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GEOCRES No. 39B-59

DIST. 8 REGION

W.P. No. 70-79-03

CONT. No. 82-90

W. O. No.

STR. SITE No. 16-112

HWY. No. 2

LOCATION Macilhennys  
Michael Henry Creek Bridge

No of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

# OVERSIZE DRAWING

ENGINEERING MATERIALS OFFICE  
PAVEMENT & FOUNDATION DESIGN SECTION

WP 70-79-03

DIST 8

HWY 2

STR SITE 16-112

~~Michael Henry Creek Bridge~~

*Macilhennys*

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## FOUNDATION INVESTIGATION REPORT

For

Michael Henry Creek Bridge  
W.P. 70-79-03, Site 16-112  
Highway 2, District 8, Kingston

### INTRODUCTION

This report contains the results of a foundation investigation for the above site. Field work consisted of 4 sampled boreholes advanced during the period November 20th to December 1st, 1980. A truck mounted CME 75 auger operating from the road surface was employed. The 2 boreholes through the east approach were advanced with hollow stem augers while 'N' and 'B' size casing were employed for the boreholes through the structure deck. Bedrock was proven in the 2 boreholes through the deck by recovering BXL size rock cores.

### SITE DESCRIPTION

The site is located where Highway 2 crosses Michael Henry Creek some 10 km west of Brockville. The surrounding area with its shallow overburden and frequent granite outcrops is typical of this physiographic region which is referred to as Leeds Knobs and Flats.

Michael Henry Creek at the time of the investigation was 7 to 15 metres in width and up to 2 metres in depth. It flows in a narrow valley bordered by frequent rock outcrops exhibiting very steep slopes which, it may be assumed, are typical of the subsurface rock slopes.

The existing structure is a 14 metre single span bridge supported on full height abutments. The opening is skewed at 45 degrees. It was constructed in 1921 and reinforced with extra steel beams in 1968 due to the deterioration of the original girders. The river channel in the area is lined with rip rap.

### SUBSURFACE CONDITIONS

#### General

Subsoil beneath the rip rap lined stream channel consists of about 4.5 metres of stiff to hard silty clay overlying a 1 to 4 metre

thick granular layer consisting of sand, gravel, cobbles and boulders. This layer in turn overlies sound bedrock.

Reference should be made to Drawing 707903-A which shows the location and elevation of all borings as well as an inferred subsoil stratigraphy. Record of Borehole sheets which show the boundaries between soil types as well as a record of all field and laboratory tests performed are contained in the report Appendix. A more detailed subsoil description follows.

#### Silty Clay

The stream channel is underlain by a layer of silty clay having a thickness of approximately 4.5 metres. It's consistency is stiff to hard based on the undrained shear strength measured by field vane which ranges from 50 to in excess of 100 kPa. Results of Atterberg Limit testing are shown in Figure 1.

#### Sand, Gravel, Cobbles and Boulders

The silty clay deposit is underlain by a 1 to 4 metre thick granular layer consisting of sand, gravel, cobbles and boulders. Diamond coring was employed to penetrate this layer because of the presence of cobbles and boulders. As a result, only 1 Standard Penetration Test, which gave an 'N' value of 19, was carried out. The relative density is assessed as compact to dense.

#### Approach Fill

The east approach fill consists of silty sand to sand and gravel. Refusal to augering was met at a relatively shallow depth on what appeared to be rock fill or boulders.

#### Bedrock

Sound metagabro bedrock was encountered at elevation 67 and 70 some 7 to 10 metres below the river surface. Bedrock geology maps show frequent dykes of similar material in the area. It is therefore assumed that a gabro dyke was intruded into the surrounding granite bedrock. Since the dyke was softer than the surrounding rock it weathered more rapidly thereby forming the channel of Michael Henry Creek. Bedrock outcrops in the area frequently have very steeply sloping surfaces. It may be assumed that subsurface slopes are similar with locally very steep sections.

## PROPOSAL AND RECOMMENDATIONS

### Proposal

It is proposed to construct a new structure of approximately equal hydrologic capacity on the same alignment as the existing. Recommendations are also to be provided for a bailey bridge on the same alignment to carry traffic above the construction of the replacement structure.

### RECOMMENDATIONS

#### Concrete Box

The replacement structure may be a concrete box founded at elevation 75 or below. It should be placed on a pad of compacted granular 'A' with a minimum thickness of 0.3 metres. Assuming an allowable settlement of 25 mm the maximum serviceability limit state loading is 70 kPa and the ultimate limit state loading is 200 kPa.

#### Steel H-Piles

Alternatively, the replacement structure may be supported on steel H-piles. To minimize damage due to the bouldery layer overlying the bedrock a 310 x 110 section with the tips fitted with standard flange plates should be employed. A factored capacity of 1100 kN is recommended with the east abutment piles driven to bedrock at elevation 70 and the west abutment driven in accordance with SS 3-11 to an ultimate resistance of 2750 kN per pile. It is estimated this capacity will be achieved at approximate elevation 68. Due to the steeply sloping nature of the bedrock in the area further field work should be carried out after preliminary design to determine the subsurface conditions at the actual abutment locations if this option is employed.

#### Earth Pressure

Earth pressure should be calculated in accordance with section 6.6.1.2 of the code. Rigid structures should be designed for the at rest condition with ultimate limit state design values of 10 kPa/m. Where structures are free to deflect under load, active earth pressure applies and values of 8 kPa/m for ultimate limit state and 6.5 kPa/m for serviceability limit state conditions should be used.

### Unwatering

Unwatering of excavations may be carried out inside a clay dyke by pumping from sumps.

### Approach Embankments

No stability problems will occur if 2 horizontal to 1 vertical slopes are employed.

### Bailey Bridge Support

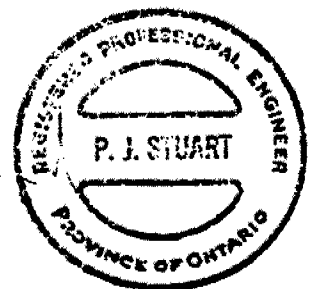
When the existing abutments are removed a forward slope of 1.5 horizontal to 1 vertical is acceptable provided it is protected from scour and erosion by a layer of rip rap. If the front edge of the bailey bridge footing is 3 metres horizontally from the top of the 1.5:1 slope a serviceability limit state loading of 100 kPa and an ultimate limit state loading of 150 kPa may be used. If the horizontal distance is reduced to 1.5 metres the loading should be reduced to 75 kPa for the serviceability limit state loading and 120 kPa for ultimate limit state loading.

Consideration should be given to shifting the new structure so that one of the existing abutments can be used for support of the bailey bridge.

### Frost Protection

The base of all footings or pile caps should be protected from frost action by a minimum of 1.8 metres of cover.

