	8	8.76 – 10.29	100	93				
	9	10.29 – 11.81	100	100			280 mm; 80° 50 mm; 20°	
	10	11.81 – 13.34	100	100			58 mm; 36°	
	11	13.34 – 14.86	100	72			54 mm; 29° gneissocity	
	12	14.86 – 16.39	100	100	5.53 – 10.20	Alternating black and pink in bands of about 0.7 m, coarse grained.	Core 47 mm dia.	42 cm
	13	16.39 – 17.91	100	100				
					10.20 – 10.44	black micaceous, stained along joints, friable and broken		8 cm
					10.44 – 17.91	Alternating black and pink in bands of 0.5 to 1.0 m, coarsely crystalline		40 cm

RQD = Rock Quality Designation

Core Recovery data provided by Peto MacCallum Ltd.
Core Description by Franklin Geotechnical Ltd.

PetoMacCallum Ltd.
CONSULTING ENGINEERS

PML Ref: 97TF088G

TABLE II

BOREHOLE DATA FROM GEOTECHNICAL INVESTIGATION FOR THE PROPOSED CP RAIL SHORT DIVERSION

WP 402-97-01
GWP 290-97-01, SITE NO. 44-379

CP Short Diversion
Datum Centre Line

9+940	6.5 LT C/L	D+100
0-100	Blk Si Tps	
100	NFP BR	
9+940	C/L	D
0-250	Blk Si Tps	
250-500	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
500	NFP Bld	
9+940	6.5 RT C/L	D-100
0-200	Blk Si Tps	
200	NFP BR	
9+960	6.5 LT C/L	D+100
0-50	Blk Si Tps	
50-350	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
350	NFP Bld Fr Wat @ 300	
9+960	C/L	D
0-300	Blk Si Tps	
300-600	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
600	NFP Bld Fr Wat @ 300	
9+960	6.5 RT C/L	D+400
0-50	Blk Si Tps	
50	NFP BR	
9+980	6.5 LT C/L	D+100
0-250	Blk Si Tps	
250	NFP BR	
9+980	C/L	D
0-150	Blk Si Tps	
150-350	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
350	NFP Bld/Poss BR	
9+980	6.5 RT C/L	D-1.20
0-150	Blk Si Tps	
150-400	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
400	NFP Bld	
10+000	6.5 LT C/L	D+400
0-150	Blk Si Tps	
150	NFP BR	
10+000	C/L	D
0	NFP BR	
10+000	6.5 RT C/L	D-1.00
0	NFP BR	
10+020	6.5 LT C/L	D+900
0-50	Blk Si Tps	
50-700	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
700	NFP Bld	
10+020	C/L	D
0-100	Blk Si Tps	
100	NFP BR	
10+020	6.5 RT C/L	D-500
0-100	Blk Si Tps	
100-700	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
700	NFP Bld/Poss BR	
10+040	6.5 LT C/L	D+800
0	NFP BR	
10+040	C/L	D
0-100	Blk Si Tps	
100	NFP BR	
10+040	6.5 RT C/L	D-1.40
0-100	Blk Si Tps	
100	NFP BR	
10+060	6.5 LT C/L	D+1.60
0-100	Blk Si Tps	
100	NFP BR	
10+060	C/L	D
0-400	Blk Si Tps	
400	NFP BR	
10+060	6.5 RT C/L	D-1.20
0-50	Blk Si Tps	
50	NFP BR	

Reference: Geotechnical Investigation, Proposed CP Rail Short Diversion, Parry Sound, January 15, 1999

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LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

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

DYNAMIC PENETRATION RESISTANCE: - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51 mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS. 0.3 m 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. 379S-1

N 5 007 954
E 279 792

PROJECT W. P. 402-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE CPR Rail Grade Separation, Site 44-379
LOCATION Station 15+696 34.1 m Lt.
BORING METHOD NQ Rock Coring
BORING DATE August 17 & 18, 1998

OUR PROJECT 97TF088G
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

GROUND ELEVATION 251.04

Soil profile: 0 to 15.42 m — BEDROCK : Gneiss. BOREHOLE TERMINATED AT 15.42m.

Sample Number	Type	Run (mm)	Recovery (%)	RQD (%)	Drill Water Return (%)
1	RC	1120	100	100	100
2	RC	560	95	77	100
3	RC	965	97	71	100
4	RC	1525	102	65	100
5	RC	510	100	25	100
6	RC	1015	98	55	0
7	RC	1525	100	100	0
8	RC	1525	100	78	0
9	RC	1525	100	100	0
10	RC	1525	100	100	0
11	RC	1525	100	93	0
12	RC	1525	100	85	0
13	RC	585	100	96	0

Column headings: 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; WATER CONTENT % 10 20 30; GROUNDWATER OBSERVATIONS AND REMARKS

Depth scale: 0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15.0, 15.42, 16.5
Elevation scale: 250, 249, 248, 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235

GROUNDWATER OBSERVATIONS AND REMARKS: 100% Drill water return to 5.10m depth, then 0% return.

NOTES:
Borehole drilled at inclination of 16 degrees to vertical towards northwest.
On September 4, 1998, water at 7.15m.

CHECKED BY: M.R.A.

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LOG OF BOREHOLE NO. 379S–2

N 5 007 933
E 279 830

PROJECT W. P. 402–97–01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE — OUR PROJECT 97TF088G
SITE CP Rail Grade Separation, Site 44–379
LOCATION Station 15+670 C/L — BORING DATE August 12 & 13, 1998 — ENGINEER M. R. Anderson
BORING METHOD NQ Rock Coring — TECHNICIAN M. Rapsey

GROUND ELEVATION 251.06

DEPTH in METRES	DESCRIPTION	ELEVATION	NUMBER	TYPE	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)	GROUNDWATER OBSERVATIONS AND REMARKS
0.23	SAND : Brown silty sand with cobbles								
	BEDROCK : Gneiss	250	1	RC	1040	56	20	00	
		249	2	RC	1550	97	50	100	
		248, 247	3	RC	1525	83*	54	100	* Lost 250mm down hole.
		246	4	RC	1525	**			** Several attempts required to recover all core due to equipment malfunction.
		245	5	RC	1525	**			
		244	6	RC	610	88	0	100	
		243	7	RC	915	100	100	100	
		242	8	RC	1525	100	97	100	
		241	9	RC	200	100	100	100	
		240	10	RC	1395	91	58	100	
		239	11	RC	1525	100	88	100	
		238, 237	12	RC	1525	100	100	100	
15.00		236							

Depth scale: 0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15.0, 16.5

Column headings: SOIL PROFILE; LEGEND; 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; WATER CONTENT % 10 20 30

continued on next page

NOTES:
Borehole drilled at inclination of 17.5 degrees to vertical towards south.

CHECKED BY: [signature]

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CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 379S-2 (cont'd)

N 5 007 933
E 279 830

PROJECT W. P. 402-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE CP Rail Grade Separation, Site 44-379
LOCATION Station 15+670 C/L
BORING METHOD NQ Rock Coring
BORING DATE August 12 & 13, 1998
OUR PROJECT 97TF088G
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	ELEVATION	NUMBER	TYPE	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)	GROUNDWATER OBSERVATIONS AND REMARKS
	GROUND ELEVATION 251.06								
15.0	BEDROCK : Gneiss	236	13	RC	1295	100	100	100	
16.38									
16.5	BOREHOLE TERMINATED AT 16.38m.	235							

Column headings also printed: SOIL PROFILE; LEGEND; 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; WATER CONTENT % 10 20 30

NOTES:
Borehole drilled at inclination of 17.5 degrees to vertical towards south.

CHECKED BY: [signature]

PetoMacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 379S-3

N 5 007 926
E 279 891

PROJECT W. P. 402-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE CPR Rail Grade Separation, Site 44-379
LOCATION Station 16+646 35.5 m Rt.
BORING METHOD NQ Rock Coring
BORING DATE August 11 & 12, 1998
OUR PROJECT 97TF088G
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

GROUND ELEVATION 253.33

Depth (m)	Description	Elevation	Sample Number	Type	Run (mm)	Recovery (%)	RQD (%)	Drill Water Return (%)
0.22	TOPSOIL: Dark brown silty sand	253						
	BEDROCK: Gneiss		1	RC	1000	95	85	100
		252						
		251	2	RC	1525	98	75	100
		250						
		249	3	RC	1700	100	100	100
			4	RC	305	100	58	100
		248	5	RC	965	95	87	100
		247	6	RC	1525	100	100	100
		246						
		245	7	RC	1525	100	100	100
		244	8	RC	1525	100	93	100
		243						
		242	9	RC	1525	100	100	100
		241	10	RC	1525	100	100	100
		240						
		239	11	RC	1525	95	72	100
15.00		238						

Depth scale: 0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15.0, 16.5

Column headings: SOIL PROFILE: DEPTH in METRES, DESCRIPTION, LEGEND, ELEVATION; SAMPLES: NUMBER, TYPE, N - VALUES BLOWS/0.3m; 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; WATER CONTENT % 10 20 30; GROUNDWATER OBSERVATIONS AND REMARKS

continued on next page

NOTES:
Borehole drilled at inclination of 17 degrees to vertical towards northeast.
On September 4, 1998, water at 1.60m., (possible drill water).

CHECKED BY: MRA

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CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 379S-3 (cont'd)

N 5 007 926
E 279 891

PROJECT W. P. 402-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE CPR Rail Grade Separation, Site 44-379
LOCATION Station 16+646 35.5 m Rt.
BORING METHOD NQ Rock Coring
BORING DATE August 12 & 13, 1998

OUR PROJECT 97TF088G
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

GROUND ELEVATION 253.33

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	SAMPLE NUMBER	SAMPLE TYPE	BLOWS/0.3m N - VALUES	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)	LIQUID LIMIT W_L / PLASTIC LIMIT W_P / WATER CONTENT W (10, 20, 30)	GROUNDWATER OBSERVATIONS AND REMARKS
15.0	**BEDROCK** : Gneiss		238									
				12	RC		1525	105	100	100		
			237									
16.5												
			236	13	RC		1525	100	100			
17.91												
18.0	BOREHOLE TERMINATED AT 17.91m.											

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

NOTES:
Borehole drilled at inclination of 17 degrees to vertical toward northeast.
On September 4, 1998, water at 1.60m (possible drill water).

CHECKED BY: MRA

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DISTRICT No. 52 CONT No. 2000-0236 WP No 402-97-01	
HIGHWAY 69 - CPR GRADE SEPARATION BOREHOLE LOCATION PLAN	SHEET 186

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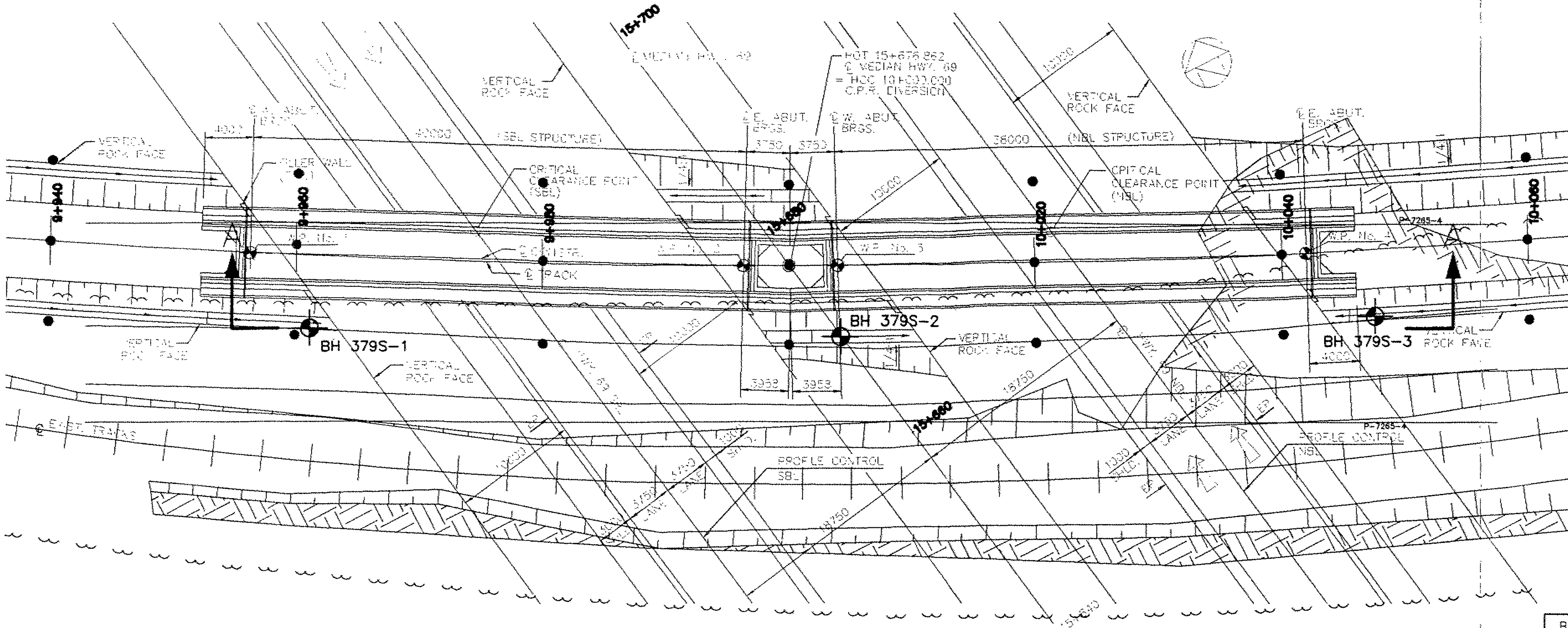
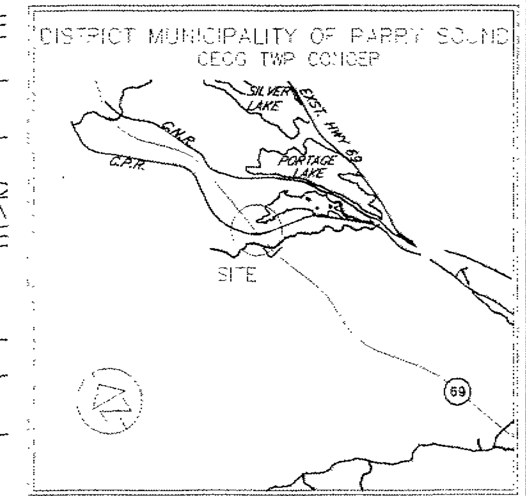
45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C6



BOREHOLE LOCATION PLAN

4m 0 4m



KEY PLAN

1km 0 1km

BOREHOLE	NORTHING	EASTING	ELEVATION
BH 379S-1	N 5 007 954	E 279 792	251.04
BH 379S-2	N 5 007 933	E 279 830	251.06
BH 379S-3	N 5 007 926	E 279 891	253.33

LEGEND

BOREHOLE & ROCK CORE

BOREHOLE FROM GEOTECHNICAL INVESTIGATION FOR PROPOSED CP RAIL SHORT DIVERSION (REFER TO TABLE II)

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.
3. REFER TO DRAWING 2 FOR SOIL PROFILES AND CROSS SECTIONS.

PROPOSED CROSSING
AT
PROPOSED C.P.R. ALIGNMENT
AND
KING'S HIGHWAY 69
DISTRICT MUNICIPALITY OF PARRY SOUND

LOT 4 GEOG TWP CONGER	SITE 44-379	CON 6 TWP OF HUMPHREY

	DATE	SCALE	JOB NO.	DRAWING NO.
DRAWN CB CHECKED MRA APPROVED DWK	APRIL 1999	AS SHOWN	97TF088G	1

DISTRICT No. 52 CONT No. 2000-0236 WP No 402-97-01	
HIGHWAY 69 – CPR GRADE SEPARATION	SHEET 187
SOIL PROFILES	

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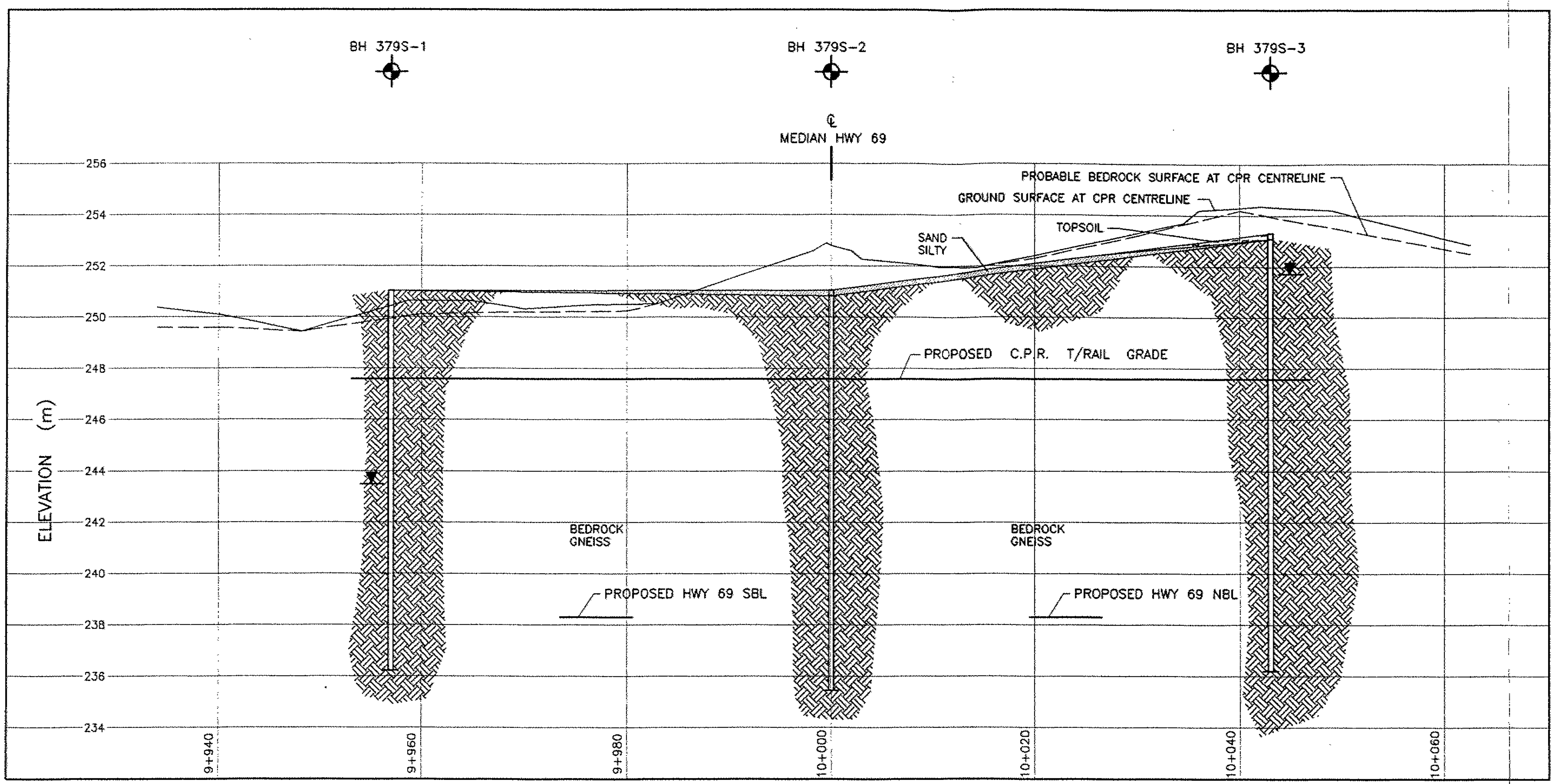
45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C6



SECTION A–A

2m 0m 2m VERT
5m 0m 5m HOR

LEGEND

BOREHOLE AND ROCK CORE

OBSERVED WATER LEVEL (POSSIBLE DRILL WATER, 17 TO 23 DAYS AFTER CORING)

NOTE

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

PROPOSED CROSSING AT
AT
PROPOSED C.P.R. ALIGNMENT
AND
KING'S HIGHWAY 69
DISTRICT MUNICIPALITY OF PARRY SOUND

LOT 4 GEOG TWP CONGER — SITE 44-379 — CON 6 TWP OF HUMPHREY

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

		DATE	SCALE	JOB NO.	DRAWING NO.
DRAWN	CB				
CHECKED	MRA	APRIL 1999	AS SHOWN	97TF088G	2
APPROVED	DWK				

FOUNDATION DESIGN REPORT
FOR
CP RAIL GRADE SEPARATION
W.P. 402-97-01
G.W.P. 290-97-00, SITE 44-379
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

Job No. 97TF088G
Geocres No. Not Assigned

July, 2000

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TABLE OF CONTENTS

Peto MacCallum Ltd.
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FOUNDATION DESIGN REPORT
For
CP Rail Grade Separation
W.P. 402-97-01
G.W.P. 290-97-00, Site 44-379
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 CP Rail grade separation structure over the proposed Highway 69 (Station 15+677 Highway 69 Chainage).

The General Arrangement drawing for this structure, dated February, 1999, indicates the underpass will consist of two single span bridges. The rail grade will be near elevation 247.6, some 3.5 to 6.0 m below existing grade. The proposed four lane Highway 69 will be constructed near elevation 238.3, 13 to 15 m below existing grade and 10 m below the proposed rail grade. An approximate 10 m wide rock ridge will remain unexcavated along the centre median. The abutment foundations will be constructed near the road grade level.

The subsurface stratigraphy revealed at the bridge site comprised a discontinuous veneer of topsoil or sand overlying bedrock at depths of 0 to 230 mm.

FOUNDATIONS

Based on the borehole information, it is considered that the structures may be supported on conventional spread footings founded on bedrock. Foundations bearing on sound bedrock at or below elevation 247.6 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 extreme east and west abutment footings of the two bridges should be founded below a line inclined upwards at 1:2 (H:V) from the toe of the Highway 69 cut and set back 2 m from the face of the rock. Footings for the centre two abutment footings may be constructed on the rock "ridge" along the median provided they are founded below a line inclined upwards at 1:1 from the toe of the excavation and the edge of footing is at least 2.0 m from the rock excavation face.

Excavation of the rock to the proposed founding level should be carried out in a manner which provides a level founding surface. Mass concrete could be placed to level minor variations in the founding surface.

It is important that blasting/excavation of the rock in the vicinity of the structure is controlled to minimize disturbance of the rock surface on which footings will bear. Recommendations for rock excavation are presented in a subsequent section of this report.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the bedrock. An unfactored friction factor of 0.6 is recommended for footings on bedrock. A value of 0.7 may be used for a roughened bedrock surface (asperity height of at least 25 mm) created by mechanical means/during rock excavation.

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. Eliminating the footing and providing overturning resistance using dowels into rock could also be considered.

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

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

If backfill is placed behind the abutment walls, they 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	42
Unit Weight (kN/m^3)	22.8	21.2	18.0
Active Earth Pressure Coefficient (K_a)	0.27	0.31	0.20
At Rest Earth Pressure Coefficient (K_o)	0.43	0.47	0.33
Passive Earth Pressure Coefficient (K_p)	3.69	3.25	5.04

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 the abutment concrete is poured neat against the cut rock face, lateral pressures exerted on the abutment wall will be negligible provided measures are implemented to prevent the build-up of hydrostatic pressure behind the concrete. This can be accomplished by placing a drainage membrane on the rock prior to concreting and weep holes at the base of the wall to allow the water to drain.

APPROACH EMBANKMENT

Approach fill and embankment construction is not anticipated since the railway diversion will be constructed in a rock cut.

EXCAVATION AND GROUNDWATER CONTROL

Excavation for construction of Highway 69, the rail diversion and structures is expected to be carried out within bedrock. Considering the type of bedrock and geometry of cut, it is anticipated that blasting will be required to excavate the rock.

The blasting and excavation operations should be conducted in accordance with Ontario Provincial standard procedures with operational constraints imposed to minimize the potential for damage to the existing CP Rail track and the grade separation structure.

The following operational constraints should be imposed:

i) Existing Railway Track

blast induced vibration at the edge of ballast nearest the blast must not exceed 100 mm/s.

ii) New Structure

blast induced vibration of the rock adjacent to structure foundations must not exceed the value computed from the following equation

$ppv < 0.4H + 10 < 100$
ppv = peak particle velocity (mm/s)
H = age of concrete (hours)

iii) Blast Attenuation Coefficients

The blast attenuation coefficients must be determined by monitoring at least four representative blasts before blasting within 200 m of the track in order to be able to compute/ establish the distance L at which normal blasts may cause vibrations at the track to exceed 100 mm/s.

iv) Blast Monitoring

The blast induced vibrations at the railway track must be monitored during all blasts within a distance of 2 L of the track.

v) Controlled Blasting

> Terminate normal blasting when the blast induced vibrations at the track exceed 100 mm/s and implement reduced scale blasting procedures and/or mechanical excavation procedures to excavate the rock.

The specifications should also call for the contractor to protect the rail from damage due to "flying" blast rock (e.g. blasting mats) and any damage repaired immediately.