*P.J. Stuart*  
P.J. Stuart, P. Eng.  
Foundations Engineer



*M. Devata*  
M. Devata, P. Eng.  
Senior Foundations Engineer



Ministry of  
Transportation and  
Communications

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No. 1

W P 70-79-03 LOCATION Sta. 10+493, 2.3 m Lt. E ORIGINATED BY R.B.  
DIST 8 HWY 2 BOREHOLE TYPE Hollow Stem Auger COMPILED BY PJS  
DATUM Geodetic DATE 1980 11 20 CHECKED BY \_\_\_\_\_

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			'N' VALUES	20						40
83.0	Ground Level													
0.0	Sand and Gravel Some Silt Compact to Loose													
			1	SS	12									
			2	SS	8									
79.8														
3.2	Refusal to Auger End of Borehole													

+3, x5: Numbers refer to  
Sensitivity

20  
15 → 5 (%) STRAIN AT FAILURE  
10



RECORD OF BOREHOLE No 2

W P 70-79-03 LOCATION <sup>20+454.2</sup> Sta. 10+484.5, 1.5 m Lt. E ORIGINATED BY RB  
DIST 8 HWY 2 BOREHOLE TYPE Hollow Stem Auger COMPILED BY PJS  
DATUM Geodetic DATE 1980 11 21 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20   40   60   80   100					W <sub>p</sub>	W	W <sub>L</sub>		
								SHEAR STRENGTH									
83.0	Ground Level																
0.0	Silty Sand Loose						82										
81.0			1	SS	7												
2.0	Refusal to Augers End of Borehole						80										

+3, x5: Numbers refer to  
Sensitivity

20  
15-5 (%) STRAIN AT FAILURE  
10



RECORD OF BOREHOLE No 3

W.P. 70-79-03 LOCATION Station 10+476, 1.5 m Lt. E ORIGINATED BY RB  
DIST 8 HWY 2 BOREHOLE TYPE B Casing COMPILED BY PJS  
DATUM Geodetic DATE 1980 11 21 CHECKED BY \_\_\_\_\_

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
82.9	Deck Level																
82.5	Concrete																
0.4							82										
							80										
							78										
77.1	Creek Level						76										
5.8																	
75.9	Creek Bottom																
75.3	Rock Rip Rap		1	BXL RC	10%												
7.6																	
	Silty Clay Stiff to Hard		2	SS	5		74										
			3	TW	PH		72										
			4	TW	PH												
71.0																	
11.9	Sand and Gravel Compact		5	SS	19		70										
70.0																	
12.9	Sound Metagabro Bedrock		6	BXL RC	95% REC												
68.4																	
14.5	End of Borehole																

+3, x5 : Numbers refer to  
Sensitivity

20  
15  
10

5 (%) STRAIN AT FAILURE



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HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

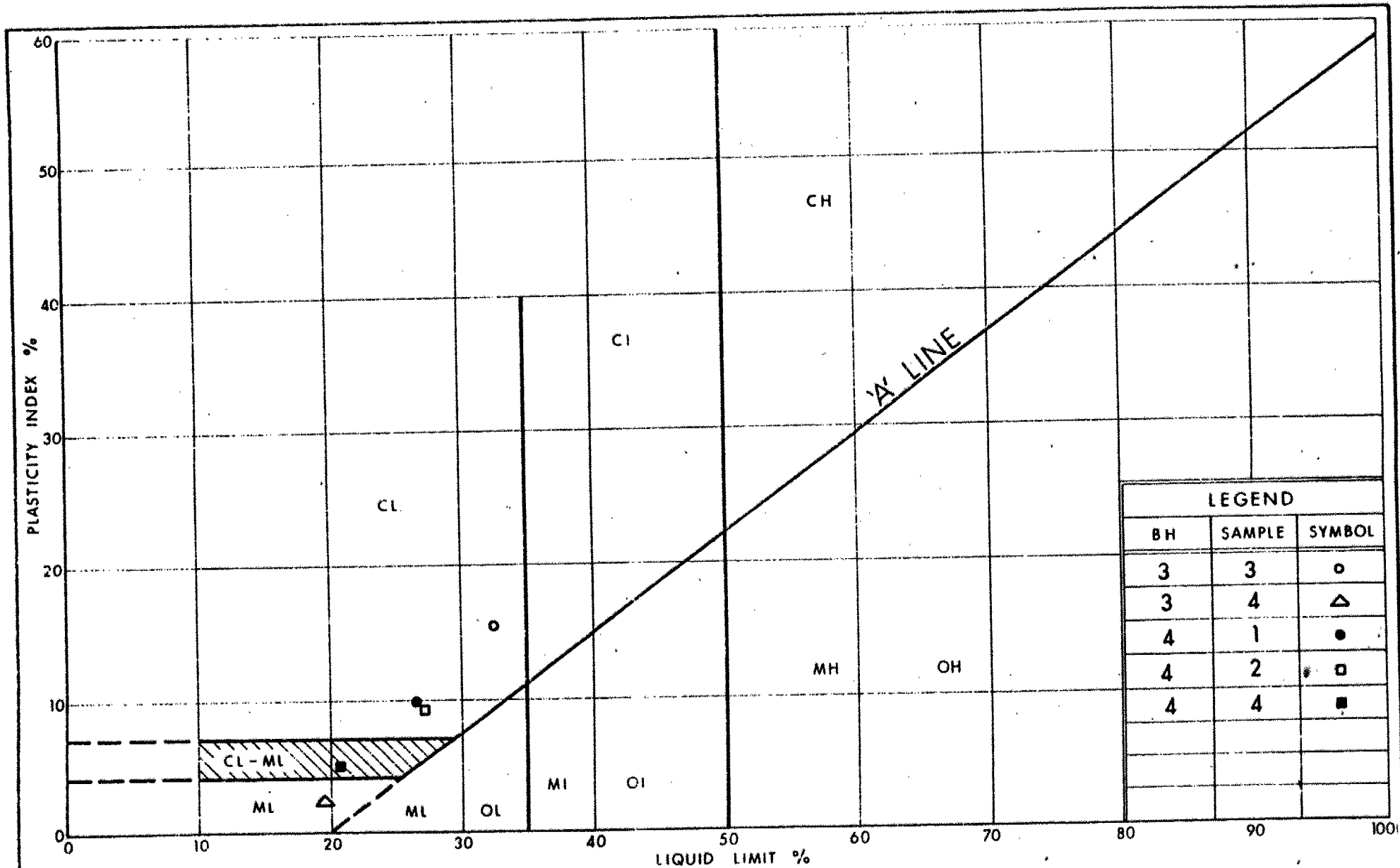
RECORD OF BOREHOLE No 4

W P 70-79-03 LOCATION Sta. 10+467.4, 2.3 m R.L. E ORIGINATED BY RB  
DIST 8 HWY 2 BOREHOLE TYPE H Casing COMPILED BY PJS  
DATUM Geodetic DATE 1980 12 01 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100		W <sub>p</sub>	W	W <sub>L</sub>		
								SHEAR STRENGTH kPa						
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE					
82.9	Deck Level							50	100			10 20 30		
82.4	Concrete													
0.5							82							
							80							
							78							
77.2	Water Level													
5.7														
76.2	Creek Bottom						76							
6.7	Rock Rip Rap													
75.5														
7.4	Silty Clay		1	SS	15									
	Stiff to Hard						74							
			2	TW	PH									
			3	TW	PH									
			4	SS	17		72							
			5	TW	PH									
71.0							70							
11.9	Sand, Gravel		6	BXL	25% REC									
	Cobbles and Boulders													
	Compact to Dense		7	BXL	25% REC		68							
66.9														
16.0	Sound Metagabro		8	BXL	95% REC		66							
65.6	Bedrock													
17.3	End of Borehole													

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10



# PLASTICITY CHART SILTY CLAY



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FIG No. 1

W P 70-79-03

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.3kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 473 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_u$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 30	30 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$C_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
M	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{v0}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_r$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_r$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