It is the responsibility of the contractor to schedule his work and design the blasts to ensure that the work is conducted in accordance with the specification without exceeding the permitted vibration levels. A blasting consultant with at least five years full-time working experience providing blast consultant services should be retained to evaluate trial blast procedures, recommend blast design, and monitor blast vibration/attenuation.

It is anticipated that monitoring of blast induced vibrations at the track will be required when the excavation face is within 50 to 75 m of the track and reduced scale blasting will be required within 35 m.

Rock excavation in the vicinity of the structure must be controlled to minimize disturbance of the rock surface on which footings will bear. In this regard, methods such as line-drilling of unloaded vertical holes, closer blasthole spacing, reduced scale blasting, and/or rock reinforcement with dowels prior to blasting should be considered in foundation areas.

Water was observed in two core holes at depths of 7.1 and 1.6 m about three weeks after drilling. It is not clear whether the water comprised groundwater or residual drillwater. Flow through the rock mass occurs only along joints. Seepage or surface water which enters the excavation should be readily handled by conventional sump pumping techniques.

Groundwater levels are subject to seasonal fluctuations and rainfall patterns. Some ice build-up on the rock face should be expected during the winter months.

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

CLOSURE

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

Yours very truly

Peto MacCallum Ltd.

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

Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAFT
CP RAIL DIVERSION
MILE 6.63 PARRY SOUND SUBDIVSION
STATION 15+662, HIGHWAY 69
HUMPHERY/CONGER, ONTARIO
FOR
69 JOINT VENTURE

Distribution:

3 cc: McCormick Rankin Corporation
1 cc: Totten Sims Hubicki Associates
1 cc: Franklin Geotechnical Ltd. (Letter Only)
1 cc: PML Hamilton
1 cc: PML Toronto

Job No. 97TF088G

September, 1998

Peto MacCallum Ltd.
CONSULTING ENGINEERS

September 16, 1998

Our Ref: 97TF088G

Mr. Paul Turner
McCormick Rankin Corporation
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

DRAFT

Dear Mr. Turner

CP Rail Crossing
Mile 6.63 Parry Sound Subdivision
Station 15+662, Highway 69
<u>Humphery/Conger, Ontario</u>

This letter provides additional preliminary comments/recommendations concerning design/construction of the CPR crossing of Highway 69.

Three testholes were drilled along the alignment of the bridge of the proposed short diversion during the period August 10 to 21, 1998. The holes were drilled at an inclination of 16.0 to 17.5° from vertical at the proposed pier/abutment locations 10 to 12 m north of the centerline of the existing track.

A detailed description of the bedrock core retrieved from the testholes along with preliminary comments/recommendations concerning the engineering aspects of design of the Highway 69/CPR Grade Separation Structure is provided in Report No. G 819.2 dated September 4, 1998 prepared by Franklin Geotechnical Ltd. (FGL). A copy of the report accompanies this letter.

This letter highlights salient aspects of the report and provides additional comments for your consideration:

1) For engineering purposes the rock in the subject area is called "gneiss". It is considered to be a high strength rock (Canadian Foundation Engineering Classification System) that tends to split along mica-rich planes of gneissocity.

   The upper 1 to 2 m of the rock has deteriorated due to weathering/frost effects; block size in the upper 4 m ranges from 5 to 40 cm.

   The bedrock is intersected by three main discontinuities which dip towards the west-southwest at angles of 30 to 50° as well as two near vertical joint sets. The spacing between discontinuities varies widely ranging from "close" (5 to 30 cm) to "wide" (1 to 3 m) (Canadian Foundation Engineering Classification System).

...2

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BARRIE, BRAMPTON, HAMILTON, KITCHENER, TORONTO

The discontinuity sets are rough, planar and tend to be continuous over a long distance; little evidence of softening due to weathering was detected. The shear strength is considered to be high (peak friction angle estimated to be at least 60°); lower values (20 to 30°) may exist locally due to pre-sheared joints or micaceous surfaces.

2) The vertical rock stress is expected to be proportional to depth, increasing at a rate of about 30 kPa per meter. Horizontal stresses as high as 14 MPa are typical.

The rock stress could have an influence on blasting and cause some differential movement along rock joints.

3) The water level in the boreholes three weeks following the completion of drilling ranged from 1.6 to 7.1 m below grade at the test locations. It is not clear whether the measured water is groundwater or water used during the drilling operation.

The joint sets at depth are typically spaced at 30 to 40 cm intervals and appear to be moderately open; the hydraulic conductivity of the rock mass is estimated to be about 10^{-7} cm/sec.

Some ice buildup on the approximate 11 m high rock face during the winter months should be expected.

4) The tender document should clearly indicate that:

- Blast induced vibrations must be controlled to avoid track damage. The actual blast design employed is the responsibility of the contractor.

- The contractor must retain a blasting specialist to provide advice at the bid stage as well as during excavation (blast design, blast monitoring).

- A greater than usual amount of oversize blocks that require secondary blasting or breakage before removal from the excavation may be required. what is usual?

  In addition, overbreak and/or underbreak is likely to be greater than usual due to the anisotropic strength and slabby jointing of the rock. ?

5) The excavation specifications should be based on OPSS standards and the Special Provisions provided in Enclosure 1.

6) We believe a limiting peak particle velocity (ppv) of 100 mm/sec. is appropriate to prevent damage to the track.

Routine excavation procedures should be suitable when the excavation is at least 50 m from the track:

- benching near the mid height of the excavation if the rock face is greater than 10 m high; bench to be about 3 m wide.
- cut slope inclined at about 0.25 horizontal to 1 vertical.
- pre split perimeter holes spaced at 0.75 m centers.
- inspect rock face and install rock bolts/shorcrete locally as required to support the cut slope. Maximum height of face to be 3.5 m before inspection/support is performed.

OK. slope maintenance

It is imperative that blast monitoring is performed when the excavation face is 50 to 75 m from the track to establish criteria for the weight of explosive, the blast delay and the attenuation coefficients of the rock when working within 50 m of the track.

Special procedures must be implemented when excavating within 50 m of the track.

OK.

- monitor blast induced vibrations
- adopt reduced scale blasting procedures to limit blast induced vibrations to 100 mm/sec. as excavation proceeds toward the track by controlling the weight of explosive, blasting delay and depth of blast
- when blast induced vibrations exceed the 100 mm/sec. criteria using reduced scale blasting procedures, adopt mechanical methods of excavation
  - line drilling and splitting using mechanical and/or hydraulic wedges or a large hoe ram
  - large hoe ram
  - large excavator with tiger toothed bucket

Light blasting could be employed to expedite excavation by mechanical methods if blast induced vibrations can be maintained below the specified criteria.

It is noteworthy that a tiger toothed bucket cannot produce a smooth face but should be suitable if the cut face is thoroughly scaled and cut back at an inclination of 1:1 for stability purposes.

Refer to Enclosure 2 for a general description of the excavation protocol.

7) The optimum separation distance between the proposed and existing track alignment will, in general, be dictated by the cost of construction, duration of construction and potential impacts to railway operations. Construction of both the highway 69 cut and the rail diversion cut must be considered.

It is anticipated that normal blasting procedures can be employed to within about 35 m of the track without exceeding the limiting ppv of 100 mm/sec. When the ppv criteria is exceeded, it will be necessary to modify the blasting procedure and ultimately adopt mechanical methods of excavating the rock immediately adjacent to the track.

The cost of excavation by mechanical methods assisted by light blasting is expected to be at least five times more than that using normal blasting procedures. The <u>average</u> cost of excavation within 35 m of the track is likely to be about two times higher than the cost of normal blasting procedures.

It is noteworthy that the cost of excavation along the rail diversion provided in the preliminary FGL report is based on an 11 m depth of cut. While the surface profile along the rail diversion has not been provided, it is expected that the depth of cut will typically range from 0 to 6 m. hence the excavation cost is likely overestimated by a factor of 2. This figure will be revised in the final report.

8) Cognizant of the length of the rail diversion, the depth of rock cut along the diversion and the time required to complete this work, we believe the original undercut alternative should be re-evaluated.

Refer to Enclosure 3 for a conceptual description of the work protocol.

The cost of construction of the undercut alternative and time required to complete the construction is expected to be substantially less than the diversion alternative.

...5

DRAFT

We trust this synopsis of the Franklin Geotechnical Ltd. report is useful and look forward to any questions you may have.

Sincerely

Peto MacCallum Ltd.

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

DWK:mma

3 cc: McCormick Rankin Corporation
1 cc: Totten Sims Hubicki Associates
1 cc: Franklin Geotechnical Ltd.

DRAFT

ENCLOSURE 1

Blasting Specifications – Special Provisions
(Section 3.4, pages 5 and 6 of Franklin Geotechnical Ltd. report)

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CONSULTING ENGINEERS

BLASTING SPECIFICATIONS – SPECIAL PROVISIONS

(Section 3.4, pages 5 and 6 of Franklin Geotechnical Ltd. report)

- Retain the services of a blasting consultant to monitor blast vibrations, measure the blast attenuation constants for the site, recommend blast designs, evaluate trial blasting and on-site blasting procedures to ensure that the peak particle velocity does not exceed 100 mm/s at the CP track. Submit copies of all blasting measurements and recommendations to the Engineer.

- Determine the blast attenuation constants H and b for the site by monitoring four representative Hwy 69 blasts. For each of these blasts record the maximum weight of explosives W in any single delay and the distances D from that delay to four blast vibration monitors, equally spaced over a range of distances up to 400 m from the point of detonation. Record and measure the maximum peak particle velocity (ppv) for vertical, radial and transverse vibrations. These measurements to be completed by a blasting specialist and the results reported before blasting approaches closer than 300 m from the CP track

- Prepare blast designs for typical blasts and determine the distance L that full-scale blasting can approach the track without exceeding 100 mm/s peak particle velocity.

- Blasts closer than the limiting distance L from the track are to be designed with reduced bench heights, burdens, spacings and quantities of explosive per delay to limit vibration levels at the track to less than 100 mm/s. Each is to be monitored to ensure that the limit is not exceeded in vertical, transverse or radial directions. Submit the design to Construction Supervisor 24 hours before drilling or loading the blast.

- Production holes to be spaced at Contractor's discretion so as to limit vibrations and achieve fragmentation. Maximum height of lift limited to 7 m to ensure accurate drilling. Maximum blasthole diameter not to exceed 75 mm.

- During mucking, the exposed height of rock face is not to exceed 3.5 m before inspection and completion of any stabilization work.

- Stemming and blasting mats to be used to prevent fly rock, and blasting mats are to be placed across the track to protect the rails where there is a risk of the blast dislodging rock from faces adjacent to the track. The contractor is to remove blasting mats and any rock fragments from the track within 30 minutes of each blasting operation.

ENCLOSURE 2

Excavation Procedures – Rail Diversion
(Section 6.1, pages 8 and 9 and Figure 8
of Franklin Geotechnical Ltd. report)

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EXCAVATION PROCEDURES – RAIL DIVERSION

(Section 6.1, pages 8 and 9 and Figure 8
of Franklin Geotechnical Ltd. report)

1. Excavate Hwy 69 from the north to within 15 m of the CP track.

2. To limit vibrations, all excavation within 50 m of the track requires blast monitoring with multiple benches of reduced height, reduced blasthole spacing and reduced explosives per delay;

3. Scale the rock face along the north side of the track and install rockbolts to prevent track heave and rockfalls from adjacent blasting. Install heave gauges at 50 m intervals over about 550 m

4. Construct a bridge across the Hwy 69 cut ~~20 m~~ from the rail track.

5. Excavate ~~a 1 km long~~ rail diversion cut ~~6 to 11 m deep over about 500 m and 0 to 6 m deep over about 500 m~~. Excavation requires a presplit perimeter and reduced-scale bench blasting to limit vibrations to less than 100 mm/s ppv.

6. Wherever needed, place blasting mats across the track to protect the rails against fly rock and rockfalls. Monitor all blasts, and after each blast remove the blasting mat and any fallen rock, and inspect and gauge the track.

7. During bench excavation, inspect rock faces along the new cut, scale and spot-bolt or shotcrete where needed.

8. Place ballast, lay track, and divert rail traffic across the new bridge.

9. Complete the remaining section of Hwy 69 south of the new CP track. All blasting within 300 m of the track requires blast monitoring, and reduced-scale blasting is to continue for as long as necessary to limit ppv to less than 100 mm/s

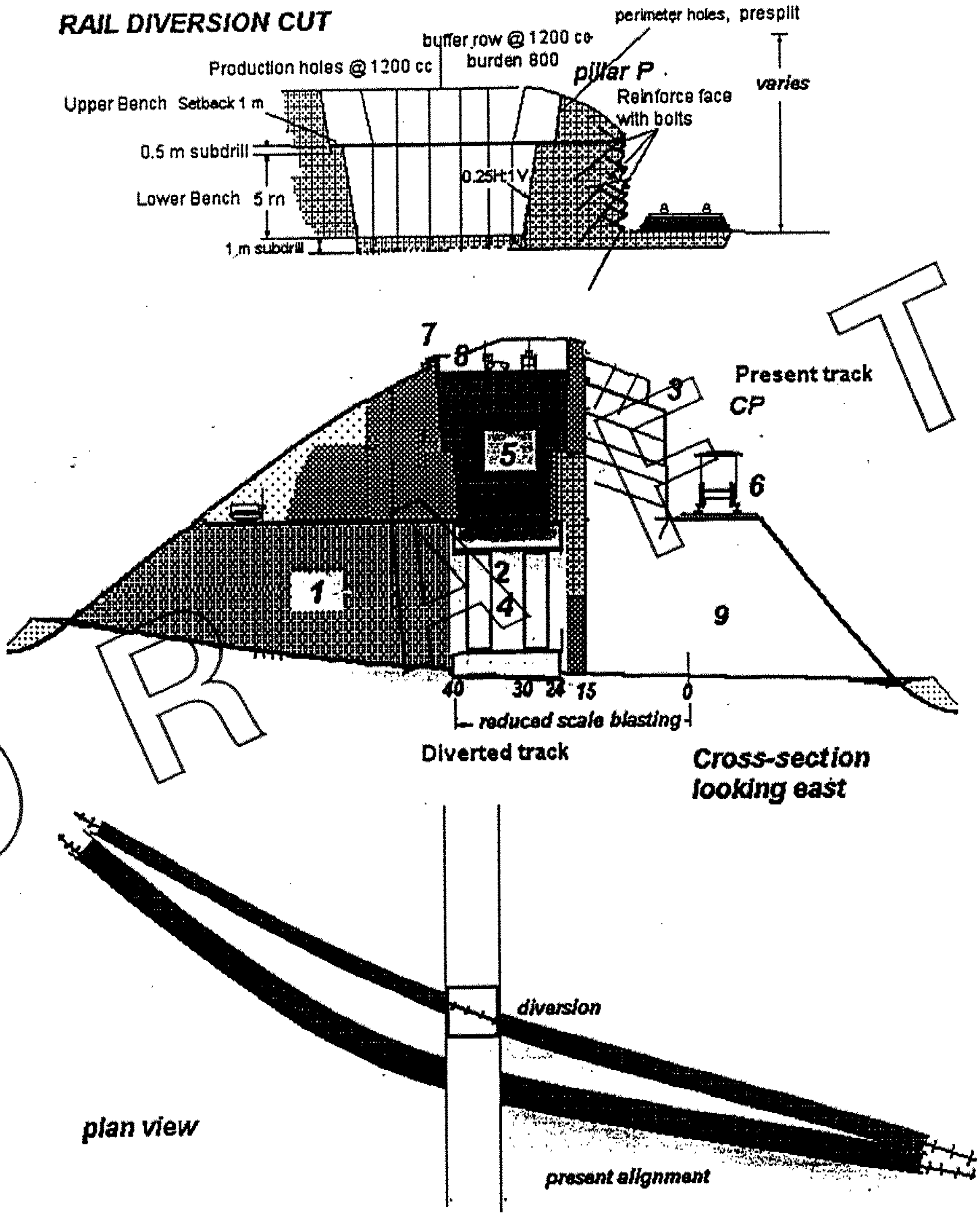

Fig. 8 Diversion alternative: construction sequence

	NAME	DATE
Drawn By	Massoud Palassi	3 Sept. 1998
Checked By	John A. Franklin	3 Sept. 1998
Revisions		

franklin geotechnical engineering

PROJECT:	FIGURE:
G819.2	8

ENCLOSURE 3

Excavation Procedures – Undercut Alternative
(Section 7.1, pages 10 and 11 and Figure 11
of Franklin Geotechnical Ltd. report)

Peto MacCallum Ltd.
CONSULTING ENGINEERS

EXCAVATION PROCEDURES – UNDERCUT ALTERNATIVE

(Section 7.1, pages 10 and 11 and Figure 11
of Franklin Geotechnical Ltd. report)

1. Scale along track 100 m either side of the crossing location to remove loose rock and prevent rockfalls during the work. If necessary install rockbolts.

2. Excavate Hwy 69 from the north and south until the cut reaches the CP track. In the last 50 m (or whatever distance is confirmed as safe from site attenuation measurements). Use reduced scale blasting to limit vibration levels to less than 100 mm/s ppv.

3. Presplit rock faces parallel to the track in a single vertical lift. Progressively expose these faces by reduced scale charges. Stockpile broken rock for use in ramping to maintain access to these faces as required to maintain access for horizontal drilling for bolts, and blastholes as described below.

4. Shotcrete the faces from the top down as they become exposed.

5. Install 3 m long fully resin-grouted rockbolts horizontally at 1.5 m centres to reinforce the central pillar and abutments.

6. Drill closely spaced (0.3 m centres) presplit holes horizontally along the faces of the central pillar and abutments using a multiple-boom jumbo drill.

7. Drill blastholes from one or both sides of the 7 m high 10 m thick rock web beneath the track, in a horizontal V-cut pattern as shown in Fig. 11. (This figure shows drilling from one side only, but drilling from both sides will give better release with reduced vibrations on the pillar and abutments).

8. Close the track for an estimated 13 hr to 1 day maximum to blast and construct a bridge deck:
9. Lift track. (1 hr)
10. Load and detonate horizontal blastholes. (3 hr)
11. Muck out upper 2 m leaving broken rock in place below that.(2 hr)
12. Trim footings of bridge beams and install vertical rockbolts in pillar and abutments. (2 hr)
13. Lay bridge beams on bedding of epoxy concrete. (2 hr)
14. Replace ballast and track. (2 hr)
15. Inspect, gauge and reopen track (1 hr) (estimated total 13 hrs)

16. Pattern bolt and shotcrete east and west rock faces of the undercut, mucking out progressively until the complete height is exposed. Trim-blast the rock floor where necessary using blasting mats to protect bridge deck. (Hoe ram excavation could be used as an alternative). Place granular base and asphalt. Open Hwy 69.

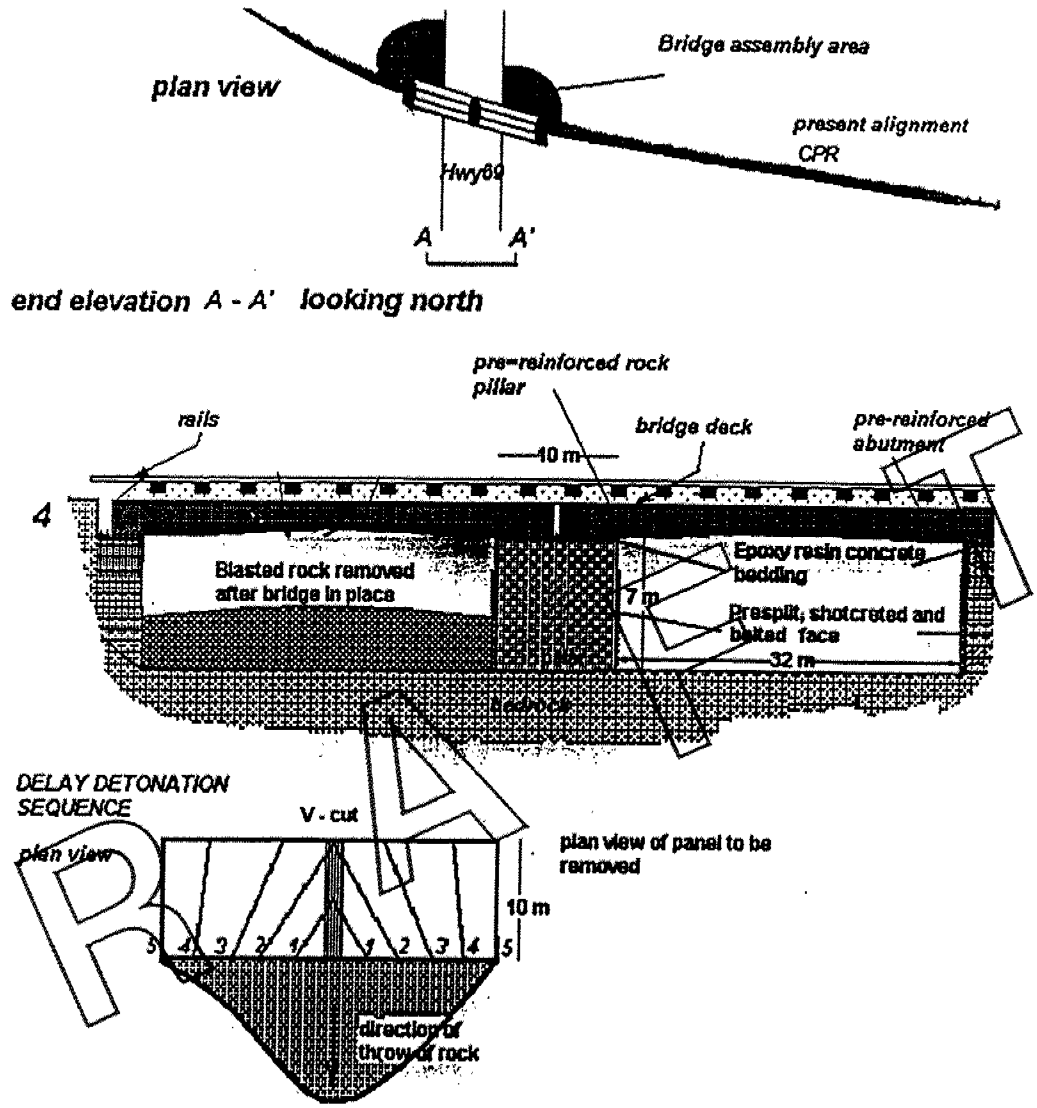

Fig. 11 Undercut alternative: construction sequence and blasting costs

	NAME	DATE
Drawn By	Massoud Palassi	3 Sept. 1998
Checked By	John A. Franklin	3 Sept. 1996
Revisions		

franklin geotechnical engineering

PROJECT: G819.2 FIGURE: 11

Highway 69 / CPR Grade Separation
Report on Rock Engineering Aspects of Design

Prepared for:

PETO MACCALLUM LTD.
45 Burford Road
Hamilton, Ontario
L8E 3C6

Prepared by:

FRANKLIN GEOTECHNICAL LTD.
The Stream
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Report G819.2

4 September, 1998

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Peto MacCallum Ltd.
45 Burford Road
Hamilton, Ontario
L8E 3C6

4 September, 1998

Attention: Mr. Dennis Kerr, P. Eng.

CPR Grade Separation, Highway 69

Please find attached four copies of our Report G819.2, which we trust is in accordance with our proposal of 17 April 1998 and your requirements. We much appreciated the opportunity of working for you on this project, and will be glad to assist further as required.

Sincerely,
FRANKLIN GEOTECHNICAL LTD.

John A. Franklin, Ph.D., P.Eng., CPGS
President

Attached: Report 819.2

JOHN A. FRANKLIN
ASSOCIATION OF PROFESSIONAL GEOLOGICAL SCIENTISTS
1963
CPGS
#3662

REGISTERED PROFESSIONAL ENGINEER
J. A. FRANKLIN
PROVINCE

SUMMARY

This report reviews rock conditions and their impact on the methods, costs and feasibility of constructing a grade separation. The site is at the intersection of relocated Highway 69 with the CP Rail track in Conger Township 3 km north of MacTier, Ontario.

Northwest trending outcrop ridges, height 5 to 15 m, are separated by bedrock troughs containing thin sandy till soils and bog. The rock is Precambrian gneiss with the high strength typical for the Canadian Shield, but anisotropic, and splits quite easily along mica-rich partings. The gneissocity dips 30° to 50° to the west-southwest (strike 119°). The upper 1 to 2 m of the rock mass has a reduced joint spacing, becoming more widely spaced at depth. Two near-vertical joint sets striking at 65° and 146° combine with the gneissocity to form prismatic slabby blocks The near-vertical joints are continuous, planar, stained, but hard, strong and interlocking. Joint shear strengths are assessed as high except for occasional mica-rich seams and a single joint in the core, which may contain 1 cm, of clay fault gouge.