METRIC

CONT No  
WP No 70-79-03

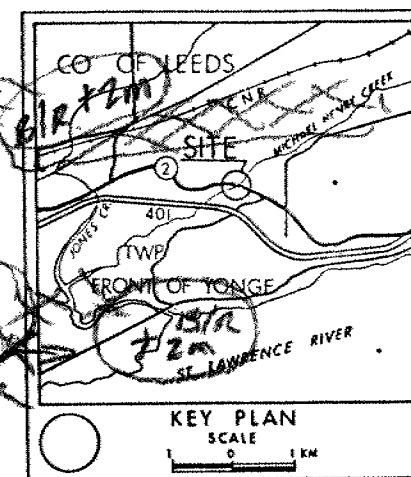
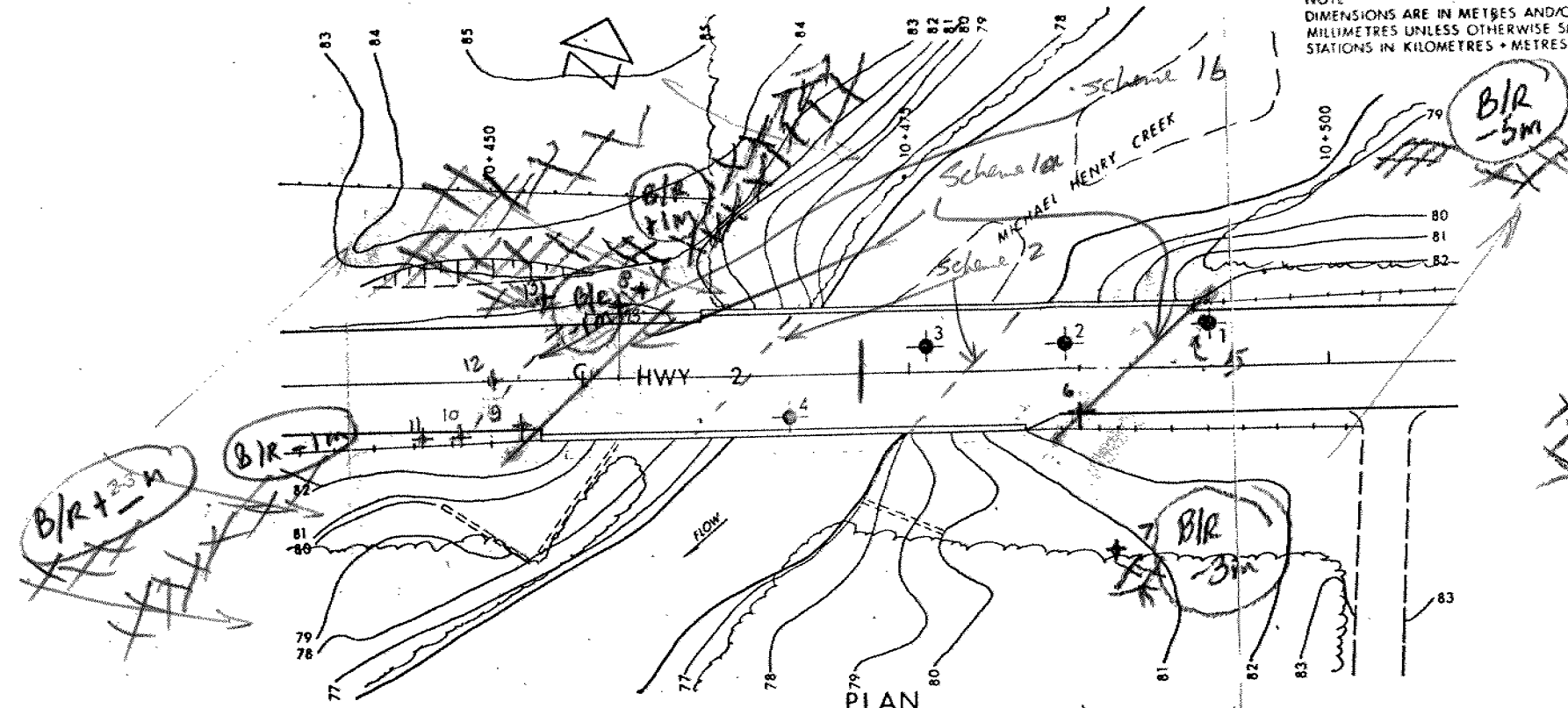
MICHAEL HENRY CR. STRUCTURE  
AT HWY 2

BORE HOLE LOCATIONS & SOIL STRATA

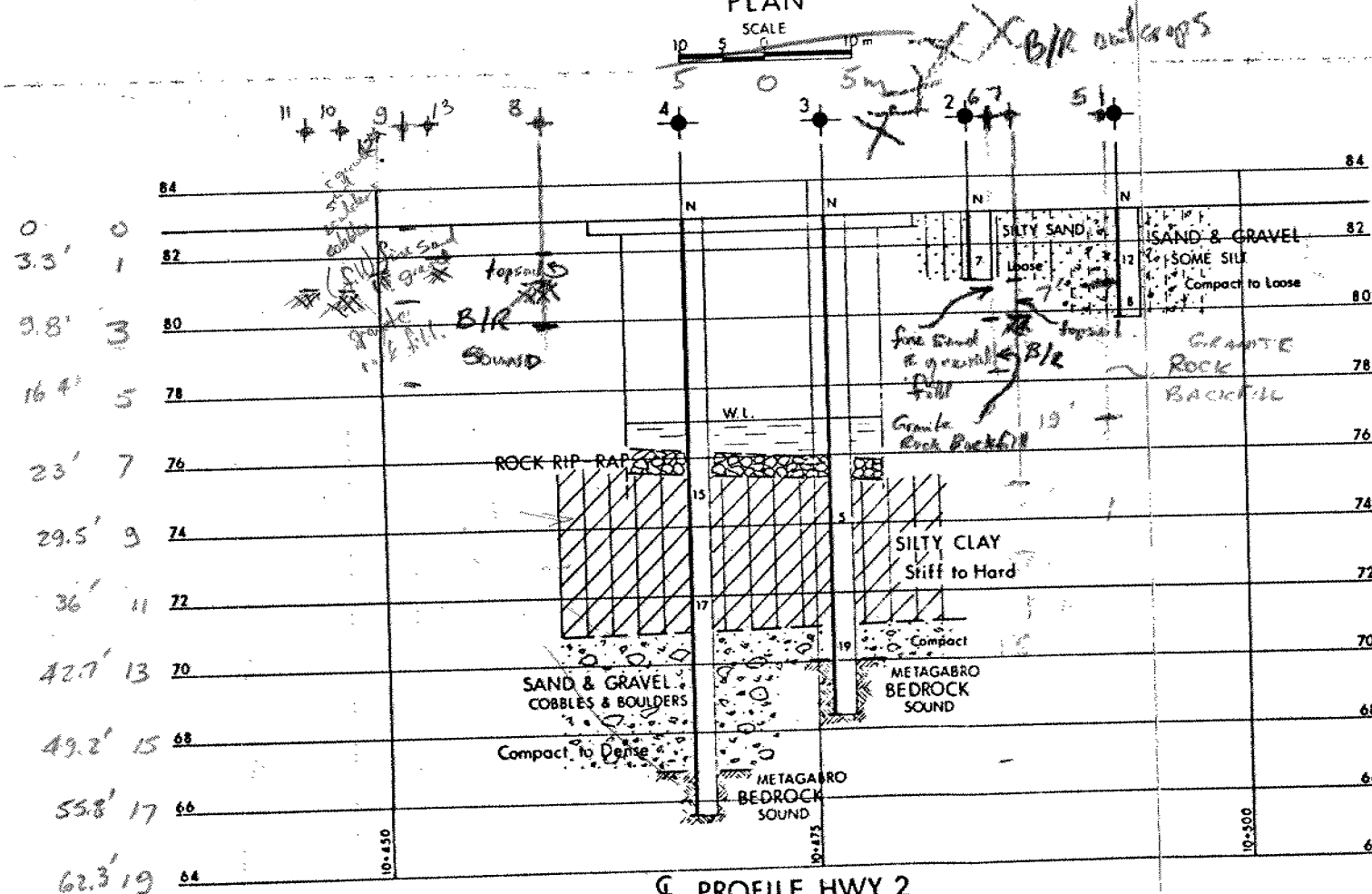


SHEET

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



- LEGEND**
- ◆ Bore Hole
  - ⊕ Dynamic Cone Penetration Test (Cone)
  - ⊙ Bore Hole & Cone
  - N Blows/0.3m (Std Pen Test, 475 J/blow)
  - CONE Blows/0.3m (60° Cone, 475 J/blow)
  - W.L. at time of investigation 80 11 21