Precautions to protect the track an ensure that the CP rail service is uninterrupted include detailed planning, rock stabilization, control of blast vibrations and fly rock, and inspection and gauging of the track. Recommendations are given on each aspect. The contractor is required to prepare blast designs for typical blasts and determine the distance L that full-scale blasting can approach the track without exceeding 100 mm/s peak particle velocity. That distance is estimated as 50 m. Blasts closer than this to the track are to be designed with reduced bench heights, burdens, spacings and quantities of explosive per delay to limit vibration levels at the track to less than 100 mm/s. Each is to be monitored to ensure that the limit is not exceeded in vertical, transverse or radial directions.

The two diversion alternatives have been explored to the extent of determining that the methods are considered feasible and safe. The methods for the rail diversion involve standard excavation and support practices with a generous safety margin allowed. Similarly the method for the undercut requires only proven technology such as drilling and blasting methods long established in tunnelling. A generous allowance has been made for rock stabilization and support, by scaling, fully resin-bonded rockbolts and silica fume shotcrete. The following is a comparison of cost estimates:

1 Rail diversion alternative:

Hwy 69 last 50 m excavation:	$2,424,710
Diversion excavation:	$3,527,826
Stabilization:	$110,629
Total:	$6,063,165

2 Undercut alternative:

Hwy 69 last 50 m excavation:	$2,795,915
Undercut excavation:	$845,164
Diversion excavation:	$3,527,826
Stabilization:	$110,629
Total:	$6,063,165

These estimates are preliminary and require checking with regard to unit prices, quantities, and dimensions of the cuts. Bridge construction and other costs should be added, however, they are considered sufficiently reliable to compare the two options.

The undercut represents substantial cost savings and is likely to take less than one month to complete in comparison to several months for the rail diversion alternative.

TABLE OF CONTENTS

LIST OF FIGURES AND APPENDICES

1. INTRODUCTION

Ministry of Transportation Contract WP398-97-01 includes a grade separation to be constructed at the intersection of relocated Highway 69 with the CP Rail track in Conger Township 3 km north of MacTier (1).

Franklin Geotechnical Ltd. (FGL) have been engaged by Peto MacCallum Ltd to assist in rock engineering aspects of the site investigation and design. The scope of work was defined in FGL proposal P302 of 17 April 1998, and includes the following items:

- Determine the rock mass characteristics at the site by studies of maps, air photographs, rock outcrops, and rock core obtained from three exploratory holes;
- Define alternative excavation/construction sequences and compare their feasibility and cost;
- Establish criteria for protecting the CP rail track from blast disturbance. Determine the maximum vibration level (peak particle velocity ppv) that can be tolerated by the rail track, and the relationship between scaled distance and ppv for the site. Investigate alternative blast designs and specify precautions to limit ppv within acceptable limits;
- Determine how close to the track blasting is likely to be able to approach; plan and specify blast monitoring to be conducted during construction;
- Consider alternative methods of excavation such as line drilling, splitting or hoe-ram breakage;
- Identify other potential hazards to the CPR track and highway 69 including rock falls, rock squeeze or heave etc. Design stabilization or other measures to avoid or minimise these hazards;
- Contribute to specifications for the rock work.

Work under these terms of reference has now been completed and the results are presented in this report.

We attended a meeting at the CPR offices in Sudbury on 18 April 1998 to review grade separation requirements, and visited the site the same day to inspect and take photographs of rock outcrops (Appendix 3). Three exploratory coreholes were subsequently drilled under the superrvision of Peto MacCallum Ltd. at the site, inclined to sample the steeply dipping joint sets identified in outcrop and lineament studies:

Hole #	1.	2	3
Offset from centreline of track (m)	11.9	10.4	12.3
Direction of drilling:			
Degrees down from horizontal;	16°	17.5°	17°
Azimuth clockwise from North	320°	190°	060°
Elevation above rail (m)	3.273	3.304	5.515
Length of drilling (m)	15.48	16.13	17.90

The rock core was examined and logged by Dr. John Franklin, a geological engineer. Core logs are given in Appendix 1 and core photographs in Appendix 2.

2. ROCK MASS CHARACTERISTICS

2.1 Soil cover and bedrock topography

The region is characterized by a series of northwest-trending bedrock ridges and troughs. The ridges generally are bare rock outcrop, and the troughs are typically filled with soils just one or two metres thick, but occasionally several tens of metres.

The regional soils are shown on Map P3103, reproduced in Fig. 2, as Unit 2a "thin discontinuous till over bedrock" overlain by ribbons of Unit 8 "organic deposits, peat, wetlands" along creeks. The CP rail track at the site location is cut into the side of one of these ridges which forms a rock face about 4 m high on the north side of the track and a soil-filled trough on the south side.

2.2 Bedrock

For engineering purposes the rock in this area may be called a "gneiss". This simpler name is chosen in preference to the geological description "banded hornblende migmatite, moderately granitized" (Unit 5dy on Map 2118). A migmatite is a zone of mixed granite and country rock, in this case likely of sedimentary origin but highly metamorphosed with few traces of the sedimentary structures remaining. A gneiss is a rock formed by recrystallization from any other, colour-banded and with less than 50% mica (a "schist" contains more than 50% mica).

As shown in the drillhole logs (Appendix 1) the rock at the site consists of pink, black and grey varieties of gneiss, the colour variations being caused by concentrations of pink and/or white feldspar, black hornblende and some biotite mica. The rock is moderately coarse grained, with visible crystals. The mica flakes have sub-parallel alignment and occur both throughout the matrix and concentrated in veins.

The strength and microfabric are anisotropic, i.e. the rock is generally strong (estimated as "High Strength, 50 to 200 MPa using the classification of the Canadian Foundation Engineering Manual) but tends to split along mica-rich planes of gneissocity. Mica content ranges from near-zero in the pink granitic gneisses to an estimated 30-50% or the black gneisses and more along mica-rich bands, seams and partings.

The rock quality has deteriorated in the upper 1 to 2 metres as result of weathering and frost action over millions of years. Fig. 4 shows a reduction in block size from 40 cm down to 5 cm in the upper 4 metres of drillholes 2 and 3, caused by weathering of the more micacous black gneiss. The mica-rich zones tend to be more porous, weathered, fissile and friable than the pink granitic rock types.

2.3 Jointing

The rocks on site are intersected by three main sets of discontinuities, the gneissocity, which typically dips towards the west-southwest at angles of 30 to 50 degrees, plus two near-vertical joint sets (Fig. 3 and 4).

Most of the breaks observed in the core occur along gneissocity joints, and most of these coincide with mica-rich partings. The dip of gneissocity can fluctuate by ten or twenty degrees over distances of just a few metres as can be seen in the Appendix 3 outcrop photographs. The typical core stick length was 40 cm except in the upper weathered parts of Drillholes 2 and 3 where 3 to 5 cm was typical. The longest core stick was 1.28 m in the pink granitic gneiss of Drillhole 2. This gives an estimated spacing of about 15 cm measured perpendicular to the gneissocity, and a maximum spacing of about 0.7 m which is similar to the thickness of slab observed in outcrop. The spacing designation covers a wide range from "close" (5 to 30 cm), through "moderately close" (0.3 m to 1 m) to "wide" (1 m to 3 m) using the classification of the Canadian Foundation Engineering Manual).

Rock Quality Designation RQD is another way of expressing the intensity of jointing. It is the percentage of core stick longer than 10 cm. RQD values were close to 100% in much of the core, although with some more closely jointed zones. As can be seen in the lower left graph of Fig. 4, RQD becomes 100% for block size greater than 20 cm so is not very useful for relatively widely spaced joints such as those at this site. The above block size is a better index.

Lineaments are linear portions of lake edges, streams, topographic ridges etc. corresponding with major regional joint and fault directions or changes in rock hardness. In Fig. 3, the locations, lengths and directions of lineaments measured in the vicinity of the site are shown relative to the CP tracks and rerouted Hwy 69. Fig. 4 shows the cumulative length of these lineaments. From this histogram combined with outcrop and core observations, the main directions of jointing evident on as regional scale are sets J1 of 146°, J2 of 65° and gneissocity at 119°. The direction of the striae (scratch marks) caused by ice movement during the Pleistocene glaciation is shown on Map P3103 as 208° and some of the overburden soil deposits and surface channels scoured by glacial ice give lineaments in this direction.

All discontinuity sets are rough and planar. Joints in the steeply dipping sets are in most places iron-stained (brown) but hard, with little if any softening as a result of weathering. The shear strengths are estimated as high (peak friction angles of more than 60°), except for occasional pre-sheared joints or micaceous surfaces, which may have friction angles as low as 20° to 30°. The joints tend to be continuous over distances of tens of metres.

2.4 Groundwater and Stress

The water table is likely to be found at a depth of 2-4 metres judging from the elevations of standing water in nearby creeks and swamps. Standpipe piezometers have been installed and readings are not available at the time of writing. Drilling used water flush, so that the short-term readings are affected by pumping and recirculation.

Flow through the rock mass occurs only along joints and the hydraulic conductivity depends on joint spacings, apertures and fillings. The joints on site are typically spaced at about 30 to 40 cm intervals and are moderately open, from which the rock mass is estimated to have a hydraulic conductivity of about 10^{-5} m/s. Water seepages are therefore expected from the rock faces of the 11 m deep excavations, and ice may build up on rock faces in winter.

The vertical stresses in rocks are expected to be proportional to depth and weight of overburden, increasing at a rate of about 30 kPa per metre. Horizontal stresses as high as 14 MPa are typical of the rocks of Ontario. The high stress levels are thought to be caused by continental drift on a global scale. Usually they have little consequence in engineering works, but can influence the results of blasting, and also require precautions to allow for differential movement along rock joints when casting concrete in contact with rock.

3. BLASTING METHODS AND PRECAUTIONS

3.1 Background

Much of the blasting close to the grade separation is subject to restrictions on vibration levels, and as a result, at some locations excavation costs could be five or more times the norm, taken as $37/cu m for purposes of cost estimation.

The blast designs presented in this report have been developed solely to assist in estimating and comparing costs and evaluating the feasibility of alternative procedures. Contractors tendering for the project should prepare their own blast designs for cost estimation purposes and later in more detail after award of contract. One of the recommended Special Provisions for blasting (Section 3.4) is that the contractor retain the services of a blasting specialist to conduct blast monitoring and to provide blast designs. Because of the impact of vibration limits on the cost of the work, contractors should be advised of the need for blasting input from a quialified blasting specialist at the bid stage also.

3.2 Criteria to avoid blast damage to the track

Blast damage to the track is to be avoided by specifying an upper limit for the peak particle velocity (ppv), which is then monitored on site to ensure that it is not exceeded. Vibration levels are to be controlled using delay blasting where groups of blastholes are detonated at intervals of milliseconds. The ppv depends on the weight of explosives in each delay and not on the total explosives in the blast. Two parameters can be adjusted to limit vibrations:

- the weight of explosives W in each blast delay (proportional to the number of holes detonated in each delay). Vibration levels can be limited by detonating holes in small groups, even one at a time, but safety restrictions then dictate that holes must also be drilled and loaded one by one, which is slow and expensive;
- the length of the hole (bench height) can be reduced, but this requires a reduction in hole burden and spacing with considerably more drilling for each cubic metre of rock excavated.

The following "scaling law" or "attenuation relationship" is recommended as a basis for predicting the maximum permissible weight of explosives (W) in each blast delay that can be detonated at distance (D) from the rail track without exceeding the specified ppv:

$$V = H\,(D/W^{1/2})^{-b}$$

or

$$D = W^{0.5}(V/H)^{-1/b}$$

H and b are constants for any given set of rock conditions, which are to be determined during the early stages of blast monitoring for the project, while the blasting is still 150 to 300 m from the CP track. These attenuation coefficients are to be obtained by measuring ppv using the method specified in Section 3.4. For purposes of preliminary design and cost estimation, values of H = 88 and b = 1.7 have been assumed (using the above equation in Imperial units where V is in inches per second and W is in lb). These values were determined by CIL for Ontario Hydro's Atikokan Generating Station site where the rock conditions and excavation depth were similar to those expected at this site.

A limit of V=100 mm/s is recommended for this project. The limit is usually taken as 50 mm/s for damage to buildings, but the tolerance of rail tracks should be substantially greater than this. A 100 mm/s limit has been used by CN rail during the last several years, and they report having experienced no disturbance or damage to tracks while this regulation has been in force. The limit is probably over-conservative and CN are considering increasing it. Enquiries among blasting specialists have failed to locate an alternative less conservative criterion with any data to support it. Any upward revision of this limit would substantially reduce blasting costs and duration of the work, but would have to be supported by field measurements, and would require prior review and approval by the project consultants and CP before being applied.

3.3 Blast design assumptions

The blast design methods and vibration measuring principles used on this project are widely accepted ones given in most textbooks and blasting manuals. A useful summary is provided in the U.S. Army Corps of Engineers 1972 blasting manual referenced in Section 9.

Assumptions have included:

- 0.83 kg/m^3 for the "powder factor", the amount of explosive needed to excavate one cubic metre of rock, again based on the Atikokan blast design;
- equal burden and spacing (a square pattern of blastholes),
- a 400 mm length of cartridge,
- a blasthole diameter of 2.5 " (63 mm).

The following equation relating the weight of explosives per hole to the depth of drilling has been obtained by curve fitting as shown in Fig. 6:

$$W = 0.3271d^2 - 0.9547d + 1.4371$$

Where W is weight of explosives per hole (kg) and d is the depth of drilling including sub-drill to ensure breakage at the toe of the cut. To assist in cost estimation, the above equation is plotted as a family of curves in Fig. 6. Use of this design chart is illustrated in Fig. 5 and 7, which gives a blast design and cost estimate for the Highway 69 rock cut as it approaches closer than 50 m from the rail track (see Section 4.2).

3.4 Blasting specifications

Contractors should be advised to expect a greater than usual amount of oversized blocks that require secondary blasting or breakage before trucking off site or using as fill. Also overbreak and underbreak are likely to be much greater than usual. These conditions result from the anisotropic strength (fissility) and slabby jointing of the rock, and concentrations of mica along seams, in combination the likely poor fragmentation where shallow benches are used to limit blast vibrations.

We recommend that the specifications be based on OPSS standards for "Rock Face" with the following Special Provisions:

- Retain the services of a blasting consultant to monitor blast vibrations, measure the blast attenuation constants for the site, recommend blast designs, evaluate trial blasting and on-site blasting procedures to ensure that the peak particle velocity does not exceed 100 mm/s at the CP track. Submit copies of all blasting measurements and recommendations to the Engineer. quoted
- Determine the blast attenuation constants H and b for the site by monitoring four representative Hwy 69 blasts. For each of these blasts record the maximum weight of explosives W in any single delay and the distances D from that delay to four blast vibration monitors, equally spaced over a range of distances up to 400 m from the point of detonation. Record and measure the maximum peak particle velocity (ppv) for vertical, radial and transverse vibrations. These measurements to be completed by a blasting specialist and the results reported before blasting approaches closer than 300 m from the CP track
- Prepare blast designs for typical blasts and determine the distance L that full-scale blasting can approach the track without exceeding 100 mm/s peak particle velocity.
- Blasts closer than the limiting distance L from the track are to be designed with reduced bench heights, burdens, spacings and quantities of explosive per delay to limit vibration levels at the track to less than 100 mm/s. Each is to be monitored to ensure that the limit is not exceeded in vertical, transverse or radial directions. Submit the design to Construction Supervisor 24 hours before drilling or loading the blast.

- Production holes to be spaced at Contractor's discretion so as to limit vibrations and achieve fragmentation. Maximum height of lift limited to 7 m to ensure accurate drilling. Maximum blasthole diameter not to exceed 75 mm.
- During mucking, the exposed height of rock face is not to exceed 3.5 m before inspection and completion of any stabilization work.
- Stemming and blasting mats to be used to prevent fly rock, and blasting mats are to be placed across the track to protect the rails where there is a risk of the blast dislodging rock from faces adjacent to the track. The contractor is to remove blasting mats and any rock fragments from the track within 30 minutes of each blasting operation.

3.5 Mechanical excavation alternatives

The reduced scale blasting procedure is slow and expensive, but is likely to be several times less expensive than most mechanical excavation alternatives:

- Hoe ram excavation in rock with this strength would require rental of a 10,000 ft-lb machine at a rate of $500/hr. It could easily take two hours to remove a cubic metre of the more massive pink gneiss.
- Where there is room to manouver, a large excavator with a "tiger-toothed " bucket should be quite efficient at lifting out slabs. This method might be useful in wide excavations such as the Hwy 69 cut but not in narrow headings or corners. It could provide a very efficient and rapid method for removing the upper approximately 2 m of rock where the jointing is open. Evidently this method can not produce a smooth face. If the contract is to benefit from the possible economies, it should be specified separately as an option with the provision that loose irregular faces are acceptable in place of pre-split smooth ones provided they are thoroughly scaled and cut back at angles no greater than 1:1 for stability. No payment would be made for such scaling, or for rock removed beyond the theoretical limits of the smooth-blasted alternative.
- Splitting can be achieved using either mechanical wedges (plug and feathers) or hydraulic wedges. Both require line drilling at about 30 cm to 1 m spacing, and are expensive.
- Another mechanical excavation method that achieved a 1 m/day rate of advance in a tunnel in Japan, in massive granite, is a combination of line drilling and hoe ram breakage. All of these methods work best when assisted by light blasting to loosen the rock.

Mechanical excavation involves greater risk than blasting of being incapable of adequate progress. Trials of any given type of machine would be needed to determine its performance.

All methods work best when assisted by light blasting to loosen the rock, and a combination of blasting and mechanical excavation should be anticipated in the drafting of specifications.

The preliminary cost estimates have been based on blasting only.

4. HIGHWAY 69 ROCK CUT EXCAVATION

4.1 Hwy 69 rock cut at distances greater than 50 m from the CP track

The design and excavation sequence for the Hwy 69 rock cut as it approaches from the north towards the CP track is likely to be the same whatever the method used for the grade separation. A proposed design and blasting pattern is shown in Fig. 5. The cut has a depth of 14 m, and a width of 78 m at the crest and 64 m at highway level. Features of the design are as follows:

- Excavation in two 7 m benches. Blasthole drilling becomes inaccurate at depths of more than about 10 m;

- A 3 m wide rock catch bench at mid-height. This width allows periodic cleaning by bulldozer, needed for the bench to remain functional.

- Side slopes at 0.25H:1V . This provides some additional stability but mainly is proposed because high vertical faces appear overhanging and threatening to the motorist. Rock faces with this batter have been adopted for example by the Oregon State Highways Department, who have determined the ditch and shoulder widths necessary to avoid overspill of rockfalls onto the pavement.

- Pre-split perimeter holes spaced at 0.75 m centres and an outer row of buffer holes with reduced burden to minimize wall damage.

Provision of a bench and battered side slopes increases the volume of excavation and hence construction cost by about 9% but provides an excavation profile that is less likely to require bolts or shotcrete, and unlikely to require maintenance.

The following is a suggested construction sequence at distances greater than 50 m from the CP track:

- strip overburden,
- pre-split the perimeter of the upper bench,
- blast, excavate, scale and inspect the upper bench,
- pre-split the perimeter of the lower bench,
- blast, excavate, scale and inspect the lower bench.

Reinforcement of the presplit faces is unlikely to be needed except at isolated locations along the crest. The rock faces should be inspected and rockbolts installed or shotcrete applied locally where necessary during mucking and while the face is still supported by broken rock and within reach. Not more than 3.5 m height of face should be exposed during mucking before completing this inspection and support.

4.2 Highway ~~6~~09 rock cut closer than 50 m from the track

Fig. 7 gives a blasting sequence and cost estimate for the Hwy 69 rock cut developed using the Fig. 6 chart. The 8 m drill depth curve in Fig. 6 shows that with 10 holes per delay, excavation can approach within 35 m of the track without exceeding a ppv of 100 mm/s.

Bench heights from there on must be progressively reduced, first to three 5 m benches, then 4x4, 5x3, and 7 2 m benches, at the same time reducing the number of holes per delay from 10 to 5 and then 2. The final 3 m closest to the track may be slashed with lightweight presplit charges or by closely spaced line drilling with webs between holes split by detonating cord.

4.3 Hwy 69 cost estimate

Fig. 7 shows a total blasting cost of about $2,796,000 to excavate the 50 m of Hwy 69 closest to the track. The average unit price is estimated as about $60/cu m compared with a base price of $37/cu m, which has been assumed for blasting with no ppv restriction. This represents an increase of 62% caused by the ppv restriction.

The cost estimates are based on a model that relates cost to the depth of drilling and holes per round, assuming a 500% increase for the worst case of a 2 m bench height and one hole per delay.

In this and other cost estimates the unit price for blasting at any given location has been estimated considering both bench height and holes per delay according to the empirical model discussed earlier. The base price for unrestricted bench blasting (at least 10 holes per delay and a permissible drilling depth of at least 9 metres) has been assumed as $37/cu m based on Ministry of Transportation statistics for highway construction in northern Ontario. The maximum price has been arbitrarily limited to $185/cu m (five times the base value). The prices intermediate between these two limits are obtained by linear interpolation.

The arbitrary upper limit is reduced considerably from the possible maximum calculated taking into account the time needed for drilling, secondary breakage of slabby oversized rock, increased loading and haulage costs etc. The x 5 limit was considered more realistic in today's competitive market.

5. OUTLINE OF GRADE SEPARATION ALTERNATIVES

Two approaches have been considered for constructing the grade separation, a "diversion alternative" in which a section of track is re-routed to pass over a bridge constructed north of the present track location; and an "undercut alternative" in which the highway is taken beneath the railway at its present location. The diversion alternative was approved in principle by CP at the April 8, 1998 meeting, subject to review by CP geotechnical specialists. Options were left open to explore alternatives that might reduce the cost of the work and the level and duration of risk and inconvenience.

6. GRADE SEPARATION - DIVERSION ALTERNATIVE

6.1 Procedure - Diversion Alternative

(Figures 8-10):

1. Excavate Hwy 69 from the north to within 15 m of the CP track. ✓ quoted
2. To limit vibrations, all excavation within 50 m of the track requires blast monitoring with multiple benches of reduced height, reduced blasthole spacing and reduced explosives per delay;
3. Scale the rock face along the north side of the track and install rockbolts to prevent track heave and rockfalls from adjacent blasting. Install heave gauges at 50 m intervals over about 550 m
4. Construct a bridge across the Hwy 69 cut 20 m from the rail track.

CPR STRUCTURE - COST ESTIMATE

HIGHWAY 69 FOUR LANING, FROM TOWER ROAD (MacTier), NORTHERLY 26.5km
to 2km NORTH OF HIGHWAY 141
G.W.P. 290- 97 - 00

WORK TYPE	Unit	Option 1 - 'Long Diversion'		Option 2 - 'Short Diversion'-VE Option	
		Quantity	Total	Quantity	Total
Rock Excavation - Highway 69	m^3	290,545	$5,229,810	326,452	$5,876,136 ✓
Rock Excavation - Track	m^3	67,461	$1,214,298	10,137	$182,466 ✓
Rail, Ties, Ballast	m	2,100	$1,377,600	480	$314,880
Structure	each	1	$1,400,000	1	$1,300,000
TOTAL			$9,221,708		$7,673,482
UTILITY RELOCATION AND BLASTING PROTECTION					
ITT Fibre Optic	l.s	1	$400,000	1	$30,000
Blasting Premium and CPR flagman	l.s	-	$50,000	1	$400,000
TOTAL			$9,671,708		$8,103,482

Saving $1.5m

5. Excavate a 1 km long rail diversion cut 6 to 11 m deep over about 500 m and 0 to 6 m deep over about 500 m. Excavation requires a presplit perimeter and reduced-scale bench blasting to limit vibrations to less than 100 mm/s ppv.

6. Wherever needed, place blasting mats across the track to protect the rails against fly rock and rockfalls. Monitor all blasts, and after each blast remove the blasting mat and any fallen rock, and inspect and gauge the track.

7. During bench excavation, inspect rock faces along the new cut, scale and spot-bolt or shotcrete where needed.

8. Place ballast, lay track, and divert rail traffic across the new bridge.

9. Complete the remaining section of Hwy 69 south of the new CP track. All blasting within 300 m of the track requires blast monitoring, and reduced-scale blasting is to continue for as long as necessary to limit ppv to less than 100 mm/s

6.2 Excavation time and costs

Because of the need to limit ppv to less than 100 mm/s, it will be necessary to excavate in benches of limited height and with reduced numbers of holes per delay. Because of these restrictions, we estimate that the contractor will find it necessary to work several headings and more than one bench in each heading simultaneously in order to completed the rail diversion cut in 4 to 6 months.

A cost of about $3,527,000 is estimated for blasting the rail diversion cut. Assumptions and methods of estimation have been outlined in Section 4.3 earlier.

This is about 90% more than if no vibration limits were to apply, which may be compared with an increase of only 62% above the norm for blasting Hwy 69 with a 100 mm/s ppv restriction averaged over the 50 m closest to the rail track. The difference reflects the much more difficult blasting needed in the rail cut because it has about ten times the length in close proximity to the track.

The estimated costs for blasting (Fig. 9) have been obtained using the same methods and assumptions as used for Hwy 69, described in Section 4.3 above. The main differences between this and the Highway 69 blast design are as follows:

- Because the blasting runs sub-parallel to the rail track rather than perpendicular, the distance between the existing track and the diversion cut (the "proximity" shown in the cost estimate table, Fig. 9) reaches a maximum at about the mid-point of the diversion, and reduces progressively to zero at each end where the new and existing tracks meet. This proximity value governs the maximum permitted explosives per delay and hence the cost of blasting.

- Costs have been calculated assuming a maximum "proximity" of 20 m. Smaller values such as the 8 m of an earlier proposed diversion route lead to estimates of cost and duration that are beyond the range worth considering.

Topography along the route requires rock excavation over about half of the 1 km total length. The depth of rail cut ranges from zero to 11 m. Variations of both depth and proximity have been taken into account in assessing blasting methods and estimating costs.

6.3 Stabilization costs

The cost estimate for rock stabilization (diversion alternative) is about $111,000 (Fig. 10). The types and locations of stabilization measures are discussed below with reference to Fig. 8. A combination of scaling and bolting with meshing or shotcreting will be required. The quantities of bolts etc. need to be confirmed after completion of scaling, when the need for further stabilization can be assessed. This requires that support items be tendered on a unit price basis. Details of methods and materials are given in the Appendix 4 specifications.

Pillar P (upper part of Fig. 8) requires the most treatment. The rock is slabby and loose and the joints dip towards the track. This pillar is likely to be considerably shaken by blast vibrations and gases from blasting only a few metres away. Trying to preserve its integrity would be expensive and it might even be cheaper to remove it.

As a first step prior to any support installation, the face will be scaled by a combination of hoe ram and manual scaling (2 person crew). We estimate 44 hours of each at a total scaling cost of about $22,000. Only the lower part of the pillar should be bolted, allowing the upper narrower part to loosen, and trimming off loose blocks by scaling. The scaling should stop when it becomes difficult, bolting the remaining rock in place only to the extent required for long-term stability of the diversion cut. The rock removed will be in large slabs requiring secondary breakage prior to loading and transportation off site. A hoe ram will be needed full-time for this and other scaling and secondary breakage work. The track may need to be protected with blasting mats at some locations. Blasting mats will be needed in the diversion cut to control fly rock, and the same crane can handle mats on both sides of Pillar A.

The same south-dipping gneissocity joints affect the north face of the diversion cut, but here presplitting will make the requirement for support minimal. Reinforcement of the presplit faces is unlikely to be needed except at isolated locations along the crest. The rock faces should be inspected and rockbolts installed or shotcrete applied locally where necessary during mucking and while the face is still supported by broken rock and within in reach. No more than 3.5 m height of face should be exposed before completing this inspection and support.

All bolts installed are to be of fully resin bonded type, 2 m long and 25 mm dia, complete with 10 by 10 cm face plates and beveled washers. A specification is provided in Appendix 4.

For cost estimation purposes we have allowed for rockbolting at the toe of the slope to limit heave and lifting of the track, and at the crest to be installed after removing any superficial loose material.

The total rockbolting requirement is estimated at 1277 bolts at $80 each = $102,000. This seems excessive and quite likely would be reduced by at least 50% if scaling is effective.

7. UNDERCUT ALTERNATIVE

7.1 Procedure

(Figures 11 and 12)

1. Scale along track 100 m either side of the crossing location to remove loose rock and prevent rockfalls during the work. If necessary install rockbolts.

quoted

2. Excavate Hwy 69 from the north and south until the cut reaches the CP track. In the last 50 m (or whatever distance is confirmed as safe from site attenuation measurements). Use reduced scale blasting to limit vibration levels to less than 100 mm/s ppv.

3. Presplit rock faces parallel to the track in a single vertical lift. Progressively expose these faces by reduced scale charges. Stockpile broken rock for use in ramping to maintain access to these faces as required to maintain access for horizontal drilling for bolts, and blastholes as described below.

4. Shotcrete the faces from the top down as they become exposed.

5. Install 3 m long fully resin-grouted rockbolts horizontally at 1.5 m centres to reinforce the central pillar and abutments.

6. Drill closely spaced (0.3 m centres) presplit holes horizontally along the faces of the central pillar and abutments using a multiple-boom jumbo drill.

7. Drill blastholes from one or both sides of the 7 m high 10 m thick rock web beneath the track, in a horizontal V-cut pattern as shown in Fig. 11. (This figure shows drilling from one side only, but drilling from both sides will give better release with reduced vibrations on the pillar and abutments).

8. Close the track for an estimated 13 hr to 1 day maximum to blast and construct a bridge deck:
9. Lift track. (1 hr)
10. Load and detonate horizontal blastholes. (3 hr)
11. Muck out upper 2 m leaving broken rock in place below that.(2 hr)
12. Trim footings of bridge beams and install vertical rockbolts in pillar and abutments. (2 hr)
13. Lay bridge beams on bedding of epoxy concrete. (2 hr)
14. Replace ballast and track. (2 hr)
15. Inspect, gauge and reopen track (1 hr) (estimated total 13 hrs)

16. Pattern bolt and shotcrete east and west rock faces of the undercut, mucking out progressively until the complete height is exposed. Trim-blast the rock floor where necessary using blasting mats to protect bridge deck. (Hoe ram excavation could be used as an alternative). Place granular base and asphalt. Open Hwy 69.

7.2 Excavation costs

The total cost for excavating the undercut alternative (Fig. 11) is estimated as about $845,000 which is 24% of the estimated blasting cost for the rail diversion cut. This is the result of a much reduced volume of rock to be excavated. The time required should be reduced by an even greater margin. We estimate that the undercut grade separation can be completed in one to two weeks compared with several months for the diversion.

The costs for undercut excavation have been estimated assuming twice the unit price for normal bench blasting, i.e. $74/cu.m. We have included excavation of 528 cu.m of rock to finish the north and south vertical faces within the Hwy 69 cut, with an additional 6006 cu. m in the two panels beneath the bridge deck.

7.3 Stabilization Costs - Undercut Alternative

These costs are tabulated in Fig. 12. The total of about $296,000 includes scaling and trimming of the north and south vertical faces and marking and scaling in the undercut; line drilling of presplit holes to protect the faces of the pillar and abutments; shotcrete of the complete north and south faces plus all exposed rock in the pillar and abutments; fully resin grouted bolting to reinforce pillars and abutments; and full time rock engineering inspection during the work.

8. COMPARISON AND CONCLUSIONS

The total rock engineering costs for the final 50 m of Hwy 69 north of the crossing plus excavation and stabilization costs for each of the two alternatives for grade separation are summarized as follows:

Rail diversion alternative:

Hwy 69 last 50 m excavation:	$2,424,710
Diversion excavation:	$3,527,826
Stabilization:	$110,629
Total:	$6,063,165

Undercut alternative:

Hwy 69 last 50 m excavation:	$2,795,915
Undercut excavation:	$845,164
Stabilization:	$295,850
Total:	$3,936,929

The two estimates for Hwy 69 excavation are different because excavation stops 15 m from the track for the diversion alternative, but only 3 m from the track for the undercut (see Fig. 7).

It is emphasized that these estimates are preliminary and more reliable estimates can be obtained by checking both unit prices and quantities. They are intended for preliminary assessment of the feasibility of alternatives and before use in design need reworking using improved data.. Also they include only rock engineering costs and do not include such items as costs for reconstructing the bridge or those to be incurred by CPR or by other consultants for design and supervision.

Time estimates of necessity are even more approximate. We estimate the undercut option will take 2 to 3 weeks in comparison to several months for the rail diversion alternative. Also, the reliability of estimation is greater for the undercut alternative which is of shorter duration. Cost overruns are likely to be proportional to duration.

Duration of the work is important in that it affects not only costs but also risks to the rail track and inconvenience to CP, who have indicated that they will carry out all necessary track gauging.

The two alternatives have been explored to the extent of determining that the methods are considered feasible and safe. The methods for the rail diversion involve standard excavation and support practices with a generous safety margin allowed. Similarly the method for the undercut requires only proven technology such as drilling and blasting methods long established in tunnelling. Sufficient allowance has been made for rock stabilization and support.

9. REFERENCES

Map P3103 Quaternary Geology, Lake Joseph - northern Ontario
Map 2118 Parry Sound Huntsville Area, Ontario Division of Mines Scale 1:126720 (1 in = 2 mi).
U.S. Army Corps of Engineers, 1972. Engineer Manual "Systematic Drilling and Blasting for Surface Excavations. EM 1110-2-3800. Office of the Chief of Engineers, Washington D.C.,

Respectfully submitted,
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Massoud Palassi, Ph.D., P.Eng. John A. Franklin, Ph.D., P.Eng., CPGS

FIGURES

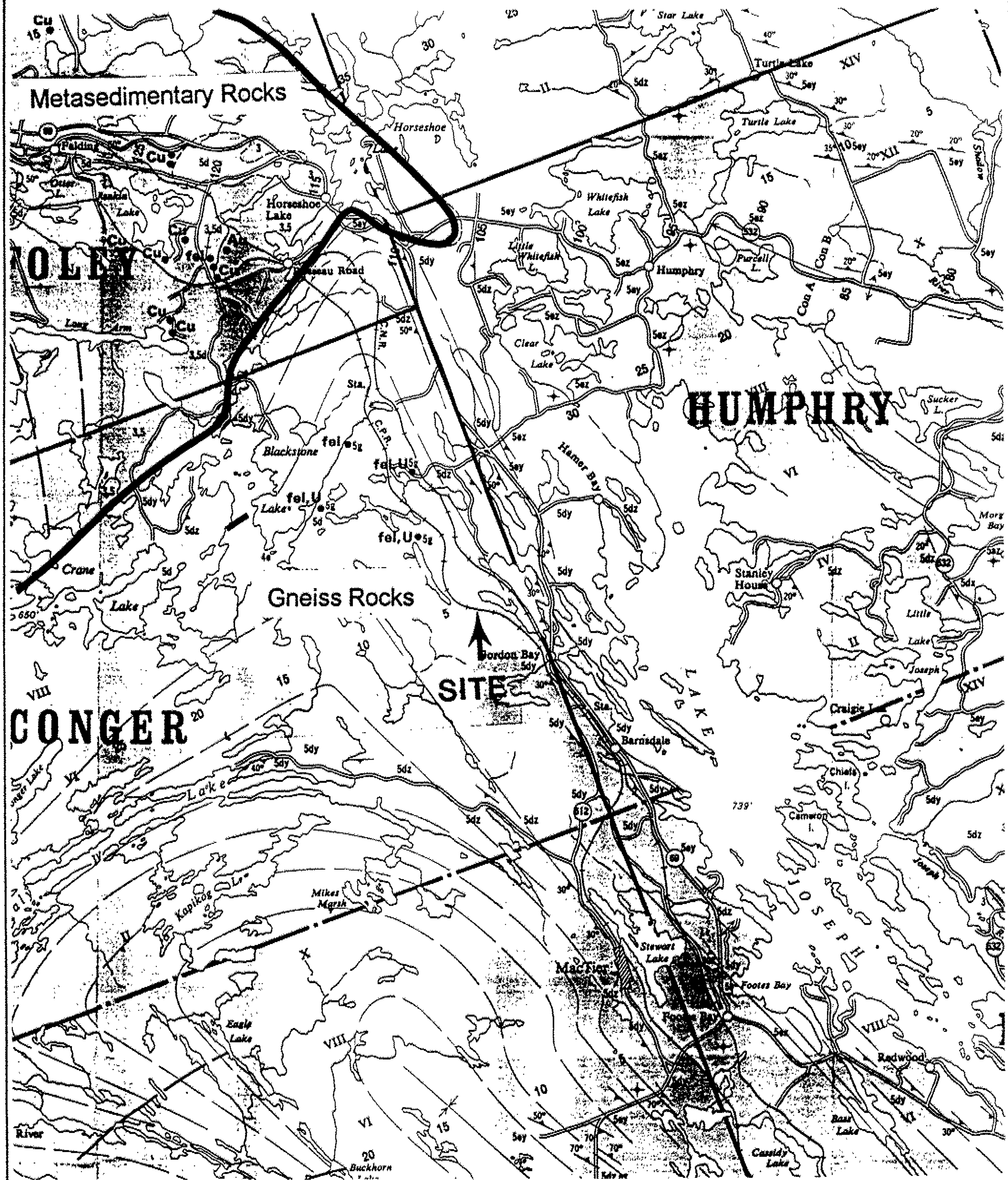

Fig. 1 Site location and bedrock map

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FIGURE: 1

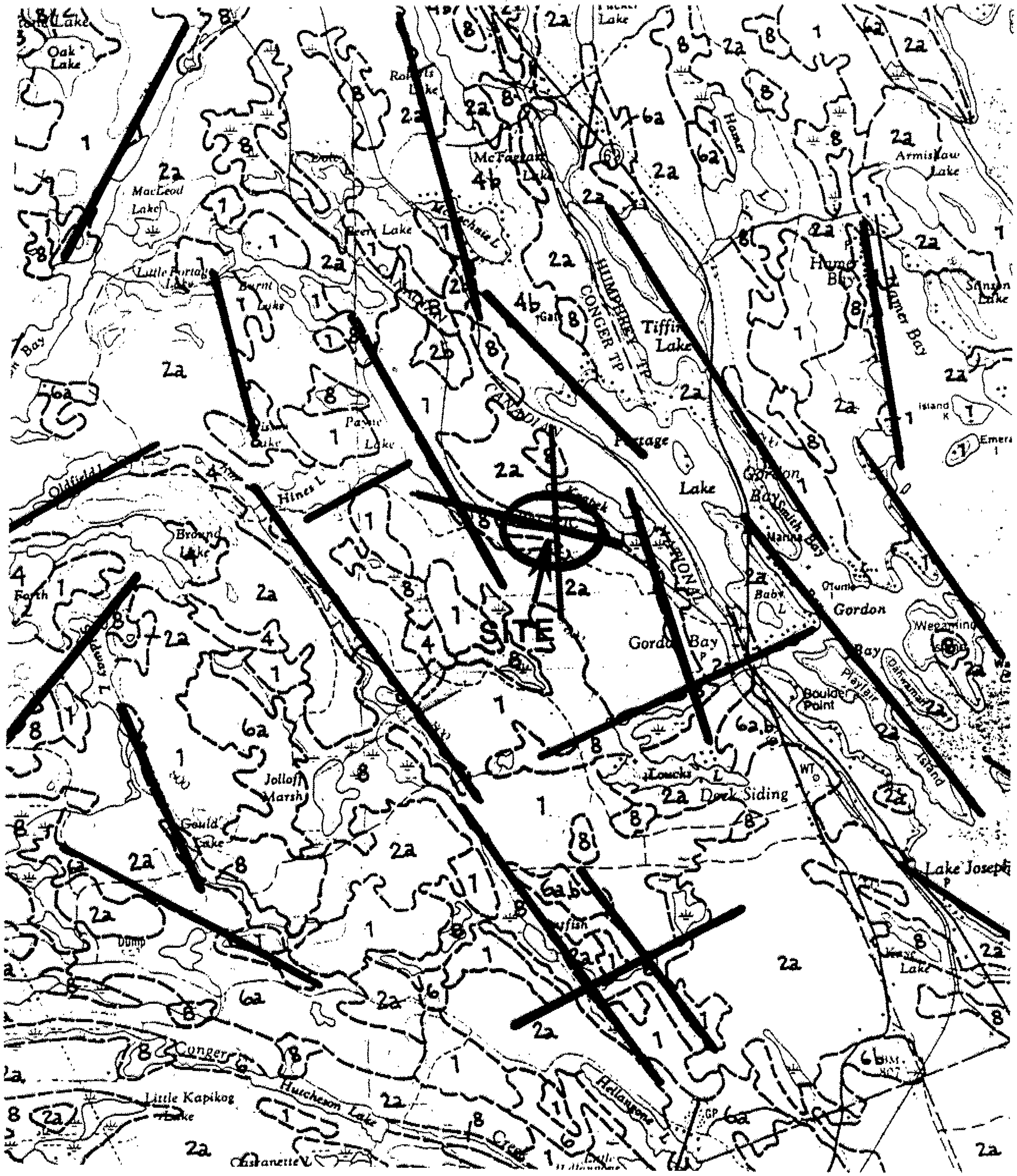

Fig. 2 Pattern of bedrock lineaments

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FIGURE: 2

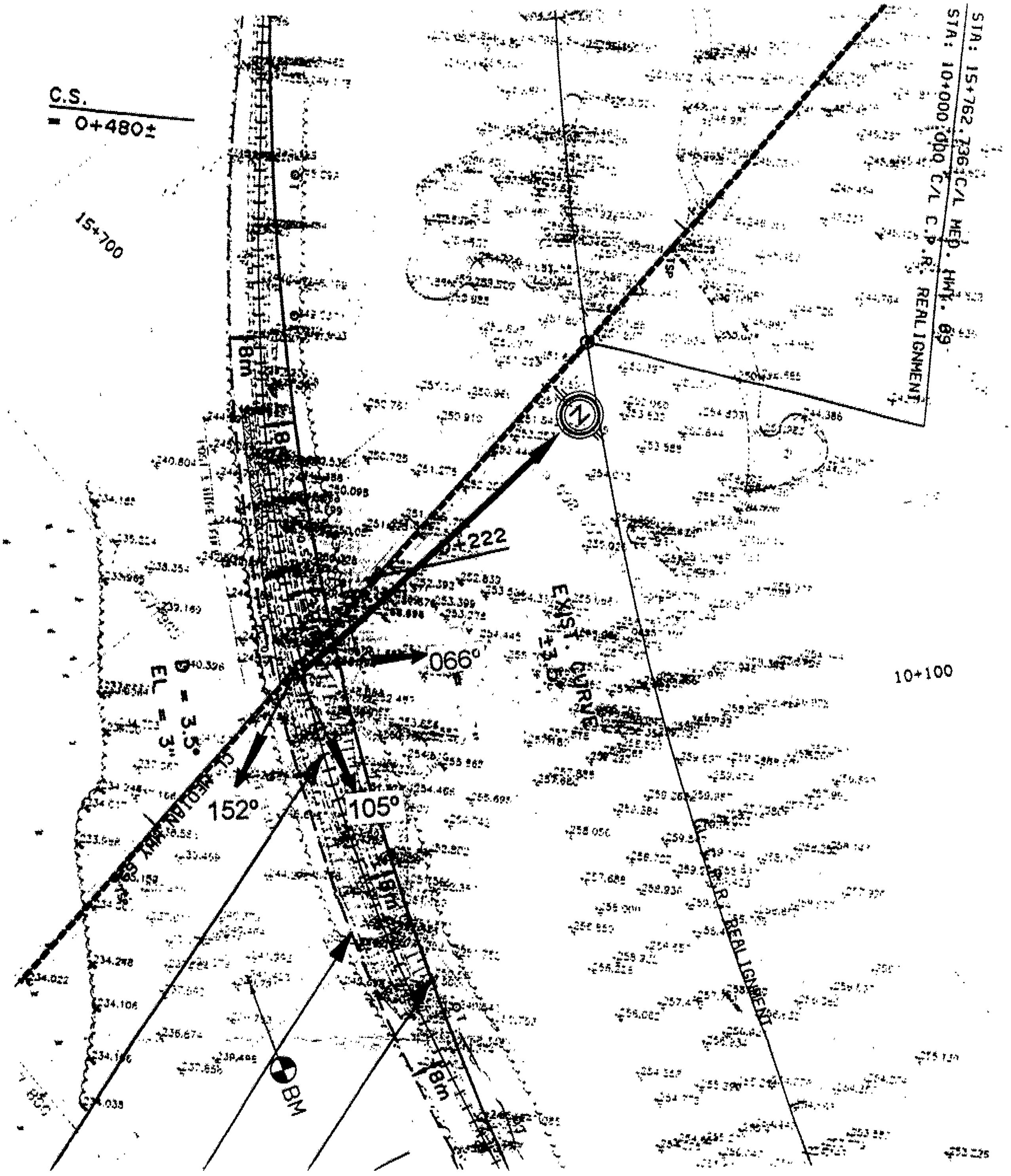

Fig. 3 Inferred major jointing directions

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	gneissocity	J1	J2	glacial
STRIKE				
estd from photos	115	150	45	
measured from lineaments	119	146	65	28
DIP				
estd from photos	20-50	80	85	
measured from lineaments				
DD				
estd from photos	205	240	135	
measured from lineaments				