No	ELEVATION	STATION	OFFSET
1	83.0	10+493.0	2.3 LT.
2	83.0	10+484.5	1.5 LT.
3	82.9	10+476.0	1.5 LT.
4	82.9	10+467.4	2.3 RT.

**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 318-59  
HWY No 2  
SUBMDP 5 CHECKED DATE 01 02 26 SITE 16-112  
DRAWNOL J. CHECKED APPROVED DWG 707903-A

# memorandum



To: Mr. T.C. Kingsland  
Head, Structural Section  
Eastern Region

Date: 81 07 07

From: Pavement & Foundation Design Section  
Room 313, Central Building  
Downsview

Re: Maclihennys Creek Bridge  
W.P. 70-79-03, Site 16-112  
Highway 2, District 8, Kingston

This memorandum outlines the findings of a further soils investigation which was carried out at the above mentioned site and provides recommendations to the structure foundations and the related earthworks. A foundation report has already been issued, however, due to change in design geometry an additional investigation was required. This memorandum should be read in conjunction with the previously issued foundation report. Appended please find the additional borehole logsheets and a revised drawing showing the updated subsurface information. No further report will be issued for this project except for the report and drawing required for contract purposes.

The additional fieldwork consisted of a combination of 9 sampled boreholes and probeholes carried out during the period of May 28th to June 3rd, 1981. Hollow stem augers were advanced to refusal at which time BX size casing, and in one particular case NX size casing, was drilled ahead using a muskeg mounted CME 55 auger machine. In some locations rock core samples were also obtained using BXL and NXL core barrels.

## Subsurface Conditions

At this site the bedrock is exposed and it is very undulating and in places it is dipping very steeply. In the immediate vicinity of the structure the approaches are constructed with rockfill consisting of cobbles and boulders. In view of the presence of the rockfill, rock drilling techniques using NXL and BXL core barrels were employed, however, the lower boundary of the rockfill was not established.

cont'd...../2

At certain locations the rockfill is overlain with granular fill material ranging in depths from 1.8 metres to 3.3 metres. This material can be described as loose to dense, fine silty sand with gravel, with 'N' values ranging from 5 to 41 blows/0.3 m. It can be inferred that this material is part of the road base course or granular backfill to the existing structure.

At the east creek bank both on the north and south sides of the road the casing was drilled to refusal. Refusal was met on the hard angular fragments of the rockfill.

Bedrock could be seen from the surface approximately 5 metres below the road surface just north of the east bank and possible bedrock was located with a probehole approximately 3 metres below the road surface just south of the east bank.

At the west creek bank two different subsurface conditions were found. On the north side of the west bank, sound bedrock was encountered 0.2 metres below ground elevation; on the south side of the west bank, a surficial fill material consisting of compact silty sand with occasional stiff silty clay zones, with a typical 'N' value of 13 blows/0.3 m was encountered. This upper fill material is underlain by a cobble and boulder rockfill.

At approximately 10.5 metres west of the west creek bank a 0.4 to 1.8 metre sand and gravel base course material intermingled with some cobbles and boulders was encountered. By obtaining BXL core barrels at B.H. #10 and by lowering probeholes #11, 12 and 13 the rock surface was delineated and was found to vary from 1.8 m below the road surface at B.H. #10 to 0.4 m below the road surface at B.H. #13.

#### Discussion and Recommendations

There are presently two schemes being considered for the Macilhennys Creek Bridge.

Scheme 1 a)-a beam type structure supported on spill-through type abutments on spread footings with a clear span of about 32 metres.

1 b)-alternatively a similar structure with a clear span of about 42.5 metres (i.e. an additional 10.5 metres added on the west side only).

Scheme 2 - a rigid frame bridge founded on piles with a clear span of about 18.5 metres.

Scheme 1a) - West Abutment

This alternative requires that the footing be built partially on rockfill and partially on bedrock. This type of subsurface condition could cause differential settlements resulting in the cracking of the footing and possibly the abutment.

The following recommendations are to be carried out if this option is chosen:

The abutment footing is to be located on rockfill with a maximum loading of 190 kPa. Where the footing infringes on the bedrock the bedrock is to be removed to a depth of 1 metre below the base of the footing and then back-filled with well compacted rockfill material with a maximum gradation of 350 mm. Where the footing infringes on the rockfill, a 450 mm depth of well compacted crushed rock with a maximum gradation of 75 mm should be placed over the rockfill prior to pouring the concrete footing. The above is required to prevent the migration of the concrete through the large rockfill voids. Figure A illustrates the aforementioned.

Scheme 1 b)- West Abutment

To avoid building the footings partially on rockfill and partially on bedrock the west abutment footing could be shifted westerly to Sta. 10+445.5 where the footings can be founded directly on bedrock.

The following recommendations are to be carried out if this option is chosen:

The abutment footing is to be located on bedrock with a maximum loading of 2.4 MPa. The bedrock is to be removed to a required footing depth with mass concrete being used to level the footing area.

Scheme 1 a) & 1 b) - East Abutment

As the proposed east abutment is at Station 10+488.0 it may be founded on the existing rockfill. For footing placement and loading details refer to recommendations as outlined in 'Scheme 1 a) - West Abutment' footing on rockfill.

## TREATMENT REQUIRED AT WEST ABUTMENT - SCHEME 1a

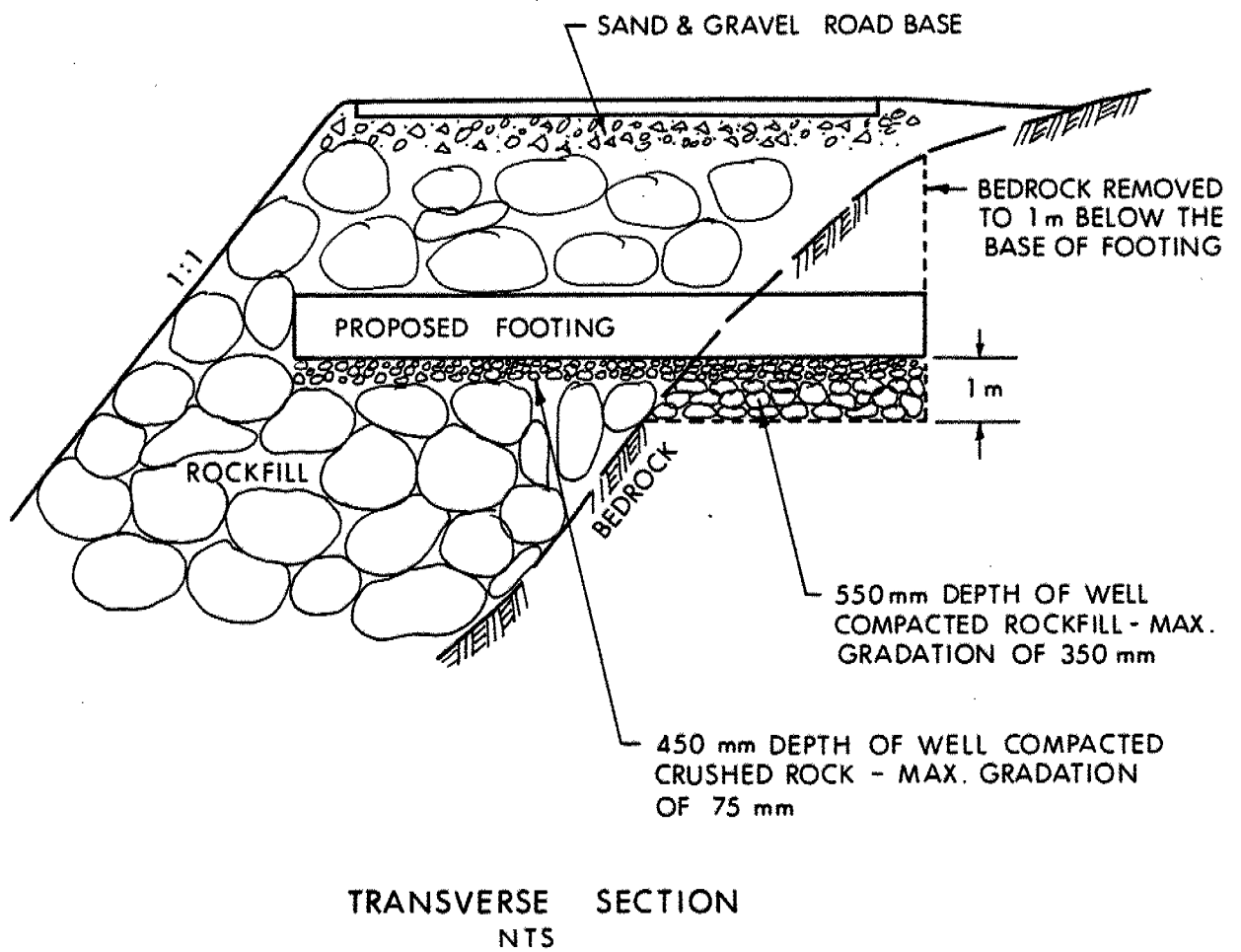


FIGURE A

## Scheme 2

If this option is chosen the following should be adhered to:

The structure should be founded on steel 'H' piles equipped with reinforced flange plates and 'Oslo' tips to insure penetration of the cobble and boulder stratum and proper seating of the piles on sloping bedrock.

In consideration that the steel 'H' piles could meet refusal in the overlying cobbles and boulders, the piles should be designed for an allowable compressive loading of 700 kN for a 310 HP 110 pile. In accordance with OHBDC requirements the capacity at S.L.S. Type II is 700 kN and the factored capacity at U.L.S. is 1600 kN.

The average estimated tip elevations are as follows:

West Abutment - pile tip elevation approximately 68.

East Abutment - pile tip elevation approximately 70.

In order to facilitate pile penetration through the rock fill, it is recommended that in the area of the piles all the cobble and boulder rockfill and the existing structure (i.e., the retaining wall and footings) be removed, backfilled with acceptable well compacted earth material having a maximum gradation of 75 mm.

In order to insure the stability of fills the following should be adhered to:

Rockfill slopes to be constructed with  $1\frac{1}{2}$ :1 slopes in the transverse direction and  $1\frac{1}{2}$ :1 slopes in the forward direction to ensure adequate protection against the scouring action of the creek.

Earthfill slopes to be constructed with 2:1 slopes with an adequate rip rap protection scheme to protect against river scour action.

The base of all footings or pile caps should be protected from frost action by a minimum of 1.8 metres of cover.

In view of the presence of the cobbles and boulders and the sloping nature of the bedrock Scheme 2, a rigid frame bridge founded on piles should not be considered

to be a viable alternative. In our opinion, Scheme 1, a simply supported structure on spread footings, should be implemented. We would like to further point out to you that Scheme 1 b), the structure with the longer span, will effectively minimize the removal of the existing materials, thereby shortening construction complications and time, and resulting in a possible reduction in final costs.

We trust that the information provided is sufficient in scope for your requirements. Please feel free to contact this Section if further discussion is required.

*Nick Stea.*

N. Stea  
Project Foundations Engineer

For: M. Devata  
Senior Foundations Engineer

NS:ea

cc: W.E. Blum  
J.W. Reid  
R.W. Oddson (2)  
K.G. Bassi  
B.J. Giroux  
R. Hore



Ministry of  
Transportation and  
Communications

# RECORD OF BOREHOLE No 5

METRIC

W P 70-79-03 LOCATION Sta. 10 + 492.8; 0/S 2.5 m LT & Hwy. 2 ORIGINATED BY NS  
DIST 8 HWY 2 BOREHOLE TYPE BX Casing, BXL Rock Core COMPILED BY NS  
DATUM Geodetic DATE 81 05 28 to 29 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80					
83.0	Ground Level															
0.0	Fine Silty Sand With Gravel		1	SS	5											
81.2	Loose Fill		2	SS	69											
1.8	Gravel, Cobbles and Boulders Rock Fill		3	BXL												
			4	BXL												
			5	BXL												
			6	BXL												
			7	BXL												
77.2																
5.8	Refusal to Drill End of Borehole															

+3, x5: Numbers refer to Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10



## METRIC

W P 70-79-03 LOCATION Sta. 10 + 485.3, O/S 2.4 m RT of Hwy. 2 ORIGINATED BY NS  
DIST 8 HWY 2 BOREHOLE TYPE NX and BX Casing, NXL Rock Core, BXL Rock Core COMPILED BY NS  
DATUM Geodetic DATE 81 06 01 to 02 CHECKED BY NS

[illegible]

+3, x5 : Numbers refer to Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 7

METRIC

W P 70-79-03 LOCATION Sta. 40+487.8; 0/S 11.3 m RT of Hwy. 2 ORIGINATED BY NS  
DIST 8 HWY 2 BOREHOLE TYPE Hollow Stem Continuous Flight Augers COMPILED BY NS  
DATUM Geodetic DATE 81 06 02 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
80.6	Ground Level																
80.8	Topsoil																
0.6	Refusal to Auger Probable Bedrock End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 8

METRIC

W P 70-79-03 LOCATION <sup>20+429.4</sup> Sta. 10+459-6 O/S, 5.2 m LT @ Hwy. 2 ORIGINATED BY NS  
 DIST 8 HWY 2 BOREHOLE TYPE BX Casing, BXL Rock Core COMPILED BY NS  
 DATUM Geodetic DATE 81 06 02 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80	100	W <sub>p</sub>	W		
83.0	Ground Level															
0.0	Bedrock Good Quality		1	BXL RC	33% REC											
	Hard		2	BXL RC	100% REC											
79.9																
3.1	End of Borehole															