CORE JOINT DIP ANGLES

HOLE 1	HOLE 2	HOLE 3
66.3	65.9	63.3
76.4	76.4	35.9
65.9	72.9	29.5
64.7	66.1	19.9
29.5	64.7	
19.9	25.3	

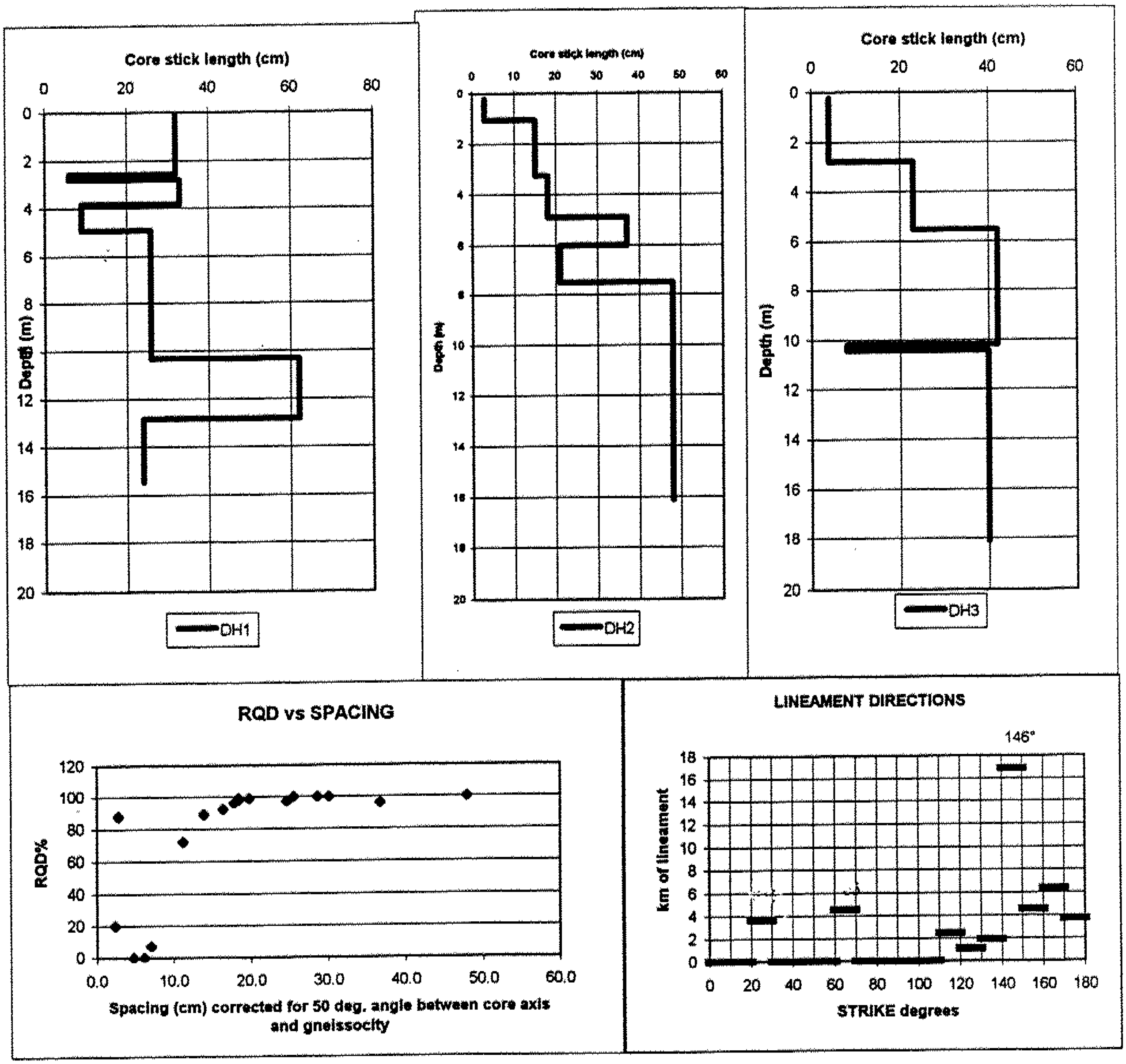

Fig. 4 Joint and lineament data

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FIGURE: 4

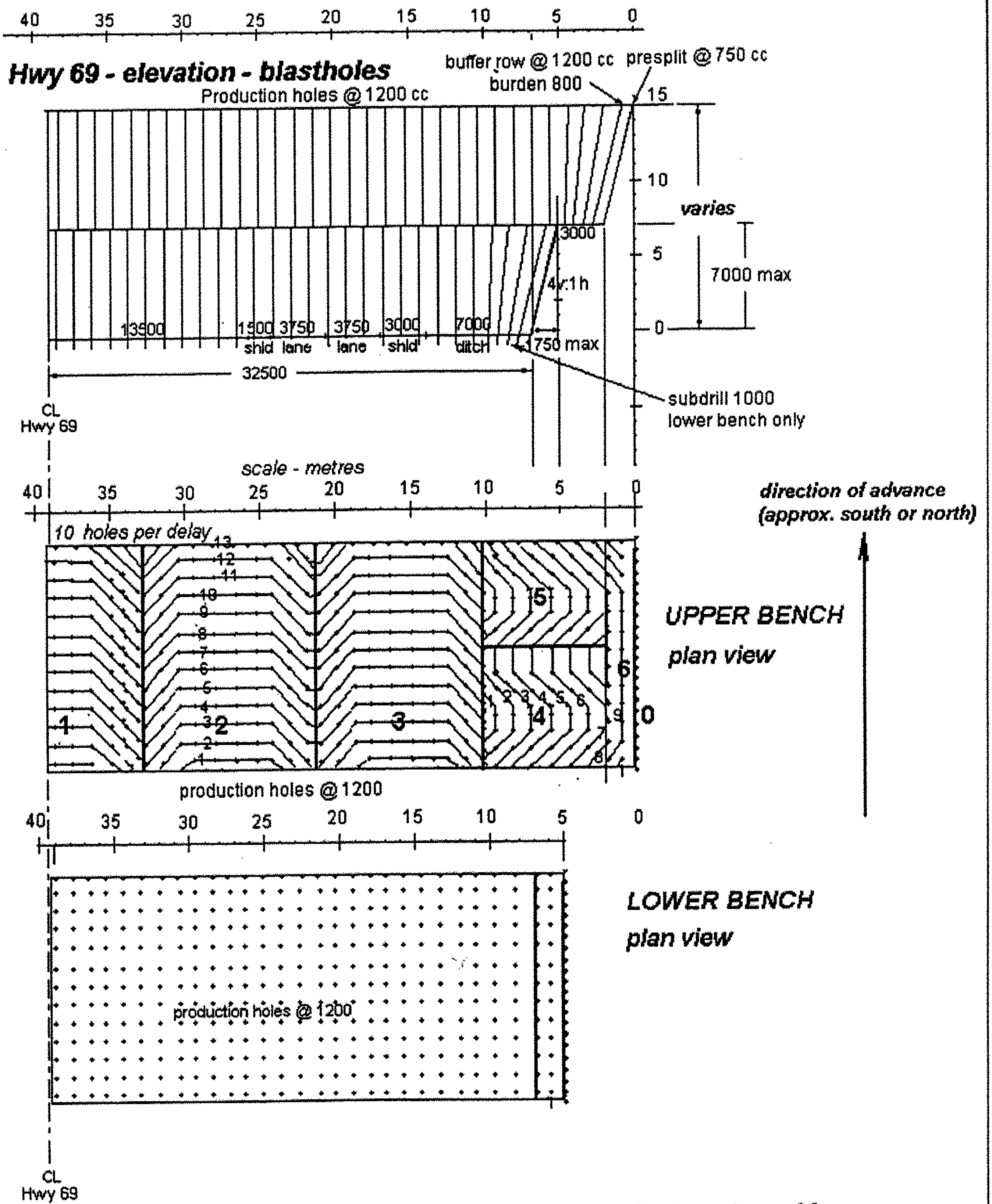

Fig. 5 Preliminary blast design, hwy 69

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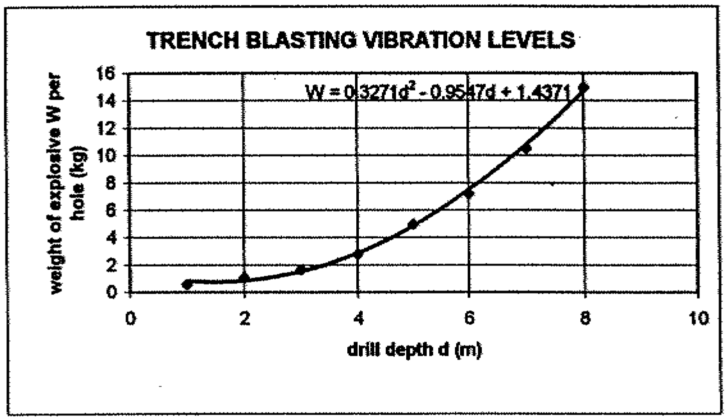
PROJECT: G819.2 FIGURE: 5

drill depth m	wt expl/hole kg
1	0.555
2	1.11
3	1.665
4	2.775
5	4.995
6	7.215
7	10.545
8	14.985

holes/dly	depth1 m	2 m	3 m	4 m	5 m	6 m	7 m	8 m	9 m
1	2.53	2.573286	3.46612	4.688424	6.290181	7.699537	9.241388	10.80304	12.377
2	3.580965	3.639177	4.90184	6.630433	8.895659	10.88879	13.0693	15.27781	17.50372
3	4.385768	4.457063	6.0035	8.120589	10.89491	13.33599	16.00655	18.71141	21.43759
4	5.064249	5.146573	6.93224	9.376848	12.58036	15.39907	18.48278	21.60608	24.754
5	5.662002	5.754043	7.75048	10.48363	14.06527	17.21669	20.66437	24.15633	27.67582
6	6.202413	6.303239	8.49023	11.48425	15.40773	18.85994	22.63668	26.46194	30.31734
7	6.699371	6.808276	9.1705	12.4044	16.64225	20.37106	24.45041	28.58216	32.74647
8	7.161929	7.278353	9.80367	13.26087	17.79132	21.77758	26.13859	30.55561	35.00745
9	7.596373	7.719859	10.3984	14.06527	18.87054	23.09861	27.72416	32.40912	37.131
10	8.00728	8.137446	10.9608	14.8261	19.8913	24.34807	29.22383	34.16221	39.13951



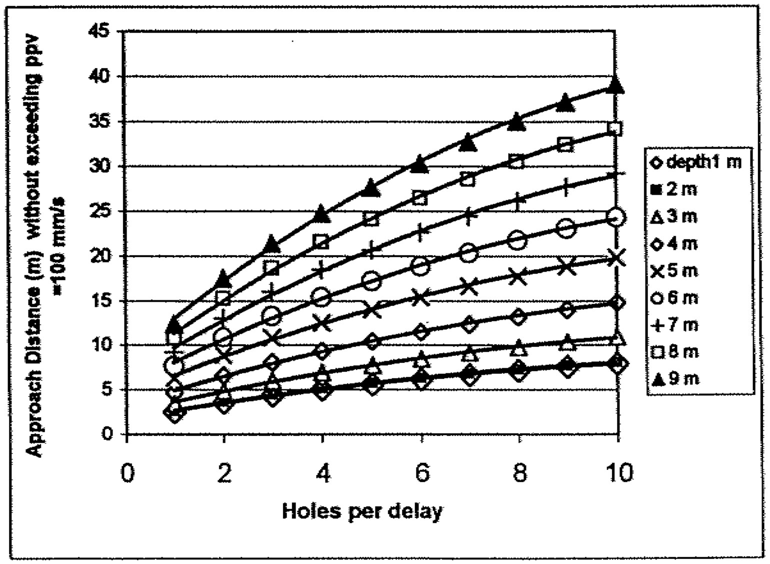


Fig. 6 Curves for blast design (ppv = 100 mm/s)

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FIGURE: 6

CHAINAGE	from	40	30	24	15	9	5	3	0	
	to	+	40	30	24	15	9	5	3	
Proximity to CP track		per 10 m	30	24	15	9	5	3		
		10	10	10	5	5	2	2	presplit & Buffer	
Bench Pattern		2x7 m	2x7 m	3x5 m	4x4m	5x3m	5x3m	7x2m	light charges	
Blast cost		**Bench 1**	**Bench 1**	**Bench 1**	**Bench 1**	**Bench 1**	**Bench 1**	**Bench 1**	cost totals	Vol. totals
	Drill dpth	7	7	5	4	3	3	2		
	Volume	5,040	5,040	2,160	2,592	1,296	864	288		17,280
	Unit price	$40	$40	$44	$89	$95	$120	$132		
	F	0.65	0.65	0.92	1.15	1.53	1.53	2.29		
	Blast cost	$199,294	$199,294	$94,713	$229,981	$123,129	$103,398	$38,083	$788,597	
		Bench 2	**Bench 2**	**Bench 2**	**Bench 2**	**Bench 2**	**Bench 2**	**Bench 2**		
	Drill dpth	7.5	7.5	5	4	3	3	2		
	Volume	4,480	4,480	1,920	2,592	1,296	864	288		15,920
	Unit price	$39	$39	$44	$89	$95	$120	$132		
	F	0.61	0.61	0.92	1.15	1.53	1.53	2.29		
	Blast cost	$173,936	$173,936	$84,189	$229,981	$123,129	$103,398	$38,083	$752,715	
				Bench 3	**Bench 3**	**Bench 3**	**Bench 3**	**Bench 3**		
			Drill dpth	6	4	3	3	2		
			Volume	1,920	2,304	1,152	864	288		6,528
			Unit price	$41	$89	$95	$120	$132		
			F	0.76	1.15	1.53	1.53	2.29		
			Blast cost	$79,366	$204,428	$109,448	$103,398	$38,083	$534,722	
					Bench 4	**Bench 4**	**Bench 4**	**Bench 4**		
				Drill dpth	5	3	3	2		
		←		Volume	2,304	1,152	768	256		4,480
		$37,323	**per metre**	Unit price	$85	$95	$120	$132		
			14 m cut	F	0.92	1.53	1.53	2.29		
				Blast cost	$195,747	$109,448	$91,909	$33,851	$430,955	
						Bench 5	**Bench 5**	**Bench 5**		
					Drill dpth	4	4	2		
					Volume	1,152	768	256		2,176
					Unit price	$89	$113	$132		
					F	1.15	1.15	2.29		
					Blast cost	$102,214	$87,087	$33,851	$223,152	
								Bench 6		
							Drill dpth	2		
							Volume	256		256
							Unit price	$132		
							F	2.29		
							Blast cost	$33,851	$33,851	
								Bench 7		
							Drill dpth	2.5		
							Volume	256		256
							Unit price	$125		
							F	1.83		
							Blast cost	$31,922	$31,922	

Cost model for calculating unit prices

$U = (4N/9)F - (2N/9)H + 3N$

$U_f = (8N/9)F + 5N/9$ where N = normal unit price = $37/cu m

$U_h = (-4N/9)H + 49N/9$

$F = 4.5823/d$

F = drilling factor, m/cu.m

d = drill depth

TOTAL BLASTING COST $2,795,915

(Final 60 m approaching CP Rail track)

Avg. cost per linear m of Hwy 69 over last 60 m $55,918.31

Same cost if no ppv restriction were to apply $37,323

Total Volume	**46896**
Avg. Unit Price	**$59.62**
Base Unit Price	**$37.00**
Ratio	**$1.61**

Fig. 7 Hwy 69 rock cut cost estimate

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FIGURE: 7

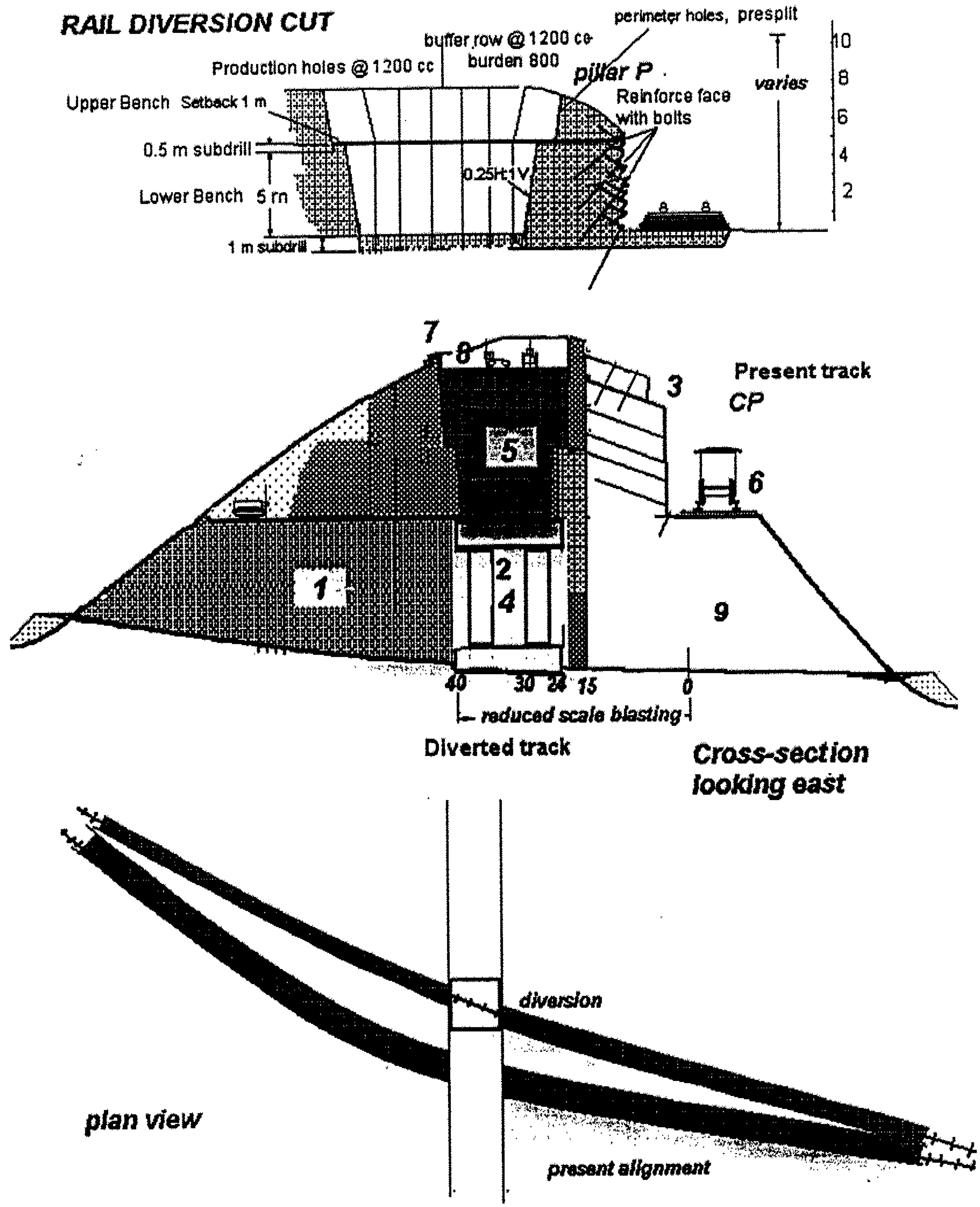

Fig. 8 Diversion alternative: construction sequence

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CHAINAGE from		0	200	550	600	700	800	900	1000	
to		200	350	600	700	800	900	1000	1105	
Proximity to CP track		1.8	5	10.4	11.8	13.6	15.4	17.2	19.4	
D for ppv>100 mm/s		2.5	5	10	11	11	15	17	19	
Total volume	**Bench 4**									TOTALS
	Drill dpth					3 m				
	Holes/dly					10				
2073	Volume					2073				
	Unit price					$54				
	Cost					$111,942				$111,942
	Bench 3									
	Drill dpth				3 m	3 m	4 m	5 m	6 m	
	Holes/dly				10	10	10	7	6	
15742	Volume				1916	1916	2392	4010	5508	
	Unit price				$54	$54	$48	$69	$74	
	Cost				$103,464	$103,464	$114,816	$276,690	$407,592	$1,006,026
	Bench 2									
	Drill dpth			3 m	3 m	3 m	4 m	5 m	6 m	
	Holes/dly			9	10	10	10	7	6	
14782	Volume			879	1759	1759	2167	3504	4714	
	Unit price			$62	$54	$54	$48	$69	$74	
	Cost			$54,498	$94,986	$94,986	$104,016	$241,776	$348,836	$939,098
	Bench 1									
	Drill dpth	2 m	2 m	3 m	3 m	3 m	4 m	5 m	6 m	
	Holes/dly	1	4	9	10	10	10	7	6	
18081	Volume	2540	2540	846	1693	1603	1942	2998	3919	
50678	Unit price	$140	$116	$62	$54	$54	$48	$69	$74	
	Cost	$355,600	$294,640	$52,452	$91,422	$86,562	$93,216	$206,862	$290,006	$1,470,760

TOTAL ESTIMATED BLASTING COST **$3,527,826**
ppv limit 100 mm/s, Rail diversion cut only

BLASTING COST ESTIMATE, RAIL DIVERSION ALTERNATIVE

Normal cost with no ppv restrictions **$1,875,086**
Based on $37/cu m

Fig. 9 **Diversion alternative: blasting costs**

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FIGURE: 9

1 Rock Stabilisation, Rail Diversion Alternative

BOLTING

	No.	$/bolt	Totals $
	1277	$80	$102,160

SCALING

Stabilisation along existing track adjacent to cut to be blasted

manual	hr/100 m	lin m	crew hr	$/hr	
	8	555	44	$180	$7,992
machine	hr/100 m				
8	555 mchine hr	$/hr			
			44	$250	$11,100

Stabilisation of rail diversion cut, inc. removal of loosened upper part of pillar P in preparation for bolting

manual	hr/100 m	lin m	crew hr	$/hr	
	8	200	16	180	$2,880

PROTECTION

Protection of track - placement & removal of blasting mats and cleanup of fallen rock

	3	650	20	250	$4,875

INSPECTION

Visual inspection after each blast plus readings on heave gauges alongside track

Installation of heave gauges (single rod extensometers) @ 50 m intervals over 555 lin m,

10 gauges @ $200 plus readout instrument, 1@ $250 — $2,250

	no. days	hr/day	hrs	$/hr	
inspector (inc. 15 hr installation)	830	1	845	60	$50,706
Rock engineer, periodic visits	50	110	$5,500		
disbursements, allow			2,000		
				Total, Stabilisation	**$110,629**

TRACK INSPECTIONS BY CP - visual inspection plus gauging and level measurements of rails

Surveying if done independently by consultant/contractor:

surveyrs	830	0.5	415	100	$41,500

Fig. 10 Diversion alternative: stabilization costs

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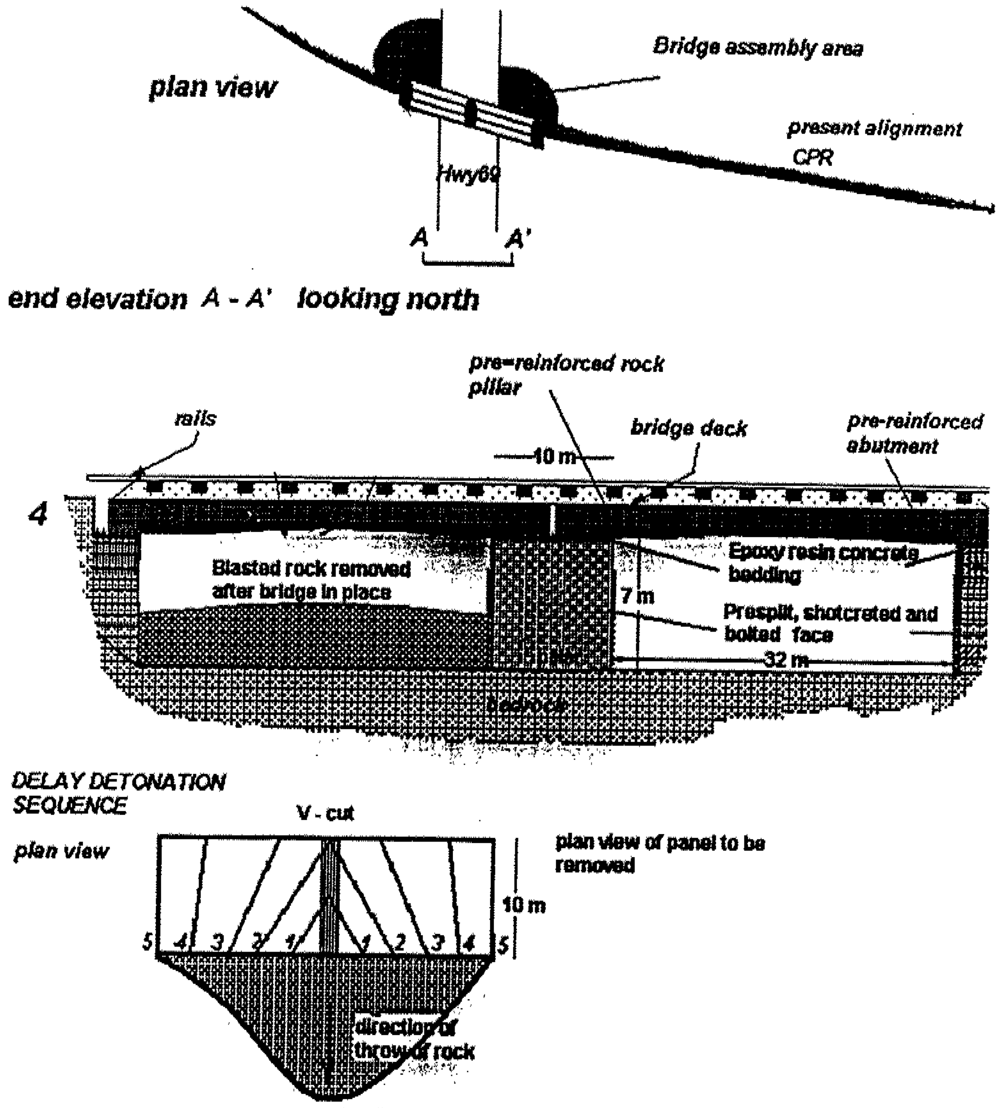
PROJECT: G819.2 FIGURE: 10



Completion of Hwy 69 to a vertical face, north and south faces (See Fig. 7)
2 x 8 m faces say 4 m at base, 2 m at top, 110 m long
Allow unit price 2x normal = $74/cu m for slashing to a presplit line

5280 cu m @	$74	$390,720

Undercut blasting, 2 panels each 3x14.3 m = 42.9 m long x 7 m high x 10 m wide =

6006 cu m @	$74/cu m	$444,444
Trim blasting of invert allow		$10,000
	Total estimate	$845,164

Fig. 11 Undercut alternative: construction sequence and blasting costs

	NAME	DATE	franklin geotechnical engineering
Drawn By	Massoud Palassi	3 Sept. 1998	
Checked By	John A. Franklin	3 Sept. 1996	PROJECT: G819.2 FIGURE: 11
Revisions			

2. Rock Stabilisation, Undercut Alternative

	crew hrs	machine hrs		
SCALING AND TRIMMING				
Scaling along north face of track, 200 m , allow	20	20		
Mucking and trimming in blasted areas, allow	20	20		
Rock trimming by jack hammer in abutment areas, allow	30	0		
Grading of beam assembly areas, etc., allow	40	20		
Total hr	110	60		
Unit price	180	250		
	19800	15000	**Total scaling cost**	**$34,800**

LINE DRILLING (presplit holes)

This could equally form part of blasting program but is estimated here
holes @ 0.2 m centres to protect central pillar and abutments

Horizontal line drilling, central pillar, 4 faces 7 m height x 10 m width			
4x 35 holes = 140 holes 10 m long @ $60/m	$84,000		
Horizontal line drilling, abutments 2 faces 7 m height, 10 m width			
Allow 2x 35 holes 10 m long @ $60	$42,000		
		Total line drilling	**$126,000**

SHOTCRETE

North and south end-faces of Hwy 69 cut, allow 120 x 8 m @ $65/sq m	$62,400		
Full height of central pillar, 4 faces, 7 m high, 40 m perimeter = 280 sq. m	$18,200		
Two abutments, allow same area as pillar	$18,200		
		Total shotcrete	**$98,800**

BOLTING

BOLTING 25 mm dia fully resin grouted
Central pillar, prebolt from N and S 20 x 3 m bolts at 1.5 m centres, 60 bolts
Pillar and abutments, horizontal from within the undercut, 4X12 = 48 bolts
Abutments, allow same as pillar, 60 bolts

No.	$/bolt		
188	$80	**otal bolting**	**$15,040**

INSPECTION

Full time inspection. Rock engineer 14 days, 9 hrs/day @ $110/hr	$13,860		
Technician, 14 days, 9 hrs/day @ $50/hr	$6,300		
Installation of heave gauges, 4 gauges @ $200/each	$800		
One readout instrument, $250	$250		
		Total inspection	**$21,210**
		TOTAL	**$295,850**

Fig. 12 Undercut alternative: stabilization costs

	NAME	DATE
Drawn By	Massoud Palassi	3 Sept. 1998
Checked By	John A. Franklin	3 Sept. 1998
Revisions		

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PROJECT: G819.2

FIGURE: 12

APPENDIX 1 – DRILLHOLE LOGS

APPENDIX 1
CORE LOGS

DRILLHOLE LOG CPR-Highway 69 Grade Separation	Drillhole **3795-1**

DEPTH not to scale	ROCK DESCRIPTION	DISCONTINUITIES	Block size	RQD
0.00 - 2.57 m	Pink , black and grey GNEISS, banded with concentrations of pink feldspar and black hornblende, some biotite mica, moderately coarse grained, (1-4) 2 mm, strong with no signs of penetrative weathering or alteration. Mod. easy to break with hammer along well-developed gneissocity.	3 sets, one formed by breaks along the gneissocity typically 50° to the core axis, and two steeply dipping sets at 5 - 15° to the core axis. All sets rough, planar. the steeply dipping sets are in most places iron-stained (brown) with slight penetrative weathering; however, the joint surfaces remain hard.	32 cm	97%
2.57 - 2.82 m	As above but more micaceous and closely jointed	Long axes of joint ellipses: acute angle to core cross-section circle (= dip for vertical drilling) 350 mm; 82° 200 mm; 77° 115 mm; 66° 110 mm; 65° 54 mm; 29° 50 mm; 20° Core dia = 47 mm	6 cm	0%
2.82 - 3.82 m	Pink GNEISS, massive		33 cm	100%
3.82 - 4.92 m	GNEISS as above but closely jointed, containing one 8 mm thick clay-filled joint at an angle of 10° to core axis at depth 3.85 (may be wash from topsoil above, but could be geological infill or gouge)		9 cm	7%
4.92 - 10.34 m	Alternating pink and grey GNEISS as above		26 cm	99%
10.34 - 12.84 m			62 cm	100%
12.84 - 15.48 m			24 cm	98%

Logging by Franklin Geotechnical Ltd., Rock Engineers
Drill Supervision by Peto MacCallum Ltd., Geotechical Engineers

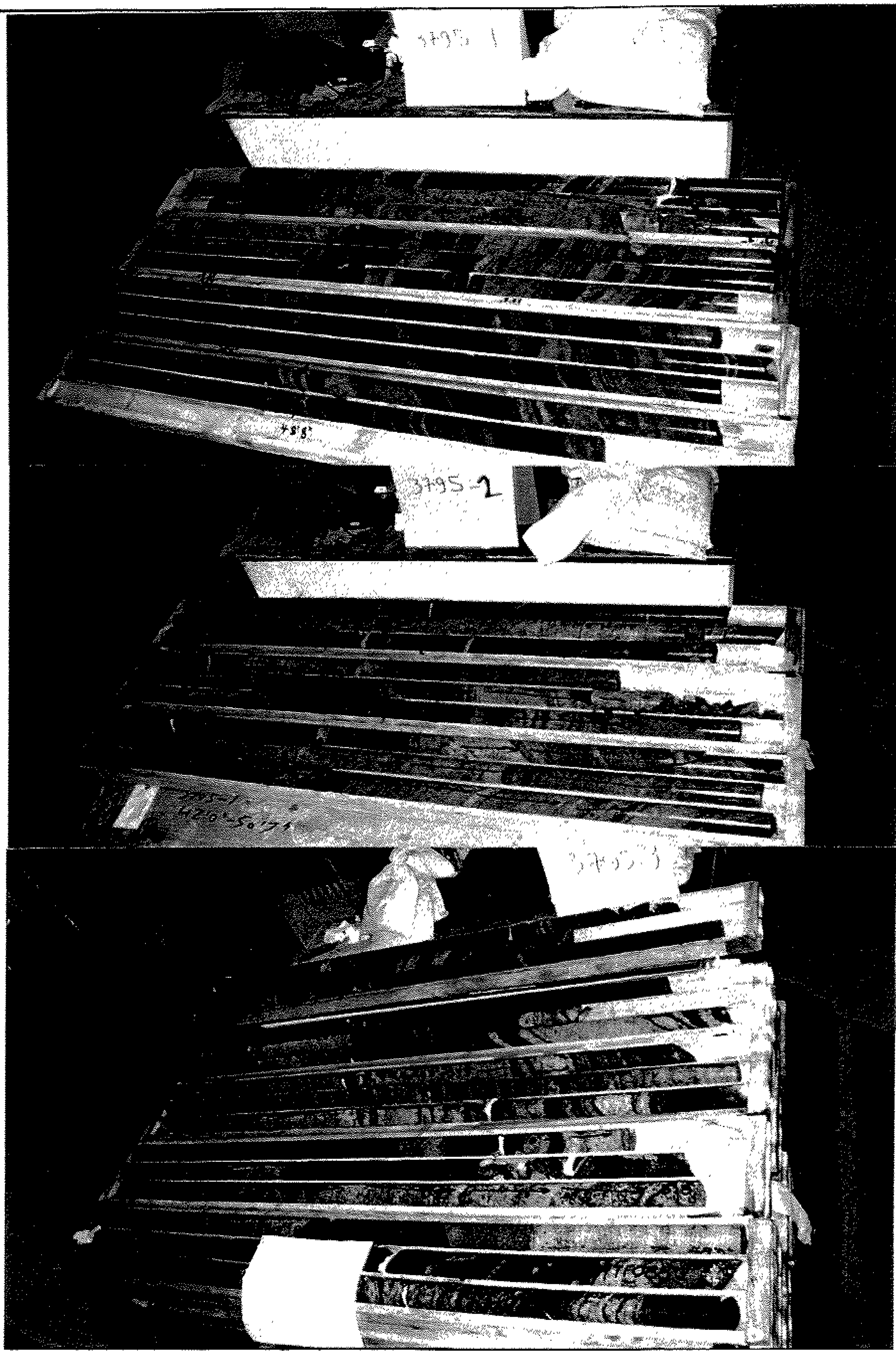
DRILLHOLE LOG - CPR-Highway 69 Grade Separation		Drillhole **3795-2**		
DEPTH not to scale	**ROCK DESCRIPTION**	**DISCONTINUITIES**	**Block Size**	**RQD**
0.23 - 1.05 m	Black GNEISS, highly weathered and friable, stained brown along joints with some penetration into the rock material	3 sets, one formed by breaks along the gneissocity typically 50° to the core axis, and two steeply dipping sets at 5 -15° to the core axis. All sets rough, planar. the steeply dipping sets are in most places iron-stained (brown) with slight penetrative weathering; however, the joint surfaces remain hard.	3(2-12)	20%
1.05 - 3.25 m	Black, pink and grey GNEISS, bands of 0.2 to 0.8 m with concentrations of pink and white feldspar and black hornblende, some biotite mica. Moderately coarse-grained, (1-4) 2 mm, strong with no signs of penetrative weathering or alteration, but mod. easy to break with hammer along well-developed gneissocity.	Long axes of joint ellipses: 410 mm; 84° 200 mm; 77° 160 mm; 73° 116 mm; 66° 110 mm; 65° 52 mm; 25° core dia = 47 mm	15(4-30)cm	72%
3.25 - 4.89 m	Dark grey to black GNEISS, micaceous, moderately fissile.		18cm	89%
4.89 - 6.01 m	Pink granite-GNEISS		37 cm	100%
6.01 - 7.51 m	Dark grey micaceous GNEISS		21 cm	92%
7.51 - 16.13 m	Pink granite GNEISS		48(15-128)cm	96%

Logging by Franklin Geotechnical Ltd., Rock Engineers
Drill Supervision by Peto MacCallum Ltd., Geotechical Engineers

DRILLHOLE LOG - CPR-Highway 69 Grade Separation		Drillhole 3795-3		
DEPTH not to scale	**ROCK DESCRIPTION**	**DISCONTINUITIES**	**Block Size**	**RQD**
0.22 - 2.78 m	Black and white micaceous GNEISS, weathered brown in places. banded with concentrations of pink feldspar and black hornblende, some biotite mica, moderately coarse grained, (1-4) 2 mm, strong with no signs of penetrative weathering or alteration. Mod. easy to break with hammer along well-developed gneissocity.	3 sets, one formed by breaks along the gneissocity typically 50° to the core axis, and two steeply dipping sets at 5 -15° to the core axis. All sets rough, planar. the steeply dipping sets are in most places iron-stained (brown) with slight penetrative weathering; however, the joint surfaces remain hard.	4(2 - 12) cm	88%
2.78 - 5.53 m	black micaceous GNEISS, homogeneous, weathered, fissile.	Long axes of joint ellipses; angle betwen joint and core cross-section = dip 280 mm; 80° 58 mm 36° 54 mm; 29° gneissocity 50 mm 20° Core 47mm dia	23 cm	96%
5.53 - 10.20 m	Alternating black and pink GNEISS in bands of about 0.7 m, coarse grained.		42 cm	100%
10.20 -10.44 m	black micaceous GNEISS stained along joints, friable and broken		8 cm	0%
10.44 – 18.10 m	Alternating black and pink GNEISS in bands of 0.5 to 1.0 m, coarsely crystalline		40 cm	99%

Logging by Franklin Geotechnical Ltd., Rock Engineers
Drill Supervision by Peto MacCallum Ltd., Geotechical Engineers

APPENDIX 2 – CORE PHOTOGRAPHS



	NAME	DATE
Drawn By	Massoud Palassi	2 Sept. 1998
Checked By	John A. Franklin	2 Sept. 1998
Revisions		

franklin geotechnical engineering

PROJECT: G819.2	FIGURE: A2.1

APPENDIX 3 – OUTCROP PHOTOGRAPHS

	NAME	DATE	franklin geotechnical engineering	
Drawn By	Massoud Palassi	2 Sept. 1998		
Checked By	John A. Franklin	2 Sept. 1998	PROJECT: G819.2	FIGURE: A3.1
Revisions				

	NAME	DATE
Drawn By	Massoud Palassi	2 Sept. 1998
Checked By	John A. Franklin	2 Sept. 1998
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PROJECT: G819.2

FIGURE: A3.2

	NAME	DATE
Drawn By	Massoud Palassi	2 Sept. 1998
Checked By	John A. Franklin	2 Sept. 1998
Revisions		

franklin geotechnical engineering

PROJECT: G819.2

FIGURE: A3.3

APPENDIX 4 - SPECIFICATIONS

APPENDIX 4
SPECIFICATIONS FOR ROCK STABILIZATION WORK

The following are draft contributions to contract specifications for the rock work on this project. Contributions to blasting specifications are contained in the text of the report, Section 3.4.

SCALING AND TRIMMING

Hoe-Ram Scaling

Supply and operate tractor-mounted hoe-rams to sound, scale and trim the rock face at locations and to the extent directed by the Contract Administrator.

Employ only operators that are highly skilled and experienced in trimming and scaling work. Remove only hazardous rock as and where directed, and avoid loosening rock that is to remain in place. Scale only while in full radio communication with the Contractor's foreman.

Supply and maintain hoe-rams in good operating condition and equipped with a 550 kg-m (4,000 ft-lb) or larger rock breakers, maximum operating height at least 11 m.

Complete all scaling and trimming from the top down to avoid producing unsafe overhangs, and from the side beneath a previously scaled face so that the machine is not endangered by unstable rock.
Provide all necessary materials, personnel and equipment for ramping and access.

Manual Scaling

Provide a two-person manual scaling crew to carry out rock stabilization work as and where directed by the Contract Administrator, mainly at locations not readily accessible to the hoe-ram. Manual scaling work shall mean all authorized work by the manual scaling crew, not only scaling but also grubbing, sounding, trimming with hand-tools and hand-held machine tools, and where directed also trim-blasting with small quantities of explosives.

Employ only personnel that are trained, experienced and fully competent to carry out efficiently all necessary operations both low and high on the rock face and upper rock slopes. Scale and trim only while in full radio communication with the Contractor's foreman.

Maintain the crew equipped at all times with the tools and materials needed for efficient performance of the work including radio communication, climbing equipment, ropes and safety harnesses, scaling bars, rock drills, rock splitting wedges, jacking cables, chain saws, cutters, shaped-charge emulsion explosives, and the fuel or power needed for operation of tools. Provide traffic control persons and signing.

Provide as and where needed a crane-mounted suspension platform and operator or alternative means of access to rock faces and slopes up to 30 m above and 30 m beyond the edge of highway. Provide alternative face access with the aid of ropes and harnesses.

Complete grubbing and remove from site all grubbed materials including earth, trees and vegetation before the start of rock scaling. Do not allow vegetation to mix with loose rock in the ditch.

Complete all sounding, scaling and trimming from the top down to avoid producing unsafe overhangs, and from the side beneath a previously scaled face so that personnel are not endangered by unstable rock.

Rock Removal

The item applies to rock faces designated for scaling and trimming and not to locations of bulk or bench blasting, where rock removal is included in the unit price for blasting. The work includes loading and removal to a location approved by the Contract Administrator of all loose rock and debris that accumulate on the highway and shoulders and within ditches.

Stockpile and ramp scaled rock within the work area as required to give access for scaling and remove excess materials promptly when no longer required for ramping.

Dispose of as much of the excavated materials as possible within the right of way, adjacent to the embankments and conforming to standard right of way offsets, by widening embankments, flattening side slopes and constructing modified cross sections, as specified.

Dispose of surplus excavated material that cannot be accommodated within the right of way by loading and hauling to other disposal areas as specified in the Contract, or otherwise outside the right of way in areas provided by the Contractor. The Contractor shall be fully responsible and shall indemnify the Government of Ontario for any and all costs and consequences of haulage and disposal.

Level and trim to smooth slightly the contours of all rock within disposal areas.

ROCKBOLTING

Install fully resin-grouted rockbolts at locations directed.

PRODUCTS

Rockbolts 2.0 m long (3.0 m long for special applications) 25 mm diameter (minimum) ribbed bar, threaded at one end and provided with a 100 mm square faceplate, nut and beveled or spherical washers to allow tensioning with the faceplate fully in contact with the rock or shotcrete surface while rotated at angles of $\pm 30°$ from perpendicular to the bolt.

Alternative forged-head bolts with no thread but same faceplate & washers.

Polyester resin cartridges, fast-set (10 to 20 minute). The rockbolts are to be fully resin-grouted. Drillhole diameter and cartridge type (length & setting time) are to be recommended by the supplier in order to obtain full bonding.

PROCEDURES

Drill the rockbolt holes at the diameter recommended by the resin supplier for the sizes of bolt and cartridge, and in a direction perpendicular to the rock or shotcreted face or as otherwise directed by the Contract Administrator. Drill to a measured depth so that when bolts are fully inserted in the completed drillhole they extend from the rock or shotcrete surface by 70 ±20 mm. Clean the holes using compressed air from the drill or a compressed air blowpipe. Insert sufficient polyester resin cartridges to ensure bond along the full length of hole with some overflow of resin, then insert and spin home the rockbolt in accordance with the resin supplier's installation instructions.

TESTING

Each rockbolt not fully bonded (drillhole not completely filled with hardened resin) or which protrudes by an amount greater or less than the specified amount is to be replaced at the Contractor's expense by another installed alongside.

SHOTCRETING

Apply sprayed concrete (shotcrete) in one layer 70±20 mm thick to cover all rock. Responsibility for all mix designs and the quality of in-place shotcrete shall remain with the Contractor. The materials to be shot shall consist of the following mix or an approved alternative:

Normal Portland cement, 18-21% by weight of dry components, conforming to ASTM Standard C150;

Silica fume (microsilica), 10-15% by weight of Portland Cement, containing minimum of 90% SiO_2 and having a proven record of performance for use in shotcrete;

Aggregate, Gradation No. 2 in ACI 506.2-77 having a well-graded distribution with maximum size 10 mm and conforming to the requirements of ASTM Standard C33

Steel fibre reinforcement to standard ASTM 820-90 or approved equivalent in quantity recommended by the supplier;

Water, clean and free of substances which might be deleterious or corrosive to concrete or steel, and sufficient to give good placement characteristics and a ratio of water to total cementitious materials in the range 0.35:1 to 0.40:1;

Additives such as superplasticisers, high-range water reducing agents or air entrainers as required to achieve optimum strength and placement characteristics.

The foreman shall have at least 2 years experience as a shotcrete nozzleman. The nozzleman shall have served at least 6 months apprenticeship on similar applications and shall be able to demonstrate his ability to apply shotcrete of the required quality and uniformity.

All shotcrete shall achieve a minimum compressive strength after 3 days of 20 MPa and after 7 days of

30 MPa according to ASTM Standard C42. The Contractor will be required to shoot up to five test panels on site at the start and during the work as directed by the Inspector. Specimens from the panels may be tested at no cost to the Contractor as necessary to determine compliance with this performance specification.

Maintain applied shotcrete moist by water spraying at the start, middle and end of each working day for the first three days after placement.

When the shotcrete has gained sufficient strength and within 36 hours after placement, test in the presence of the Inspector all in place shotcrete by hammering at about 300 mm centres to detect "drummy" areas resulting from inadequate mixing or materials, rebound pockets, or poor bonding to the substrate. Shotcrete determined by the Inspector to be unsatisfactory or inadequately bonded is to be broken out by hammering or pneumatic tools and replaced by further shotcrete and the sounding, breaking out and replacement repeated until adequate shotcrete quality and bonding have been achieved.

SUBMISSIONS

Schedule:	Schedule showing the anticipated workforce and equipment availability, and duration of the work.
Shotcrete:	Shotcrete mix design, and names and experience of the foreman and nozzlemen.

MEASUREMENTS FOR PAYMENT

Shotcreting will be paid by the square metre of applied shotcrete meeting specifications, determined by tape measurements along and in contact with the shotcrete surface. The area for payment shall be that of all shotcrete at least 50 mm thick. Shotcrete required to fill cavities, where specified by the Inspector, shall be paid as two or more layers of equivalent volume, as determined on site by the Inspector. Thicknesses shall be confirmed if necessary by core drilling. The unit price is to include all surface preparation, shotcreting, rebound and wastage, provision of shotcrete test panels, sounding of shotcrete surfaces, replacement of defective shotcrete, and verification of shotcrete thicknesses.

memorandum

Ontario

To:	Mike Pearsall, P. Eng. Senior Project Manager Planning and Design Section Northern Region	2000 09 07
From:	Pavements and Foundations Section Room 223, Central Building Downsview, Ontario	
Re:	Final Foundation Report CP Rail Grade Separation Highway 69 - 4 Laning from Tower Road (MacTier) Northerly 26.5 km to 2 km north of Highway 141 G.W.P. 290-97-00, W.P. 402-97-01, Site: 44-379, Hwy 69, District 52, Huntsville	

We have conceptually reviewed the final Foundation report, produced by Peto MacCallum Ltd. for McCormick Rankin Corporation dated July 2000 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 and liability of the consultant. The Ministry assumes no responsibility or liability for these aspects of the report. These aspects will be reviewed in order to assess the Consultant's performance in this assignment upon implementation of the recommendations in the design and upon review of the performance of the foundations for the completed project. Our comments are as follows:

- The recommendations in the Foundation report are as agreed during several discussions, and comments from our office on this project. In the report, the Diversion Alternative (the most favored option) is recommended for design.

- In our memo dated 1998 11 12 we had requested that a cross section should be provided in the final Foundation report showing the geometry of the proposed rock excavation in relation to the existing railway track. However, the cross section is not provided in the final report.

If you have any questions, please advise.

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

cc. T. Kazmierowski

File:c:\ken ahmad\2909700.mike5.doc.

memorandum

Ontario

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

1998 11 12

From: Pavements and Foundations Section
Room 223, Central Building
Downsview, Ontario

Re: CPR Track Diversion at the Proposed Hwy 69 Bridge Location
Highway 69 - 4 Laning from Tower Road (MacTier)
Northerly 26.5 km to 2 km north of Highway 141
Hwy 69/CPR Subway, Parry Sound Subdivision, Mileage 6.63
GWP 290-97-00, Hwy 69, District 52, Huntsville

We have conceptually reviewed the design/construction (Foundation) reports, produced by Peto MacCallum Ltd. Consulting Engineers dated September 1998 for McCormick Rankin Corporation and a report by Franklin Geotechnical Ltd., for Peto MacCallum Ltd dated September 4, 1998, 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 and liability of the consultant. The Ministry assumes no responsibility or liability for these aspects of the report. These aspects will be reviewed in order to assess the consultant's performance in this assignment upon implementation of the recommendations in the design and upon review of the performance of the foundations for the completed project. After reviewing the reports we discussed our concerns in a meeting on October 9, 1998 in the office of McCormick Rankin Corporation. Our comments that we discussed in the meeting are as follows:

Diversion Alternative: In the Franklins report (Page 8) it is proposed to construct the bridge and the track diversion at an offset of 20m from the existing tracks (earlier an eight-meter offset was proposed). We understand (Figure 5 of the Report) that the proposed excavation will be up to 15m deep and the bottom of the excavation will be at a distance of 15m from the railway track. A 3m mid height bench is also proposed. It appears that as per the new proposal, the excavation will be kept outside an imaginary line drawn at 0.5H:1V from the edge of the railway. A cross section in a true scale should be provided in the final report to clearly define the geometry of the excavation. The Consultant's recommendation appears to be reasonable if the excavation to construct the new bridge is kept outside an imaginary plane drawn at 0.5H:1V from the edge of the railway and the side of the rock cut is properly stabilized as proposed in the Consultant's report.

Undercut Alternative: We learnt in the meeting that this option, is not under consideration anymore. We also felt that this option was not feasible. The integrity of the proposed rock column (to act as a pier for the proposed bridge) would be questionable after the blasting and excavation was carried out around it. The estimated time to carry out the full operation (lifting the existing tracks, rock blasting and excavation and construction of the bridge deck within a day) did not appear to be sufficient.

It was noted and acknowledged by others, that on Page 2 of the Peto MacCallum's report indicated that the geotechnical boreholes were drilled at an inclination of 16 to 17.5 degrees from the vertical, whereas the Franklin's report (Page 1) indicated these angles to be degrees down from horizontal. This should be corrected in the final report.

The locations of the boreholes should be shown on the plan.

If you have any further questions, please advise.

K. Ahmad, P. Eng
Foundation Engineer

For

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

cc: S. Senior
J. McDougall
T. Kazmierowski

Peto MacCallum Ltd.

CONSULTING ENGINEERS

April 23, 1998 | Our Ref: 97TF088A

Mr. Marko Prgin, P.Eng.
Totten Sims Hubicki Associates
30 Dupont Street East
Waterloo, Ontario
N2J 2G9

Dear Mr Prgin

C.P. Rail Diversion
Mile 6.63 Parry Sound Subdivision
<u>Station 15+662, Highway 69 (Humphery/Coger)</u>

We understand from our recent discussions and our April 8, 1998 meeting with CPR and MTO personnel in Sudbury that construction of an approximate 500 m long track diversion about 8 m north of the existing track is the preferred method of constructing the grade separation structure at this location.

We also understand that:

- an approximate 6 m rock cut is required to lower the existing grade to the existing track level along the centreline of the proposed alignment of Highway 69.

- the finished grade of highway 69 will be at least 7 to 9 m below railway grade at the crossing.

We attended the site following our meeting in Sudbury to examine the rock cut at this location in order to obtain preliminary information concerning the inclination/orientation of the joint sets and the dip/direction of the bedding planes.

It was noted that the maximum depth of rock cut along the proposed diversion was approximately coincident with the Highway 69 crossing; the depth of rock cut decreased to about 1 to 2 m within 100 m to the north and south of the Highway 69 centreline. The rock cut over the remainder of the track diversion was judged to be less than 1.5 m.

The bedrock at this location is "gneiss", a highly metamorphosed sedimentary rock. Major joint sets are inferred at about 7°, 80° and 140° to the tangent of the alignment of the track diversion; bedding planes dip at 30 to 45° to the southwest, ie. downward towards the existing tracks.

...2

45 Burford Road, Hamilton, Ontario L8E 3C6
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E-mail: ham@petomac.on.ca
AURORA, BARRIE, BRAMPTON, HAMILTON, KITCHENER, TORONTO

Excavation of the bedrock along the track diversion and the underlying Highway 69 corridor should be carried out in a series of stages to minimize the impact to the existing track. General comments in this regard are provided for consideration during planning/design of construction of the diversion and highway. Refer to Figure 5 prepared by Franklin Geotechnical Engineering for a schematic illustration of the each step.

Step 1 Widen the clear zone between the rail and existing rock face to at least 4 m to protect the track from incidental rock fall induced by blasting. This could be accomplished by mechanical means (toothed excavator, hoe ram, air track drills and rock splitters)

As an alternate, the rail could be protected by installing rock reinforcement along the existing face prior to excavation of the adjacent rock.

Step 2 Excavate rock within the Highway 69 corridor down to the subgrade level of the track. This work should commence well away from the track and proceed from north to south towards the track.

The perimeter of the excavation should be presplit and blasting mats employed to prevent "fly rock" during blasting, particularly within 500 m of the track.

The excavation should be carried out in maximum 3 m deep "lifts". The face of the excavation should examined, scaled as appropriate and supported with rock bolts as needed.

Ground vibrations inducted by blasting must be monitored to determine the pertinent characteristics of the rock at this location and establish an acceptable limit for peak particle velocity (ppv) (approximately 30 mm/s, subject to results of rock coring and response of rock to blasting).

The blast pattern, delay sequence, charge etc. will be adjusted as required to keep the ppv within the acceptable limit.