OFFICE REPORT ON SOIL EXPLORATION



Ministry of  
Transportation and  
Communications  
Ontario

# RECORD OF BOREHOLE No 9

METRIC

W P 70-79-03 LOCATION Sta. 10+421.5 20+421.5 O/S 2.8 m RT of Hwy. 2 ORIGINATED BY NS  
DIST 8 HWY 2 BOREHOLE TYPE BX Casing, EXL Rock Core COMPILED BY NS  
DATUM Geodetic DATE 81 06 03 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
83.0	Ground Elevation																
0.0	Compact Silty Sand With Occasional Stiff Silty Clay Zones																
80.6	Fill		1	SS	13												
2.4	Cobbles and Boulders		2	BXL RC	58%												
78.2	Rock																
4.8	Refusal to Drill End of Borehole		3	BXL	77%												

+<sup>3</sup>, x<sup>5</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
5 (%) STRAIN AT FAILURE



Ministry of  
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Communications

# RECORD OF BOREHOLE No 10

METRIC

W P 70-79-03 LOCATION Sta. 10+443.0; O/S 3.3 m RT of Hwy. 2 ORIGINATED BY NS  
DIST 8 HWY 2 BOREHOLE TYPE BX Casing, BXL Rock Core COMPILED BY NS  
DATUM Geodetic DATE 81 06 03 CHECKED BY NS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
82.9	Ground Elevation												
0.0	Sand and Gravel												
81.1	<del>Cobbles and Boulders</del> Fill												
1.8	Bedrock		1	BXL RC	97%								
	Good Quality		2	BXL RC	100%								
78.9	Hard												
4.0	End of Borehole												

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF PROBE HOLES NO. 11, 12 & 13

METRIC

W P 70-79-03 LOCATION See Below ORIGINATED BY NS  
DIST 8 HWY 2 BOREHOLE TYPE Hollow Stem Continuous Flight Augers COMPILED BY NS  
DATUM Geodetic DATE 81 06 03 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
	# 11 Sta. 10 + 438.7 20+438.5				0/S 3.4 m RT	E Hwy. 2											
83.0	Ground Level																
0.0	Sand and Gravel																
81.1	Occasional Cobbles and Boulders	Fill					82										
1.9	Refusal to Auger																
	Probable Bedrock																
	End of Borehole																
	# 12 Sta. 10 + 447.0 20+446.7				E Hwy. 2												
83.1	Ground Level																
0.0	Sand and Gravel	Fill															
82.3	Refusal to Auger						82										
0.8	Probable Bedrock																
	End of Borehole																
	# 13 Sta. 10 + 452.6 20+452.6				0/S 5.2 m LT	Hwy. 2											
83.0	Ground Level																
82.6	Topsoil																
0.4	Refusal to Auger						82										
	Probable Bedrock																
	End of Borehole																

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

# memorandum

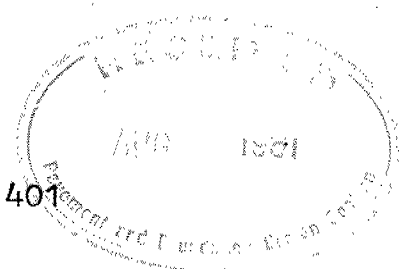


To: Mr. T.C. Kingsland  
Head, Structural Section  
Eastern Region, Kingston

Date: 81-08-05

From: Structural Section  
Eastern Region, Kingston

Re: Addendum to Structural Planning Report  
MacIlhennys Creek, 3.4 km West of Hwy. 401  
Hwy. 2, Site No. 16-112  
W.P. 70-79-03, B.W. 7007  
District 8 - Kingston



This addendum to the Structural Planning Report of 81-04-17, introduces information concerning foundation soils, structure type and detour recommendations based on information obtained subsequent to the issue of the original report.

## Foundations Information

At the request of this Section, the Pavement and Foundation Design Section has carried out further soils investigation which is detailed in a memorandum to Mr. T.C. Kingsland dated 81-07-07. A copy is attached to this addendum.

## Structure Alternatives

Four alternatives have been considered for the above structure. A discussion of these alternatives is given in the attached letter to Mr. J.W. Reid, Head, Planning and Design Section dated 81-06-16. Further considerations are outlined in the attached letter to File dated 81-06-10.

A review of the alternative structures in conjunction with the most recent Foundations recommendations and cost estimates has led to the following conclusions:

- The use of staged construction provides no advantages and is not considered practical at this site.
- A rigid frame type structure founded on piles is not considered viable for foundations reasons.
- Box type or culvert structures are not economical due to costs and the difficulties encountered in unwatering and stream diversion.
- The most suitable alternatives would be skewed single span beam type structures supported on perched abutments founded on spread footings on rock or rockfill.

- A structure with a span of 32 m would provide an adequate crossing, although the west footing would be founded on both bedrock and rockfill, which may introduce some differential settlement of the footing. This condition can be overcome by increasing the length of the structure by 10.5 m. This increased length results in substantial increase in cost and would preclude the use of a prestressed concrete beam design for the superstructure.

#### Recommendations

It is recommended that the structure be a single span precast beam bridge with a span of approximately 32 m. The abutments should be of spill through configuration founded on rock and rockfill as indicated in the foundations information.

The approach fills should consist of rockfill at slopes of  $1\frac{1}{2}:1$  on the front slopes and up to  $1\frac{1}{2}:1$  on the sideslopes. This rockfill is available either from the grading portion of the contract, or from quarries near by.

#### Detour

It is proposed to use a single span bailey bridge detour supported on rockfill approaches, located immediately to the south of the new structure.



E.C. Lane  
Sr. Structural Engineer

#### Attachment

c.c. M. Devata ←  
J.W. Reid  
R.W. Oddson  
K.G. Bassi  
J. Harris  
S.C.J. Radbone

# memorandum



To: FILE

Date: 81 06 10

From: Structural Section  
Eastern Region, Kingston

Re: Jones Creek - MacIlhennys Creek Bridge  
Review of Project

1. We have reviewed the sites for possible on-site detours and have found there to be feasible, without apparently being environmentally detrimental at either crossing site. The cost of such detours would be about \$30,000. each.

At each site the best location for a bailey bridge is the south side of the existing bridge. Only young growth is affected, mainly small poplars.

2. The foundation investigation which was carried out by consultants and was found to be unsatisfactory has been followed up by an extra in-house investigation. This is showing up poorer soils conditions than heitherto and will probably affect the chosen design for MacIlhennys Bridge. Field work is continuing and full results will be available shortly.
3. A study of possibilities for stage construction of a bridge at either site has been done (again) and found to be impracticable (as before).
4. Two possibilities for detours still remain, on site and off site. The off site routes are being investigated by Planning and Design Section. We traversed the County Road off site detours ourselves. These appeared to be good. The main factors appeared to be distance, school buses, Voyageur Colonial buses, (when running) and fire services. The latter problem could possibly be solved by a temporary agreement between adjoining townships,

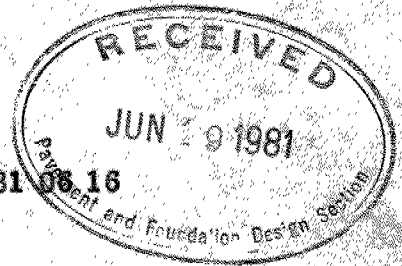
A handwritten signature in dark ink, appearing to read "T. C. Kingsland".

T. C. Kingsland  
Head, Structural Section

TCK:sh

c.c. R. W. Franks  
J. W. Reid

M.D. → T.K. → N.S.



Mr. J. W. Reid,  
Head,  
Planning & Design Section,  
Kingston, Ontario.