Step 3 When ppv exceeds the acceptable limit, excavation to continue using light blasting to keep the ppv below the acceptable limit and/or mechanical equipment as noted in stage 1.

Scaling and support of the excavation slopes should be continued as noted in Step 2.

Step 4 Excavate rock along the proposed track diversion employing blasting to loosen the rock (subject to ppv) and/or mechanically. Blast mats must be installed to control "fly rock".

...3

Step 5 Excavate the remaining rock within the Highway 69 corridor to the road subgrade level using the procedures described in Step 2.

A 3 m wide bench should be left at the base of the Step 2 excavation.

Work could be performed at different locations in Steps 2 and 5 simultaneously.

Step 6 When the ppv exceeds the acceptable limit as the excavation nears the track, proceed in accordance with Step 3.

Step 7 Build the track diversion and bridge.

Step 8 Excavate the remaining rock within the Highway 69 corridor to the road subgrade level from south to north using the procedures described in Steps 2 and 3.

We believe excavation of the bedrock and construction of the track diversion can be carried out with minimal impact to the existing track during construction by adopting these procedures. All work will be scheduled/performed in accordance with CPR operational requirements. Detailed comments can be provided when testholes are drilled to confirm pertinent engineering/geological properties of the rock.

We trust this brief description of the excavation procedures is sufficient and look forward to any questions you may have.

Sincerely

Peto MacCallum Ltd.

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

DWK:mma

1 cc: Totten Sims Hubicki Associates
1 cc: McCormick Rankin Associates, Paul Turner
1 cc: PML Toronto, B. Gray

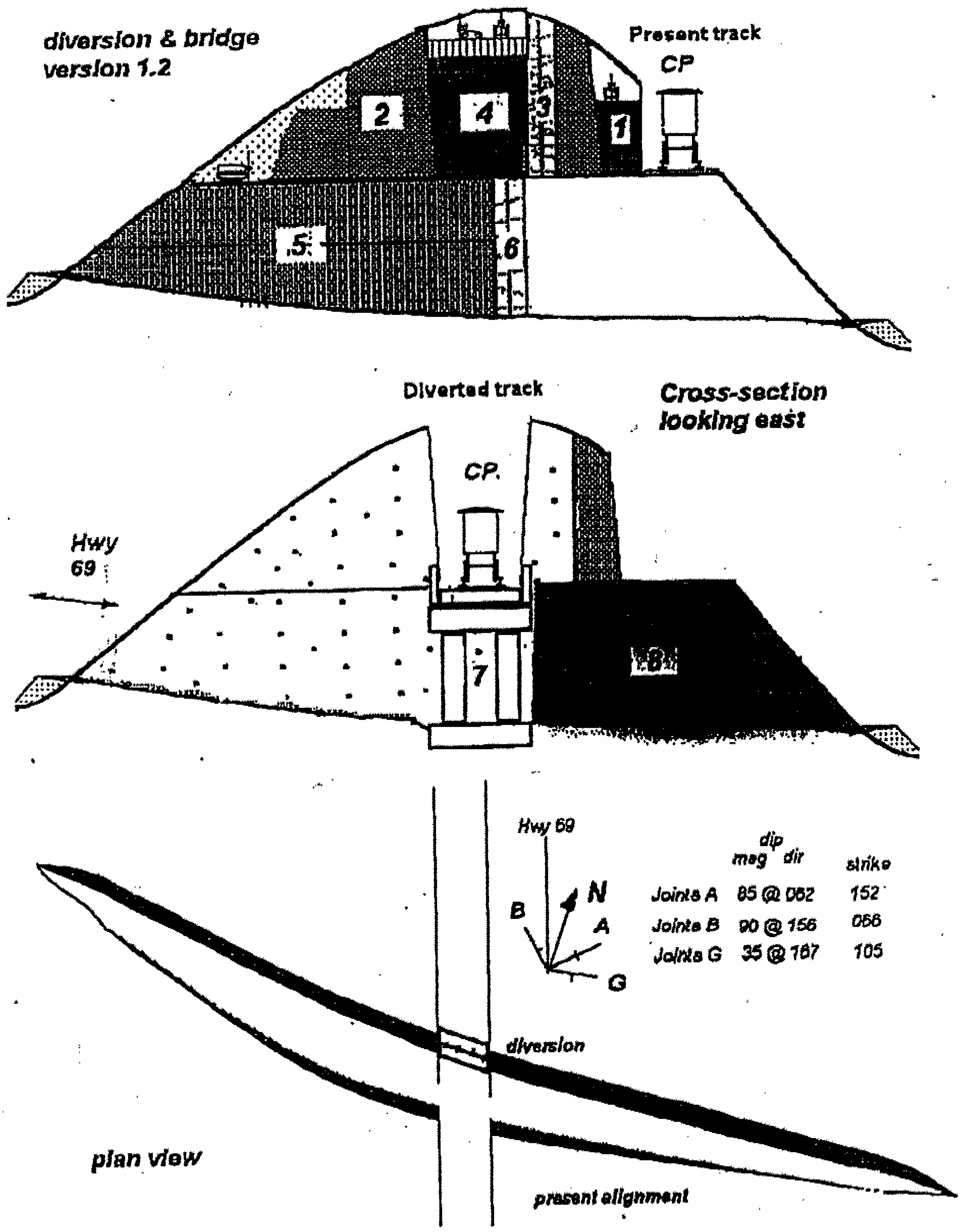

Fig. 5: Stages in Excavating Hwy69/CPR Grade Separation (Version 1.2)

	NAME	DATE
Drawn By		
Checked By		
Revisions		

franklin geotechnical engineering

PROJECT: G819

FIGURE: 5

MRC

FACSIMILE TRANSMISSION

TO: MTO - NORTHERN REGION

ATTENTION: PAUL LECOARER, P.ENG.

No of sheets transmitted: 7 including this sheet. MAIL: YES ___ NO X

FROM: PAUL TURNER DATE: MAY 13/98

W.O. NO. 3641.100 PROJECT: W.P. 290-97-00 HWY. 69

MESSAGE:

Paul

Please find attached methodology & costs associated with the CPR re-alignment geotechnical work

Y.U.T.

Paul

PLEASE COPY PAUL ASAP

FAX NO: 1-705-497-5499

McCORMICK RANKIN CORPORATION

CONSULTANTS IN TRANSPORTATION

2655 North Sheridan Way, Mississauga, Ontario, Canada L5K 2P8
Tel: (905) 823-8500 Fax: (905) 823-8503 E-mail: mrc@mrc.ca

memorandum

To: Paul Lecoarer, P. Eng.
Senior Project Engineer
Planning and Design Section
Northern Region

1998 07 27

From: Pavements and Foundations Section
Room 223, Central Building
Downsview, Ontario

Re: CPR Track Diversion at the Proposed Hwy 69 Bridge Location
Highway 69 - 4 Laning from Tower Road (MacTier)
Northerly 26.5 km to 2 km north of Highway 141
Hwy 69/CPR Subway, Parry Sound Subdivision, Mileage 6.63
GWP ~~298~~-97-00, Hwy 69, District 52, Huntsville
290

This is further to your request to review the above-mentioned proposal. We met Brian Ruck of Totten Sims Hubicki Associates on July 22, 1998 who briefed us on the project.

We understand that in order to construct the Hwy. 69 and CPR subway at the above location, a 2.2 km long permanent CPR track diversion was identified. This proposed diversion was at an offset of about 85m from the existing CPR tracks. As part of the value engineering, Totten Sims Hubicki Associates presented an alternative conceptual design that involved only 500 m long permanent track diversion at an offset of 12 m (centre to centre) from the existing CPR track.

From the plan and profile provided by Totten Sims Hubicki Associates, the CPR tracks are at elevation 247.8m. The grade of proposed Hwy 69 CPR tracks (at 12m offset) will be at elevation 239.1 m. The depth of excavation to construct the bridge will be about 9m.

The alternative conceptual design presented by Totten Sims Hubicki Associates appears to be reasonable. However, in our opinion the proposed CPR track alignment at 12m offset (centreline to centreline between existing and proposed tracks) may not be feasible due to the following:

1. Depending on the size of the footing, the excavation to construct the bridge foundation may come within 5m of the tracks. Since the depth of the excavation will be more than 9m, the bottom of the excavation, near the track may fall within an imaginary line drawn at 0.5H:1V from the edge of the tracks to the base of the

excavation, which in our opinion is too close. The schematic cross section of excavation (Figure 5) presented in Peto MacCallum's letter dated April 23, 1998 is not to scale and does not show the true geometry of the excavation.

2. The CPR track will be in use, carrying loads, and therefore, 9m deep excavation within 5m of the tracks will not be appropriate.

3. The side slope of the excavation due to rock blasting may be damaged due to over-blasting within the 0.5H:1V imaginary line and may not be suitable to support the load on the tracks.

In view of the above, we propose the following:

1. The side slope of the excavation may need to be mechanically supported and protected depending on the joint structure pattern, orientation and block stability. The cost and feasibility of such an option should be studied and included in the Value Engineering review.

2. As an alternative, the proposed alignment of the track diversion should be revised, so that the excavation to construct the bridge foundation should be outside an imaginary plane defined by 1H:1V from the edge of the CPR track (not from the centreline of the track) to the bottom of the excavation.

3. Three cross sections in true scale at each footing locations should be provided to verify that the excavation is outside the 1H:1V imaginary line.

4. The slope of the excavation and railway track should be monitored, and surveyed during excavation for instability or movement. Necessary measures should be taken to avoid any problems.

We understand that Peto MacCallum has proposed to drill three inclined boreholes (at 15 degrees). The boreholes will be 22m deep. We are asked to comment on the proposed cost of the Foundation Investigation. The cost for background review, CPR meeting, site visits, rock design by Franklin Geotechnical, Engineering, site supervision, analysis, report preparation and meeting appears to be reasonable. However, the cost for drilling appears to be high. As suggested by Peto MacCallum, if the fieldwork takes about two weeks to complete, then the cost of drilling should be between $25,000 to $30,000. However, the drilling cost may be higher due to other factors such as difficulty in access to the site, actual length of coring, availability of water near the site for drilling or transportation of water to the site. Coring at an angle up to 22m deep may cause frequent breakdown of the drilling machine. Also, there may be delays in drilling due to the shutdown when the CPR tracks are in use.

This memo was reviewed by Steve Senior, Senior Soils & Aggregates Engineer. If you have any further questions, please contact this office

K. Ahmad, P. Eng
Foundation Engineer

For

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

cc: S. Senior
C. Rogers
T. Kazmierowski

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31E-185

DIST. 52 REGION

G W.P. No. 290-97-00(E)

CONT. No.

W. O. No.

STR. SITE No. 44-379

HWY. No. 69

LOCATION Hwy 69 & CPR Subway

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

PetoMacCallum Ltd.
CONSULTING ENGINEERS

GEOCRES # 31E - 185

FOUNDATION INVESTIGATION AND DESIGN REPORT FOR CP RAIL GRADE SEPARATION W.P. 402-97-01 G.W.P. 290-97-00, SITE 44-379 HIGHWAY 69, DISTRICT 52 HUNTSVILLE, ONTARIO

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Job No. 97TF088G
Geocres No. Not Assigned

July, 2000

PetoMacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT
FOR
CP RAIL GRADE SEPARATION
W.P. 402-97-01
G.W.P. 290-97-00, SITE 44-379
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

Job No. 97TF088G
Geocres No. Not Assigned

July, 2000

TABLE OF CONTENTS

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT
For
CP Rail Grade Separation
W.P. 402-97-01
G. W.P. 290-97-00, Site 44-379
Highway 69, District 52, Huntsville

INTRODUCTION

This report summarizes the results of the foundation investigation carried out for construction of the proposed CP Rail grade separation structure at Highway 69 (Station 15+677 Highway 69 chainage).

The report pertains to the proposed bridge structure and approaches within 20 m of the abutments, between approximate stations 9+940 and 10+060, CPR chainage.

SITE DESCRIPTION

The site is located about 9 km north of MacTier and about 1.4 km west of the existing Highway 69 alignment. The proposed structure will carry CP Rail traffic over the proposed new four-lane section of Highway 69. At the underpass, the CP railway will run east-west along a diversion alignment situated approximately 16.6 m north of the existing CP Rail alignment.

The proposed bridge location comprises a bedrock outcrop. The existing CPR alignment to the south is located in a rock cut. The ground surface rises slightly to the east parallel to the railway. The area north of/adjacent to the site is heavily wooded.

The area is part of the Precambrian Laurentian peneplane. In general, the topography is relatively flat but quite irregular in detail with many small lakes separated by rocky ridges. 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.

The bedrock formations are of Precambrian age and are largely composed of veined, banded, and homogeneous pink and grey gneisses produced by injection and granitization of metamorphic gneisses of various types.

INVESTIGATION PROCEDURES

The fieldwork was carried out during the period August 11 to 18, 1998 and comprised three rock cores put down at the locations shown on Drawing 1. The core locations were situated along a preliminary alignment of the rail diversion located approximately 7 m south of the presently proposed alignment.

The cores were extended to depths of 15.4 to 17.9 m at an inclination of 16.0 to 17.5° from the vertical, oriented 120 to 130° apart. Coring commenced at the rock surface at depths of 0 to 230 mm.

The cores were advanced using NQ rock coring equipment, powered by a track-mounted CME-75 drillrig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. The core recovery and Rock Quality Designation (RQD) were documented in the field as the cores were recovered. The composition of the thin overburden layers was also recorded.

Franklin Geotechnical Ltd. (FGL) was retained to provide a detailed description of the rock characteristics as well as comments/recommendations regarding the rock engineering aspects of the Highway 69/CPR grade separation structure (Report No. G819.2 dated September 4, 1998, revised March 22, 1999). The recovered rock core samples were provided to FGL for detailed visual examination and classification.

SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Log of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, rock core descriptions and groundwater observations. A stratigraphic profile prepared from the borehole data is presented on Drawing 2. The ground surface and probable bedrock profiles along the proposed CP Rail centreline (derived from borehole information obtained during the geotechnical investigation for the rail diversion (refer to Table II) and the profile of original ground provided) are also shown on Drawing 2.

The stratigraphy revealed in the boreholes comprised a discontinuous veneer of topsoil or sand overlying bedrock. The strata encountered are summarized below.

Topsoil

A 220 mm thick layer of topsoil was encountered surficially in borehole 379S-3. The topsoil comprised dark brown silty sand.

Sand

Silty sand with cobbles was encountered surficially in borehole 379S-2. The sand mantled bedrock at a depth of 230 mm.

Bedrock

Bedrock was exposed surficially at borehole 379S-1 and contacted below the topsoil/sand at depths of 230 and 220 mm in boreholes 379S-2 and 3, respectively. The bedrock surface at the test locations undulates slightly ranging from elevation 250.8 to 253.1.

The bedrock was cored at each location to total depths of 15.4 to 17.9 m. Core recovery was generally greater than 90%. The RQD of the rock determined in the field as the core runs were recovered typically ranged from 50 to 100%. RQD values in the 0 to 25% range were measured in three cores recovered from Boreholes 379S-1 and 2.

A description of the recovered rock core is provided on Table I. In general, the bedrock consists of black, pink and grey gneiss. It is considered to be a high strength rock (Canadian Foundation Engineering Manual classification) that tends to split along mica-rich planes of gneissocity.

The upper 1 to 2 m of rock has deteriorated due to weathering/frost effects. Block size in the upper 4 m ranges from 50 mm to 400 mm, typically increasing with depth. The rock is essentially excellent quality with a very occasional layer (240 to 1100 mm thick) of very poor quality material, fair quality between 1.1 to 3.3 m depth in borehole 379S-2.

The bedrock is intersected by three main discontinuities: the gneissocity which dips towards the west-southwest at angles of 30 to 50°, and two near vertical joint sets. The spacing between discontinuities varies widely ranging from "close" (50 to 300 cm) to "wide" (1 to 3 m) (Canadian Foundation Engineering Manual classification). The discontinuity sets are rough, planar and tend to be continuous over a long distance. Little evidence of softening due to weathering was detected. The shear strength is considered to be high (peak friction angle estimated to be at least 60°); lower values (20 to 30°) may exist locally due to pre-sheared joints or micaceous surfaces.

The vertical rock stress is expected to be proportional to depth, increasing at a rate of about 30 kPa per meter. Horizontal stresses as high as 14 MPa are typical.

Groundwater

Approximately three weeks after coring (September 4, 1998), water was measured in core holes 379S-1 and 379S-3 at depths of 7.1 and 1.6 m (elevation 243.9 and 251.7). Water is used during coring; it is not clear whether the measured water levels reflect residual drillwater or groundwater levels. Groundwater levels are subject to seasonal fluctuations and rainfall patterns. A water level could not be obtained from core hole 379S-2 due to blockage.

Flow through the rock mass occurs only along joints. The joints are typically spaced at 300 to 400 mm intervals and appear to be moderately open. The hydraulic conductivity of the rock mass is estimated to be about 10^{-7} cm/sec, 10^{-5} cm/sec in the upper 2 m of weathered material.

CLOSURE

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

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

REGISTERED PROFESSIONAL ENGINEER
D. W. KERR
PROVINCE OF ONTARIO

Yours very truly

Peto MacCallum Ltd.

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

TABLE I

ROCK CORE DESCRIPTION
WP 402-97-01
GWP 290-97-01, Site No. 44-379

CORE RECOVERY

BOREHOLE	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)
379S-1	1	0.0 – 1.12	100	100
	2	1.12 – 1.68	95	77
	3	1.68 – 2.64	100	71
	4	2.64 – 4.17	100	65
	5	4.17 – 4.68	100	25
	6	4.68 – 5.69	98	55
	7	5.69 – 7.22	100	100
	8	7.22 – 8.74	100	78
	9	8.74 – 10.27	100	100
	10	10.27 – 11.79	100	100
	11	11.79 – 13.32	100	93
	12	13.32 – 14.84	100	85
	13	14.84 – 15.42	100	96

CORE DESCRIPTION

DEPTH (m)	DESCRIPTION	DISCONTINUITIES	BLOCK SIZE
0.0 – 2.57	**GNEISS,** pink, black and grey, banded with concentrations of pink feldspar and black hornblende, some biotite mica, moderately coarse grained, (1-4) 2 mm, strong with no signs of penetrative weathering or alteration. Mod. easy to break with hammer along well-developed gneissocity.	3 sets, one formed by breaks along the gneissocity typically 50° to the core axis, and two steeply dipping sets at 5-15° to the core axis. All sets rough, planar. The steeply dipping sets are in most places iron-stained (brown) with slight penetrative weathering; however, the joint surfaces remain hard.	32 cm
2.57 – 2.82	As above but more micaceous and closely jointed	Long axes of joint ellipses: acute angle to core cross-section circle (= dip for vertical drilling) 350 mm; 82° 110 mm; 65° 200 mm; 77° 54 mm; 29° 115 mm; 66° 50 mm; 20°	6 cm
2.82 – 3.82	Pink, massive	Core dia. = 47 mm	33 cm
3.82 – 4.92	As above but closely jointed, containing one 8 mm thick clay-filled joint at an angle of 10° to core axis at depth 3.85 (may be wash from topsoil above, but could be geological infill or gouge)		9 cm
4.92 – 10.34	Alternating pink and grey as above		26 cm
10.34 – 12.84			62 cm
12.84 – 15.42			24 cm

RQD = Rock Quality Designation

Core Recovery data provided by Peto MacCallum Ltd.
Core Description by Franklin Geotechnical Ltd.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PML Ref: 97TF088G

TABLE I Cont'd

ROCK CORE DESCRIPTION
WP 402-97-01
GWP 290-97-01, Site No. 44-379

CORE RECOVERY					CORE DESCRIPTION			
BOREHOLE	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION	DISCONTINUITIES	BLOCK SIZE
379S-2	1	0.23 – 1.27	56	20	0.23 – 1.05	**GNEISS,** black, highly weathered and friable, stained brown along joints with some penetration into the rock material	3 sets, one formed by breaks along the gneissocity typically 50° to the core axis, and two steeply dipping sets at 5-15° to the core axis. All sets rough, planar, the steeply dipping sets are in most places iron-stained (brown) with slight penetrative weathering; however, the joint surfaces remain hard.	3 cm (2-12)
	2	1.27 – 2.82	97	50				
	3	2.82 – 4.34	83*	54				
	4	4.34 – 5.87	**	-				
	5	5.87 – 7.39	**	-				
	6	7.39 – 8.00	88	0				
	7	8.00 – 8.92	100	100	1.05 – 3.25	Black, pink and grey, bands of 0.2 to 0.8 m with concentrations of pink and white feldspar and black hornblende, some biotite mica. Moderately coarse-grained, (1-4) 2 mm, strong with no signs of penetrative weathering or alteration, but mod. easy to break with hammer along well-developed gneissocity.	Long axes of joint ellipses: 410 mm; 84° 116 mm; 66° 200 mm; 77° 110 mm; 65° 160 mm; 73° 52 mm; 25° core dia. = 47 mm	15 cm (4-30)
	8	8.92 – 10.44	100	97				
	9	10.44 – 10.64	100	100				
	10	10.64 – 12.04	91	58				
	11	12.04 – 13.56	100	88				
	12	13.56 – 15.09	100	100	3.25 – 4.89	Dark grey to black, micaceous, moderately fissile.		18 cm
	13	15.09 – 16.38	100	100				
					4.89 – 6.01	Pink granitic		37 cm
					6.01 – 7.51	Dark grey micaceous		21 cm
					7.51 – 16.38	Pink granitic		48 cm (15-128)

* Lost 250 mm length of core down hole
** Equipment malfunction

RQD = Rock Quality Designation

Core Recovery data provided by Peto MacCallum Ltd.
Core Description by Franklin Geotechnical Ltd.

PML Ref: 97TF088G

TABLE I Cont'd

ROCK CORE DESCRIPTION
WP 402-97-01
GWP 290-97-01, Site No. 44-379

CORE RECOVERY				
BOREHOLE	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**
379S-3	1	0.22 – 1.22	95	85
	2	1.22 – 2.74	98	75
	3	2.74 – 4.45	100	100
	4	4.45 – 4.75	100	58
	5	4.75 – 5.72	95	87
	6	5.72 – 7.24	100	100
	7	7.24 – 8.76	100	100
	8	8.76 – 10.29	100	93
	9	10.29 – 11.81	100	100
	10	11.81 – 13.34	100	100
	11	13.34 – 14.86	100	72
	12	14.86 – 16.39	100	100
	13	16.39 – 17.91	100	100

CORE DESCRIPTION			
DEPTH (m)	**DESCRIPTION**	**DISCONTINUITIES**	**BLOCK SIZE**
0.22 – 2.78	**GNEISS**, black and white micaceous, weathered brown in places, banded with concentrations of pink feldspar and black hornblende, some biotite mica, moderately coarse grained, (1-4) 2 mm, strong with no signs of penetrative weathering or alteration. Mod. easy to break with hammer along well-developed gneissocity.	3 sets, one formed by breaks along the gneissocity typically 50° to the core axis, and two steeply dipping sets at 5-15° to the core axis. All sets rough, planar. The steeply dipping sets are in most places iron-stained (brown) with slight penetrative weathering; however, the joint surfaces remain hard.	4 cm (2-12)
2.78 – 5.53	black micaceous, homogeneous, weathered, fissile.	Long axes of joint ellipses; angle between joint and core cross-section = dip 280 mm; 80° 50 mm; 20° 58 mm; 36° 54 mm; 29° gneissocity	23 cm
5.53 – 10.20	Alternating black and pink in bands of about 0.7 m, coarse grained.	Core 47 mm dia.	42 cm
10.20 – 10.44	black micaceous, stained along joints, friable and broken		8 cm
10.44 – 17.91	Alternating black and pink in bands of 0.5 to 1.0 m, coarsely crystalline		40 cm

RQD = Rock Quality Designation

Core Recovery data provided by Peto MacCallum Ltd.
Core Description by Franklin Geotechnical Ltd.

PetoMacCallum Ltd.
CONSULTING ENGINEERS

PML Ref: 97TF088G

TABLE II

BOREHOLE DATA FROM GEOTECHNICAL INVESTIGATION FOR THE PROPOSED CP RAIL SHORT DIVERSION

WP 402-97-01
GWP 290-97-01, SITE NO. 44-379

CP Short Diversion
Datum Centre Line

9+940	6.5 LT C/L	D+100
0-100	Blk Si Tps	
100	NFP BR	
9+940	C/L	D
0-250	Blk Si Tps	
250-500	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
500	NFP Bld	
9+940	6.5 RT C/L	D-100
0-200	Blk Si Tps	
200	NFP BR	
9+960	6.5 LT C/L	D+100
0-50	Blk Si Tps	
50-350	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
350	NFP Bld Fr Wat @ 300	
9+960	C/L	D
0-300	Blk Si Tps	
300-600	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
600	NFP Bld Fr Wat @ 300	
9+960	6.5 RT C/L	D+400
0-50	Blk Si Tps	
50	NFP BR	
9+980	6.5 LT C/L	D+100
0-250	Blk Si Tps	
250	NFP BR	
9+980	C/L	D
0-150	Blk Si Tps	
150-350	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
350	NFP Bld/Poss BR	
9+980	6.5 RT C/L	D-1.20
0-150	Blk Si Tps	
150-400	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
400	NFP Bld	
10+000	6.5 LT C/L	D+400
0-150	Blk Si Tps	
150	NFP BR	
10+000	C/L	D
0	NFP BR	
10+000	6.5 RT C/L	D-1.00
0	NFP BR	
10+020	6.5 LT C/L	D+900
0-50	Blk Si Tps	
50-700	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
700	NFP Bld	
10+020	C/L	D
0-100	Blk Si Tps	
100	NFP BR	
10+020	6.5 RT C/L	D-500
0-100	Blk Si Tps	
100-700	Br F To Co Sa W Gr Tr Si Num Cob Num Blds	
700	NFP Bld/Poss BR	
10+040	6.5 LT C/L	D+800
0	NFP BR	
10+040	C/L	D
0-100	Blk Si Tps	
100	NFP BR	
10+040	6.5 RT C/L	D-1.40
0-100	Blk Si Tps	
100	NFP BR	
10+060	6.5 LT C/L	D+1.60
0-100	Blk Si Tps	
100	NFP BR	
10+060	C/L	D
0-400	Blk Si Tps	
400	NFP BR	
10+060	6.5 RT C/L	D-1.20
0-50	Blk Si Tps	
50	NFP BR	

Reference: Geotechnical Investigation, Proposed CP Rail Short Diversion, Parry Sound, January 15, 1999

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LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

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

DYNAMIC PENETRATION RESISTANCE: - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51 mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS. 0.3 m 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. 379S-1

N 5 007 954
E 279 792

PROJECT W. P. 402-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
OUR PROJECT 97TF088G
SITE CPR Rail Grade Separation, Site 44-379
LOCATION Station 15+696 34.1 m Lt.
BORING DATE August 17 & 18, 1998
ENGINEER M. R. Anderson
BORING METHOD NQ Rock Coring
TECHNICIAN M. Rapsey

GROUND ELEVATION 251.04

Depth (m)	Description	Elevation	Sample No.	Type	Run (mm)	Recovery (%)	RQD (%)	Drill Water Return (%)	Groundwater Observations and Remarks
0	BEDROCK : Gneiss		1	RC	1120	100	100	100	
		250	2	RC	560	95	77	100	
1.5		249	3	RC	965	97	71	100	
3.0		248	4	RC	1525	102	65	100	
		247	5	RC	510	100	25	100	
4.5		246	6	RC	1015	98	55	0	100% Drill water return to 5.10m depth, then 0% return.
6.0		245	7	RC	1525	100	100	0	
7.5		244 / 243	8	RC	1525	100	78	0	
9.0		242	9	RC	1525	100	100	0	
10.5		241 / 240	10	RC	1525	100	100	0	
12.0		239	11	RC	1525	100	93	0	
13.5		238 / 237	12	RC	1525	100	85	0	
15.0		236	13	RC	585	100	96	0	
15.42	BOREHOLE TERMINATED AT 15.42m.	235							
16.5									

SOIL PROFILE: DEPTH in METRES; DESCRIPTION; LEGEND; ELEVATION. SAMPLES: 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; WATER CONTENT % 10 20 30. GROUNDWATER OBSERVATIONS AND REMARKS.

NOTES:
Borehole drilled at inclination of 16 degrees to vertical towards northwest.
On September 4, 1998, water at 7.15m.

CHECKED BY: [signature]

PetoMacCallum Ltd.
CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 379S-2

N 5 007 933
E 279 830

PROJECT W. P. 402-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE CP Rail Grade Separation, Site 44-379
LOCATION Station 15+670 C/L
BORING METHOD NQ Rock Coring
BORING DATE August 12 & 13, 1998
OUR PROJECT 97TF088G
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

GROUND ELEVATION 251.06

DEPTH in METRES	DESCRIPTION	ELEVATION	NUMBER	TYPE	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)	GROUNDWATER OBSERVATIONS AND REMARKS
0.23	SAND : Brown silty sand with cobbles								
	BEDROCK : Gneiss	250	1	RC	1040	56	20	00	
		249	2	RC	1550	97	50	100	
		248	3	RC	1525	83*	54	100	* Lost 250mm down hole.
		247 / 246	4	RC	1525	**			** Several attempts required to recover all core due to equipment malfunction.
		245	5	RC	1525	**			
		244	6	RC	610	88	0	100	
		243	7	RC	915	100	100	100	
		242	8	RC	1525	100	97	100	
		241	9	RC	200	100	100	100	
		240	10	RC	1395	91	58	100	
		239	11	RC	1525	100	88	100	
		238 / 237	12	RC	1525	100	100	100	
15.00		236							

Depth scale: 0, 1.5, 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; BLOWS/0.3m N - VALUES

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

continued on next page

NOTES:
Borehole drilled at inclination of 17.5 degrees to vertical towards south.

CHECKED BY: [signature]

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CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 379S-2 (cont'd)

N 5 007 933
E 279 830

PROJECT W. P. 402-97-01, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE CP Rail Grade Separation, Site 44-379
LOCATION Station 15+670 C/L
BORING METHOD NQ Rock Coring
BORING DATE August 12 & 13, 1998
OUR PROJECT 97TF088G
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	SAMPLE NUMBER	SAMPLE TYPE	BLOWS/0.3m N - VALUES	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)	GROUNDWATER OBSERVATIONS AND REMARKS
	GROUND ELEVATION 251.06										
15.0	**BEDROCK** : Gneiss		236	13	RC		1295	100	100	100	
16.38											
16.5	BOREHOLE TERMINATED AT 16.38m.		235								

Chart headings: 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; WATER CONTENT % 10 20 30

NOTES:
Borehole drilled at inclination of 17.5 degrees to vertical towards south.

CHECKED BY: [signature]

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CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 379S-3

N 5 007 926
E 279 891

PROJECT W. P. 402-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE — OUR PROJECT 97TF088G
SITE CPR Rail Grade Separation, Site 44-379 — BORING DATE August 11 & 12, 1998 — ENGINEER M. R. Anderson
LOCATION Station 16+646 35.5 m Rt.
BORING METHOD NQ Rock Coring — TECHNICIAN M. Rapsey

GROUND ELEVATION 253.33

DEPTH in METRES	DESCRIPTION	ELEVATION	SAMPLE NUMBER	TYPE	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)
0.22	TOPSOIL: Dark brown silty sand	253						
	BEDROCK: Gneiss		1	RC	1000	95	85	100
		252						
		251	2	RC	1525	98	75	100
		250						
		249	3	RC	1700	100	100	100
			4	RC	305	100	58	100
		248	5	RC	965	95	87	100
		247	6	RC	1525	100	100	100
		246						
		245	7	RC	1525	100	100	100
		244	8	RC	1525	100	93	100
		243						
		242	9	RC	1525	100	100	100
		241	10	RC	1525	100	100	100
		240						
		239	11	RC	1525	95	72	100
15.00		238						

Depth scale: 0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15.0, 16.5

Column headings: SOIL PROFILE; LEGEND; SAMPLES; 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; WATER CONTENT % 10 20 30; GROUNDWATER OBSERVATIONS AND REMARKS

continued on next page

NOTES:
Borehole drilled at inclination of 17 degrees to vertical towards northeast.
On September 4, 1998, water at 1.60m., (possible drill water).

CHECKED BY: MRA

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CONSULTING ENGINEERS

LOG OF BOREHOLE NO. 379S-3 (cont'd)

N 5 007 926
E 279 891

PROJECT W. P. 402-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE CPR Rail Grade Separation, Site 44-379
LOCATION Station 16+646 35.5 m Rt.
BORING METHOD NQ Rock Coring
BORING DATE August 12 & 13, 1998
OUR PROJECT 97TF088G
ENGINEER M. R. Anderson
TECHNICIAN M. Rapsey

DEPTH in METRES	DESCRIPTION (GROUND ELEVATION 253.33)	ELEVATION	SAMPLE NUMBER	TYPE	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)	GROUNDWATER OBSERVATIONS AND REMARKS
15.0	BEDROCK : Gneiss	238	12	RC	1525	105	100	100	
16.5		237	13	RC	1525	100	100		
		236							
17.91	BOREHOLE TERMINATED AT 17.91m.								
18.0									

Column headings also printed: SOIL PROFILE; LEGEND; 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; WATER CONTENT % 10 20 30.

NOTES:
Borehole drilled at inclination of 17 degrees to vertical toward northeast.
On September 4, 1998, water at 1.60m (possible drill water).

CHECKED BY: MRA

METRIC
DIMENSIONS ARE IN METRES AND OR MILLIMETRES UNLESS OTHERWISE SHOWN

DISTRICT No. 52 CONT No. 2000-0236 WP No 402-97-01	
HIGHWAY 69 – CPR GRADE SEPARATION BOREHOLE LOCATION PLAN	SHEET 186

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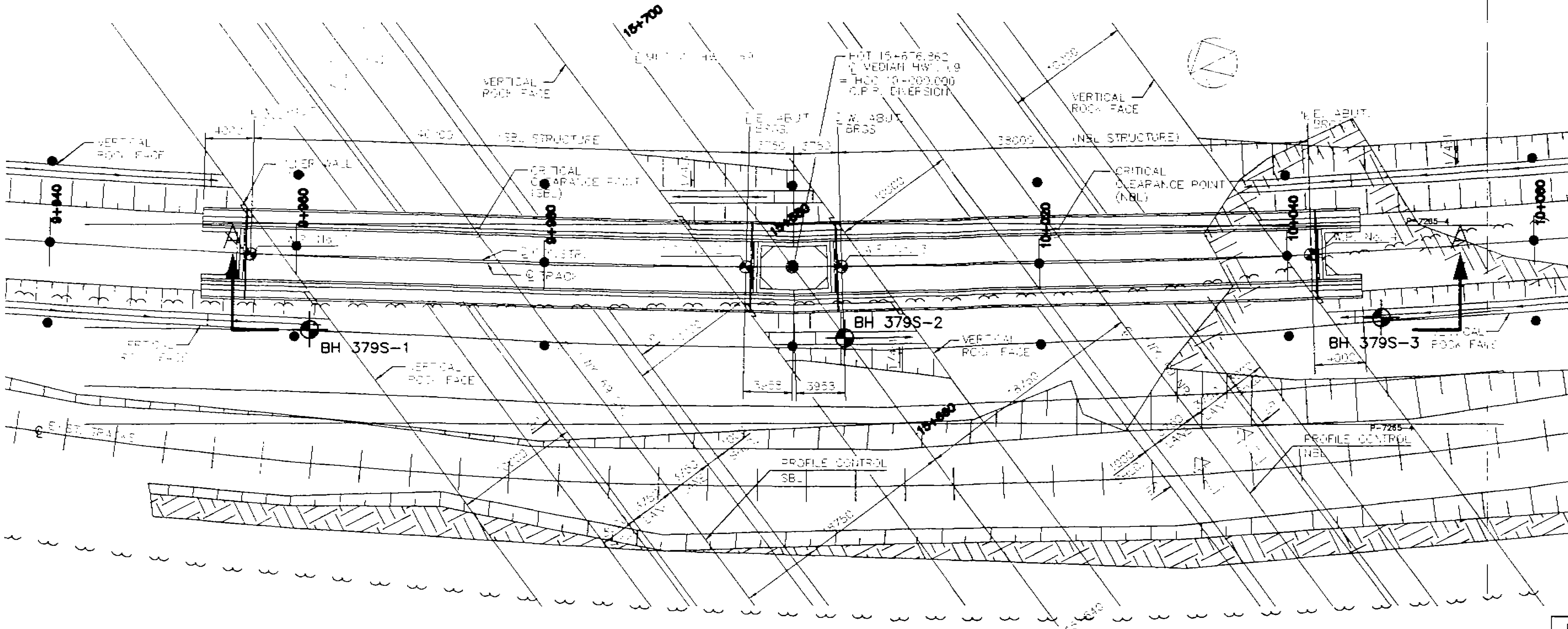
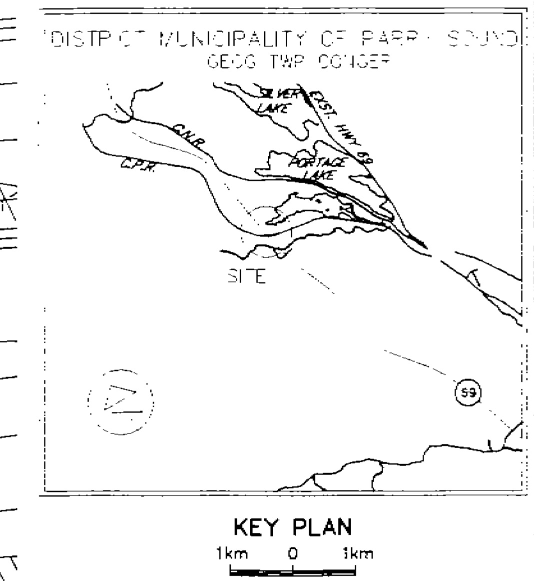


BOREHOLE LOCATION PLAN



KEY PLAN

BOREHOLE	NORTHING	EASTING	ELEVATION
BH 379S-1	N 5 007 954	E 279 792	251.04
BH 379S-2	N 5 007 933	E 279 830	251.06
BH 379S-3	N 5 007 926	E 279 891	253.33

LEGEND

BOREHOLE & ROCK CORE

BOREHOLE FROM GEOTECHNICAL INVESTIGATION FOR PROPOSED CP RAIL SHORT DIVERSION (REFER TO TABLE II)

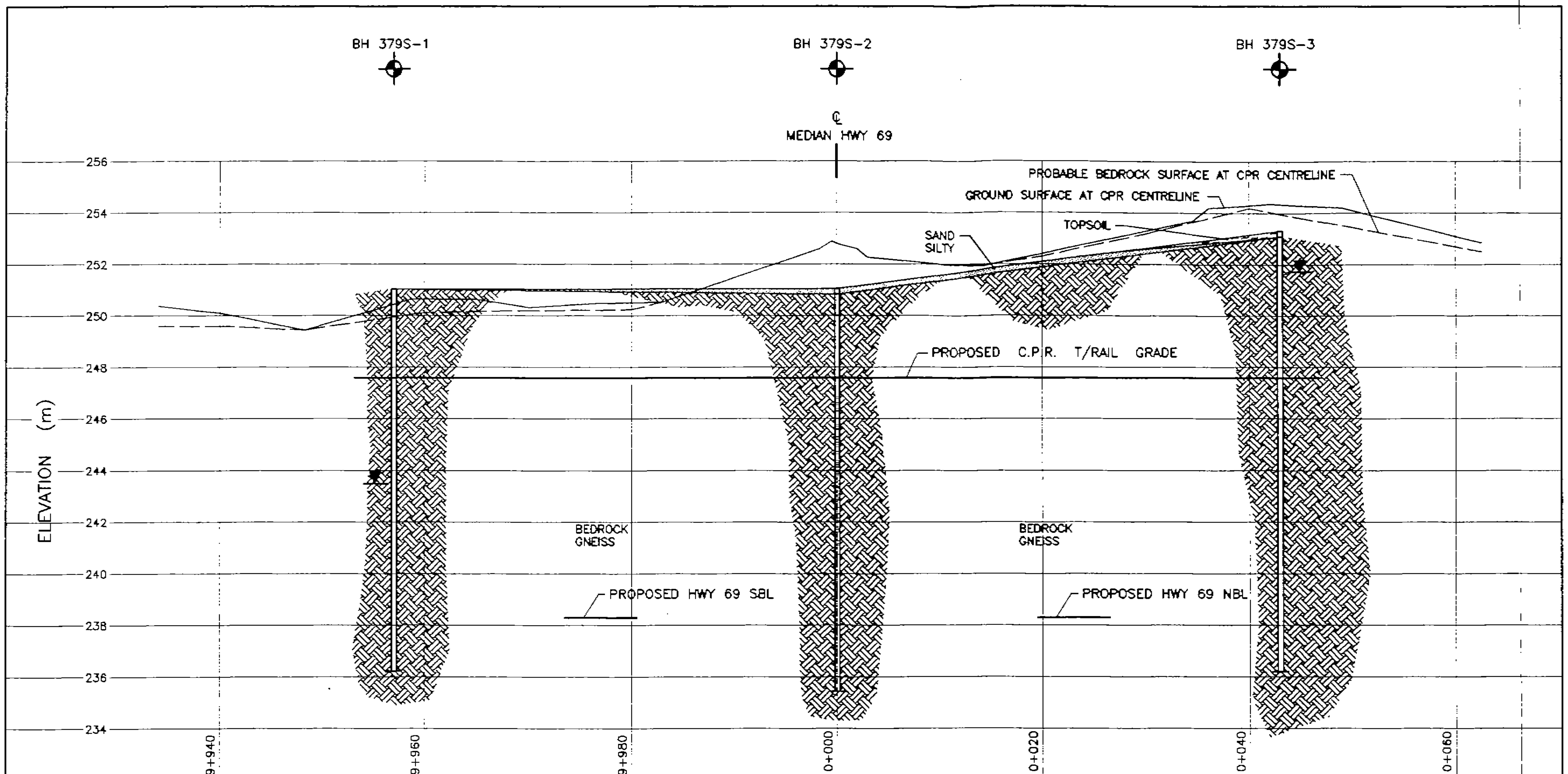
NOTE

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.
3. REFER TO DRAWING 2 FOR SOIL PROFILES AND CROSS SECTIONS.

PROPOSED CROSSING
AT
PROPOSED C.P.R. ALIGNMENT
AND
KING'S HIGHWAY 69
DISTRICT MUNICIPALITY OF PARRY SOUND

LOT 4 GEOG TWP CONGER — SITE 44-379 — CON 6 TWP OF HUMPHREY

	DATE	SCALE	JOB NO.	DRAWING NO.
DRAWN CB CHECKED MRA APPROVED DWK	APRIL 1999	AS SHOWN	97TF088C	1



SECTION A-A

DISTRICT No. 52 CONT No. 2000-0236 WP No 402-97-01	
HIGHWAY 69 - CPR GRADE SEPARATION	SHEET 187
SOIL PROFILES	

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45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C6

LEGEND

BOREHOLE AND ROCK CORE

OBSERVED WATER LEVEL
(POSSIBLE DRILL WATER, 17 TO 23 DAYS AFTER CORING)

NOTE

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

PROPOSED CROSSING AT
AT
PROPOSED C.P.R. ALIGNMENT
AND
KING'S HIGHWAY 69
DISTRICT MUNICIPALITY OF PARRY SOUND
LOT 4 GEOG TWP CONGER — SITE 44-379 — CON 6 TWP OF HUMPHREY

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

		DATE	SCALE	JOB NO.	DRAWING NO.
DRAWN	CB	APRIL 1999	AS SHOWN	97TF088G	2
CHECKED	MRA				
APPROVED	DWK				

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FOUNDATION DESIGN REPORT
FOR
CP RAIL GRADE SEPARATION
W.P. 402-97-01
G.W.P. 290-97-00, SITE 44-379
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

Job No. 97TF088G
Geocres No. Not Assigned

July, 2000

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TABLE OF CONTENTS

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FOUNDATION DESIGN REPORT
For
CP Rail Grade Separation
W.P. 402-97-01
G.W.P. 290-97-00, Site 44-379
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 CP Rail grade separation structure over the proposed Highway 69 (Station 15+677 Highway 69 Chainage).

The General Arrangement drawing for this structure, dated February, 1999, indicates the underpass will consist of two single span bridges. The rail grade will be near elevation 247.6, some 3.5 to 6.0 m below existing grade. The proposed four lane Highway 69 will be constructed near elevation 238.3, 13 to 15 m below existing grade and 10 m below the proposed rail grade. An approximate 10 m wide rock ridge will remain unexcavated along the centre median. The abutment foundations will be constructed near the road grade level.

The subsurface stratigraphy revealed at the bridge site comprised a discontinuous veneer of topsoil or sand overlying bedrock at depths of 0 to 230 mm.

FOUNDATIONS

Based on the borehole information, it is considered that the structures may be supported on conventional spread footings founded on bedrock. Foundations bearing on sound bedrock at or below elevation 247.6 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 extreme east and west abutment footings of the two bridges should be founded below a line inclined upwards at 1:2 (H:V) from the toe of the Highway 69 cut and set back 2 m from the face of the rock. Footings for the centre two abutment footings may be constructed on the rock "ridge" along the median provided they are founded below a line inclined upwards at 1:1 from the toe of the excavation and the edge of footing is at least 2.0 m from the rock excavation face.

Excavation of the rock to the proposed founding level should be carried out in a manner which provides a level founding surface. Mass concrete could be placed to level minor variations in the founding surface.

It is important that blasting/excavation of the rock in the vicinity of the structure is controlled to minimize disturbance of the rock surface on which footings will bear. Recommendations for rock excavation are presented in a subsequent section of this report.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the bedrock. An unfactored friction factor of 0.6 is recommended for footings on bedrock. A value of 0.7 may be used for a roughened bedrock surface (asperity height of at least 25 mm) created by mechanical means/during rock excavation.

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. Eliminating the footing and providing overturning resistance using dowels into rock could also be considered.

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

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

If backfill is placed behind the abutment walls, they 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	42
Unit Weight (kN/m^3)	22.8	21.2	18.0
Active Earth Pressure Coefficient (K_a)	0.27	0.31	0.20
At Rest Earth Pressure Coefficient (K_o)	0.43	0.47	0.33
Passive Earth Pressure Coefficient (K_p)	3.69	3.25	5.04

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 the abutment concrete is poured neat against the cut rock face, lateral pressures exerted on the abutment wall will be negligible provided measures are implemented to prevent the build-up of hydrostatic pressure behind the concrete. This can be accomplished by placing a drainage membrane on the rock prior to concreting and weep holes at the base of the wall to allow the water to drain.

APPROACH EMBANKMENT

Approach fill and embankment construction is not anticipated since the railway diversion will be constructed in a rock cut.

EXCAVATION AND GROUNDWATER CONTROL

Excavation for construction of Highway 69, the rail diversion and structures is expected to be carried out within bedrock. Considering the type of bedrock and geometry of cut, it is anticipated that blasting will be required to excavate the rock.

The blasting and excavation operations should be conducted in accordance with Ontario Provincial standard procedures with operational constraints imposed to minimize the potential for damage to the existing CP Rail track and the grade separation structure.

The following operational constraints should be imposed:

i) Existing Railway Track

blast induced vibration at the edge of ballast nearest the blast must not exceed 100 mm/s.

ii) New Structure

blast induced vibration of the rock adjacent to structure foundations must not exceed the value computed from the following equation

$ppv < 0.4H + 10 < 100$
ppv = peak particle velocity (mm/s)
H = age of concrete (hours)

iii) Blast Attenuation Coefficients

The blast attenuation coefficients must be determined by monitoring at least four representative blasts before blasting within 200 m of the track in order to be able to compute/ establish the distance L at which normal blasts may cause vibrations at the track to exceed 100 mm/s.

iv) Blast Monitoring

The blast induced vibrations at the railway track must be monitored during all blasts within a distance of 2 L of the track.

v) Controlled Blasting

Terminate normal blasting when the blast induced vibrations at the track exceed 100 mm/s and implement reduced scale blasting procedures and/or mechanical excavation procedures to excavate the rock.

The specifications should also call for the contractor to protect the rail from damage due to "flying" blast rock (e.g. blasting mats) and any damage repaired immediately.

It is the responsibility of the contractor to schedule his work and design the blasts to ensure that the work is conducted in accordance with the specification without exceeding the permitted vibration levels. A blasting consultant with at least five years full-time working experience providing blast consultant services should be retained to evaluate trial blast procedures, recommend blast design, and monitor blast vibration/attenuation.

It is anticipated that monitoring of blast induced vibrations at the track will be required when the excavation face is within 50 to 75 m of the track and reduced scale blasting will be required within 35 m.

Rock excavation in the vicinity of the structure must be controlled to minimize disturbance of the rock surface on which footings will bear. In this regard, methods such as line-drilling of unloaded vertical holes, closer blasthole spacing, reduced scale blasting, and/or rock reinforcement with dowels prior to blasting should be considered in foundation areas.

Water was observed in two core holes at depths of 7.1 and 1.6 m about three weeks after drilling. It is not clear whether the water comprised groundwater or residual drillwater. Flow through the rock mass occurs only along joints. Seepage or surface water which enters the excavation should be readily handled by conventional sump pumping techniques.

Groundwater levels are subject to seasonal fluctuations and rainfall patterns. Some ice build-up on the rock face should be expected during the winter months.

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

CLOSURE

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

REGISTERED PROFESSIONAL ENGINEER
D. W. KERR
PROVINCE OF ONTARIO

Yours very truly

Peto MacCallum Ltd.

Dennis W. Kerr, M.Eng., P.Eng.
Manager Geotechnical and
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REGISTERED PROFESSIONAL ENGINEER
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