Attention: Mr. P. A. Jones

From: Structural Section,  
Kingston

Re: Jones and McIlhenny Creek Bridges,  
W.P. 70-79-02, 03, Sites 16-111, 112,  
Highway 2, District 8, Kingston.

We have investigated the alternatives for the structure types and  
detour arrangements for the above structures. Our comments are  
as follows:

Jones Creek Bridge

The use of staged construction at this site was considered  
but found to be of no advantage, though increasing the cost of the  
structure by approximately \$20,000. The structure as suggested  
in the structural planning report is a spill-through abutment beam  
type structure with a clear span of about 22.0 metres on skew.  
The cost of this structure excluding removal of the existing structure  
and detours would be approximately \$151,000. Further foundation  
work has been undertaken at this site. Pending the results it may be  
possible to construct a shorter structure of about 15 m span for  
about \$115,000 utilizing steeper embankments constructed of rock fill.  
In either case, the existing alignment would be maintained. The  
construction of a box type structure is not considered practical as  
the presence of the existing structure and the foundation conditions  
make unwatering and stream diversions very difficult.

Based on a site visit and further investigations, a Bailey  
Bridge detour using a 100' span Bailey supported on rockfill embank-  
ments could be constructed to the south of the existing site. The  
cost of such a detour including rental of the Bailey Bridge would be  
about \$30,000. The cost of removing the existing structure to the level  
of the top of the footings is estimated as \$8,000.00.

McIlhennys Creek Bridge

The use of staged construction is not considered practical  
for the reasons stated above.

The Structural Planning Report suggests a beam type structure  
supported on spill-through type abutments with a clear span of about  
32 metres. The estimated cost for this structure is about \$223,600.

Further foundation investigation is being carried out at this site and it may be necessary to lengthen this type of structure to a clear span of 39 metres increasing the cost to about \$270,000.00.

An alternative type structure could be constructed at an estimated cost of \$175,000 including dewatering although diverting the creek may prove more difficult than anticipated.

The most economical structure may now be a rigid frame bridge founded on piles with a clear span of about 18.5 metres. This could be constructed at a cost of about \$147,000 including any necessary dewatering and utilizing rockfill for the approaches. The use of earth fill would increase the required length of the wingwalls and increase the cost by approximately \$40,000.

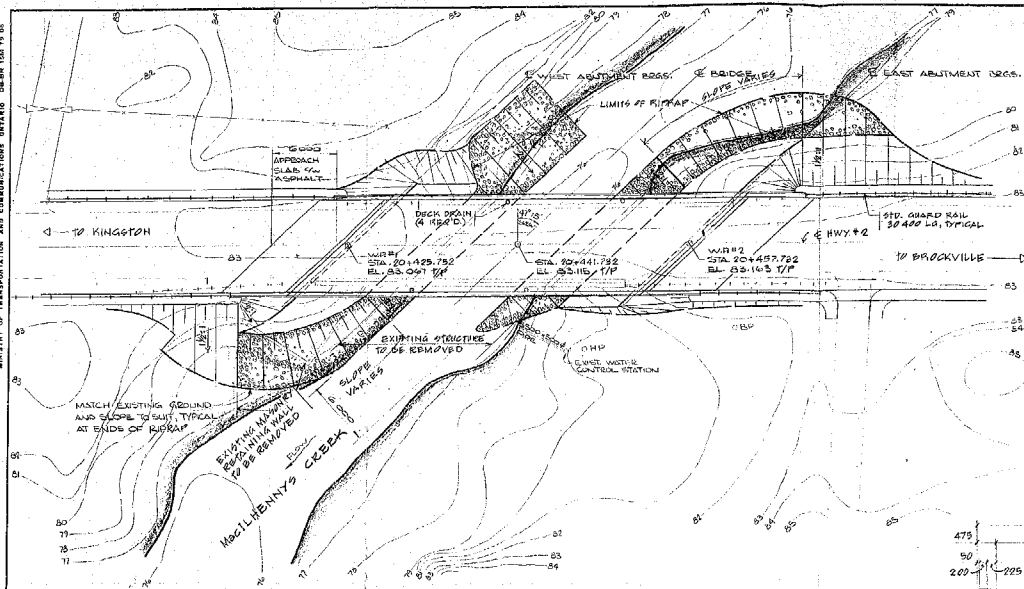
A bailey bridge detour appears feasible at this site using a 160' span bailey and rock fill approaches. The cost of such a detour would be about \$30,000. The cost of removal of the existing structure to the level of the top of footings would be about \$8,000.00.

This information is preliminary only and could change subject to release of the additional foundations information.

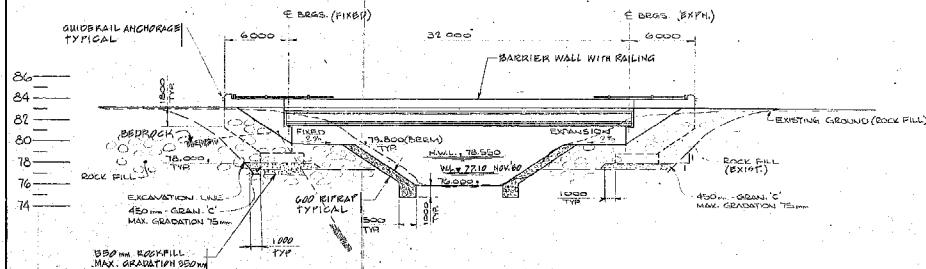
E. C. Lane,  
Structural Engineer

ECL/jtk

c. c. M. Devata ✓  
K. G. Bassi

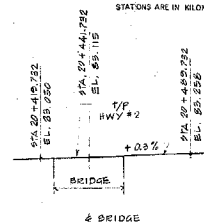


B.M. EL. 82.068  
GEODETIK DATUM  
N & W IN EAST ROOT OF  
300 mm DIA. PINE 18.800m  
LEFT OF STA. 20+325.632

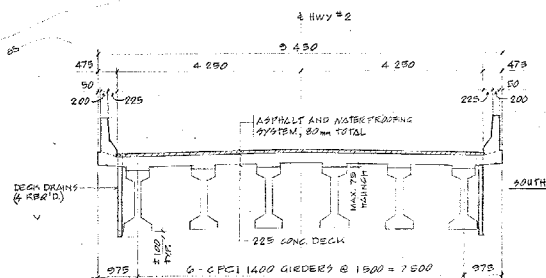


*METRIC*


DIMENSIONS ARE IN MILLIMETRES  
UNLESS OTHERWISE SHOWN.  
ELEVATIONS, COORDINATES, CURVE  
AND ALIGNMENT DATA ARE IN METRES.  
STATIONS ARE IN KILOMETRES + METRES.



PROFILE - HWY. #2  
SCALE 1:1000



DECK SECTION  
SCALE 1:50

DIST. No. 8 CONT No WP No 70-79-03	
MacILHENNYS CREEK BRIDGE GENERAL ARRANGEMENT	SHEET

**M** McNEELY ENGINEERING  
& STRUCTURES LTD.  
KINGSTON OTTAWA

GENERAL NOTES:

1. REINFORCING STEEL SHALL BE G.S.A. GS-157M-1977 GRADE 400 UNLESS OTHERWISE NOTED. BARS HAVING "X" SUFFIX "C" SHALL BE COATED BARS.
2. CLEAR COVER TO REINFORCING STEEL (UNLESS OTHERWISE NOTED)

FOOTINGS	100±25
COL. & RTG. AND WINGWALLS	75±25
3. APPROACH SLABS

DECK, TOP AND PARALLEL WALLS	70±10
DECK, BOTTOM	70±0
	40±10
4. REINFORCING STEEL NOTATION:

N.F. - NEAR FACE	T.O. - TOP
F.F. - FAR FACE	T.B. - TOP & BOTTOM
E.C. - EACH FACE	
5. CLASS OF CONCRETE:

PRECAST PRESTRESSED GIRDERS	40 MPa
DECK & PARALLEL WALLS	30 MPa
TOP OF WINGWALLS	40 MPa
REMAINING	20 MPa
6. BRANDING SEATS TO BE PUSHED DEAD LEVEL TO THE SPECIFIED ELEVATIONS TO A TOLERANCE OF ± 3mm.
7. NO CONCRETE SHALL BE PLACED ABOVE LEVEL OF BRANDING SEATS UNTIL DECK CONCRETE HAS BEEN PLACED.
8. ALL EXPOSED CONCRETE EDGES TO HAVE 20 mm ± 20 mm CHAMFER UNLESS OTHERWISE NOTED.
9. CONCRETE QUANTITIES:

CONCRETE QUANTITIES ARE LISTED FOR THE APPROXIMATE CONCRETE LUMP SUM TENDER ITEMS.	
ADJUSTMENTS & WINGWALLS	187 m³
DECK	91 m³
PARALLEL WALLS	21 m³
APPROACH SLABS	28 m³
10. LONG BARRIER WALLS ON WINGWALL SHALL NOT BE CAST UNTIL THE WINGWALL BACKFILL HAS BEEN COMPLETED.

ADJUSTMENTS & WINGWALLS	197	m
DECK	91	m
BARRIER WALLS	21	m
APPROACH SLABS	28	m

### LEGEND

T/P DENOTES TOP OF FINISHED PAVEMENT  
W.P. DENOTES WORKING POINT.



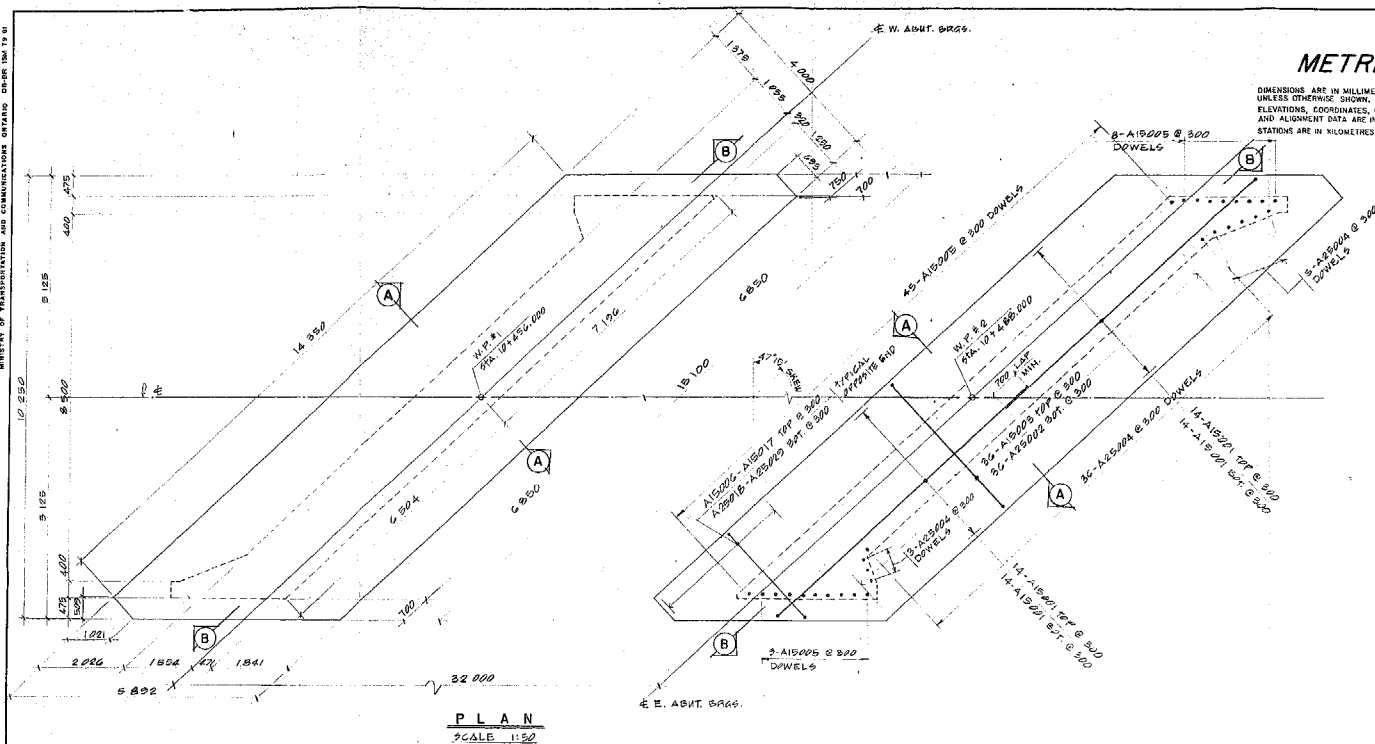
DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

REVISIONS									
	DATE	BY	DESCRIPTION						
DESIGN	S.C.	CHECK	D.K.M.	LOADING CHART		DATE		8-11	
DRAWING	2-2	CHECK	D.K.M.	SITE No 16-112		DWG		3-1	

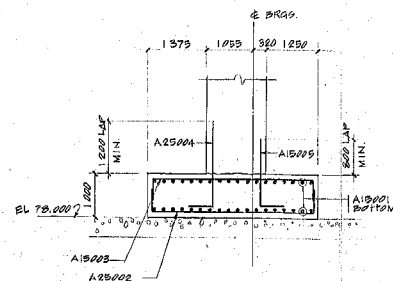
# METRIC

DIMENSIONS ARE IN MILLIMETRES  
UNLESS OTHERWISE SHOWN.  
ELEVATIONS, COORDINATES, CURVE  
AND ALIGNMENT DATA ARE IN METRES.  
STATIONS ARE IN KILOMETRES + METRES.

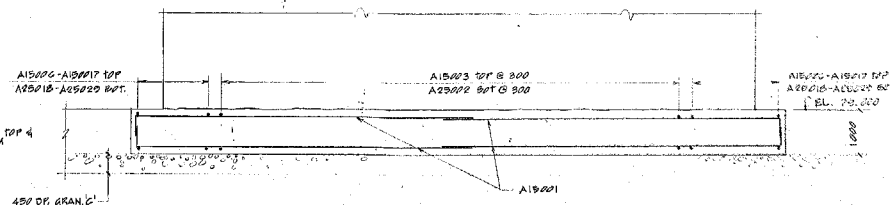
DIST. No. 8	
CONT No	
WP No. 70-79-03	
MacILHENNYS CREEK BRIDGE	SHEET
FOOTINGS	



PLAN  
SCALE 1:50



SECTION A  
SCALE 1:50



SECTION B  
SCALE 1:50

- NOTES:
- 1) SEE ASBUILT DRAWINGS FOR GEOMETRY OF WALLS ABOVE FOOTINGS.
  - 2) DIMENSIONS AND REINFORCING STEEL SHOWN ARE TYPICAL.



DATE BY	CHECK BY	DESCRIPTION	DATE
DESIGN	CHK	CHK	CHK
DRAWING	CHK	CHK	CHK

DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING