DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 31C-142

DIST. 8 REGION

W.P. No. 25-77-04

CONT. No. 91-31

W. O. No.

STR. SITE No. 17-103

HWY. No. 33

LOCATION Parrotts Bay Bridge

No. of Pages - 1

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

METRIC

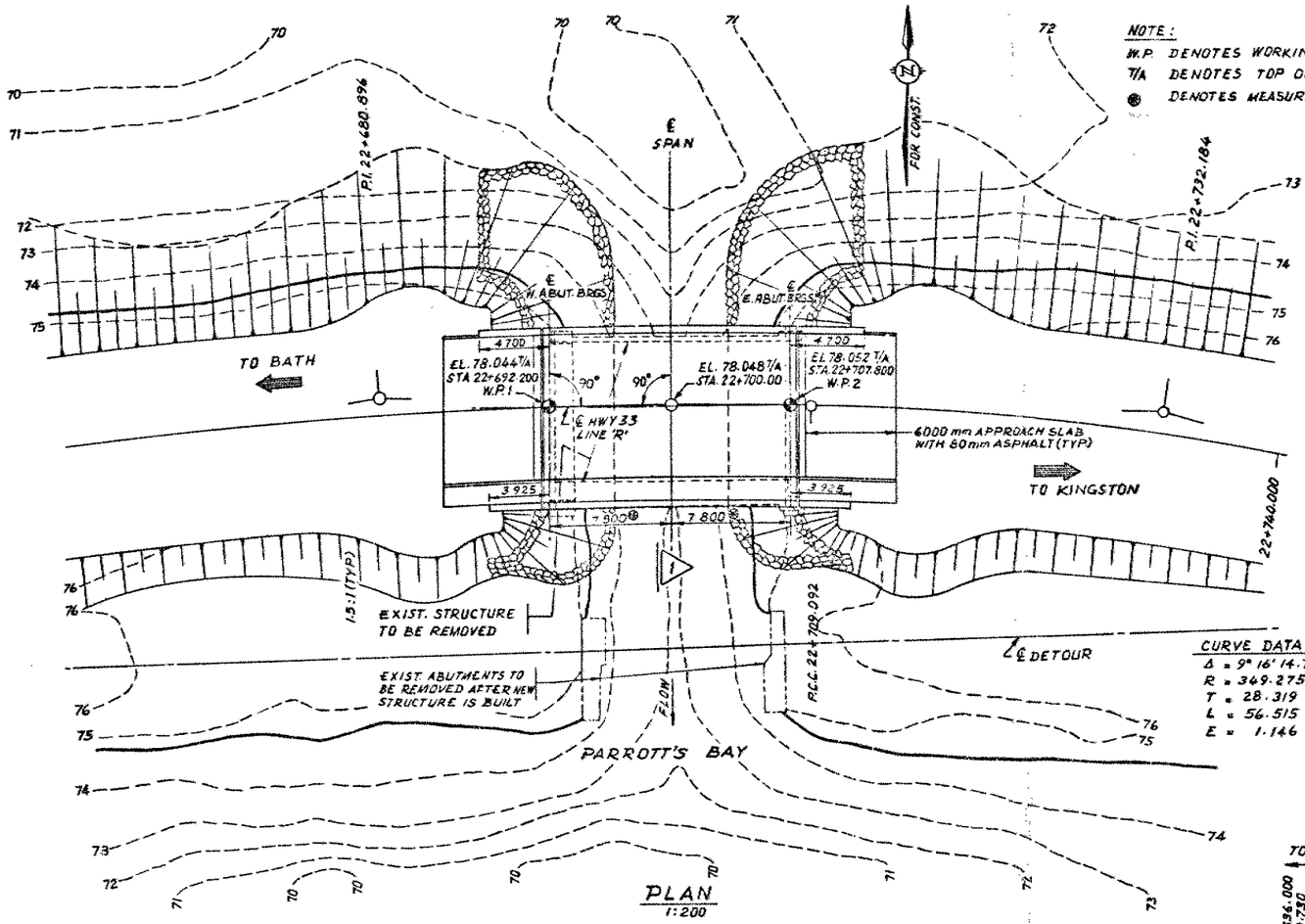
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DISTRICT - 8 KINGSTON
CONT No
WP No 25 - 77 - 04

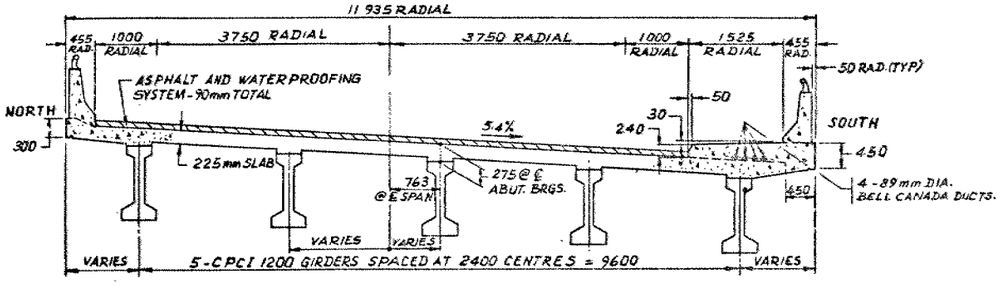
PARROTT'S BAY BRIDGE
GENERAL ARRANGEMENT

SHEET

NOTE:
W.P. DENOTES WORKING POINT
T/A DENOTES TOP OF ASPHALT
℄ DENOTES MEASURED ALONG ℄ HWY 33 LINE 'R'

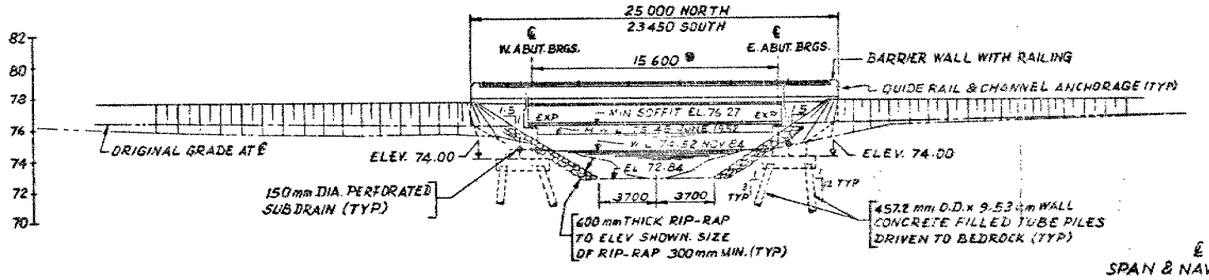


PLAN
1:200

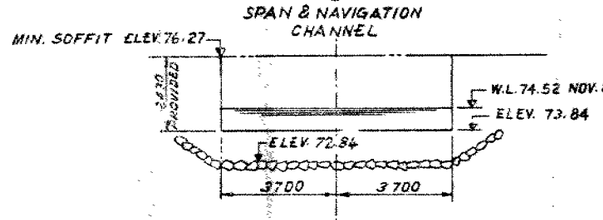


TYPICAL DECK SECTION (1)
1:50

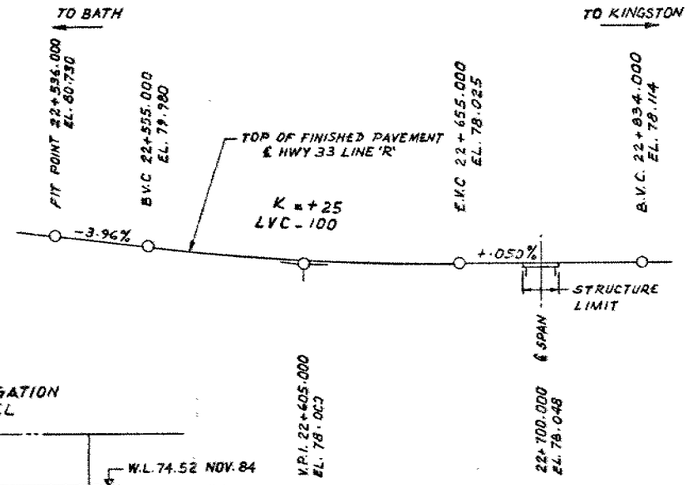
CURVE DATA (1)	CURVE DATA (2)
Δ = 9° 16' 14.77"	Δ = 15° 03' 54.48"
R = 369.275	R = 174.638
T = 28.319	T = 23.092
L = 56.515	L = 45.919
E = 1.146	E = 1.520



SOUTH ELEVATION
1:200



MINIMUM REQUIRED NAVIGATION CLEARANCE



PROFILE HWY 33
N.T.S.

NOTES:

CLASS OF CONCRETE
PRESTRESSED GIRDERS ---------- 40MPa
FOOTINGS ---------- 20MPa
REMAINDER ---------- 30MPa

REINFORCING STEEL
REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX 'C' DENOTES COATED BARS.

CLEAR COVER TO REINFORCING STEEL
FOOTINGS ---------- 100 ± 25
ABUTMENTS & WINGWALLS
- FRONT FACE ---------- 80 ± 20
- BACK FACE ---------- 70 ± 20
DECK - TOP ---------- 70 ± 20
— " — BOTTOM ---------- 40 ± 10
REMAINDER UNLESS OTHERWISE NOTED --- 70 ± 20

CONSTRUCTION NOTES
THE CONTRACTOR SHALL FINISH THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS, TO A TOLERANCE OF ± 3mm.

LIST OF DRAWINGS
17-103- 1 GENERAL ARRANGEMENT
- 2 BORE HOLE LOCATIONS AND SOIL STRATA
- 3 FOOTINGS
- 4 WEST ABUTMENT
- 5 EAST ABUTMENT
- 6 PRESTRESSED GIRDERS & BEARINGS
- 7 DECK REINFORCEMENT & DETAILS
- 8 6000 mm APPROACH SLAB
- 9 BARRIER WALL WITH RAILING
- 10 BARRIER WALL ON SIDEWALK
- 11 RAILING FOR BARRIER WALL
- 12 JOINT ANCHORAGE & ARMOURING
- 13 BRIDGE DATE AND SITE NUMBER DATA
- 14 AS CONSTRUCTED ELEV. & DIMENSIONS
- 15 BRIDGE AESTHETICS
- 16 QUANTITIES - STRUCTURE
- 17 QUANTITIES - STRUCTURE

B.M. ELEV. 76.735
GEODETIC DATUM
CUT CROSS ON ROCK OUTCROP
17.6 RT. OF STA. 22+769.6

PROFESSIONAL ENGINEER PROVINCE OF ONTARIO Q. M. ISLAM

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	R.I	CHECK	LOAD NO. OHBDC - A83	DATE DEC/85
DRAWING	T.M.N	CHECK	Q.I	SITE No. 17-103	DWG 1

METRIC

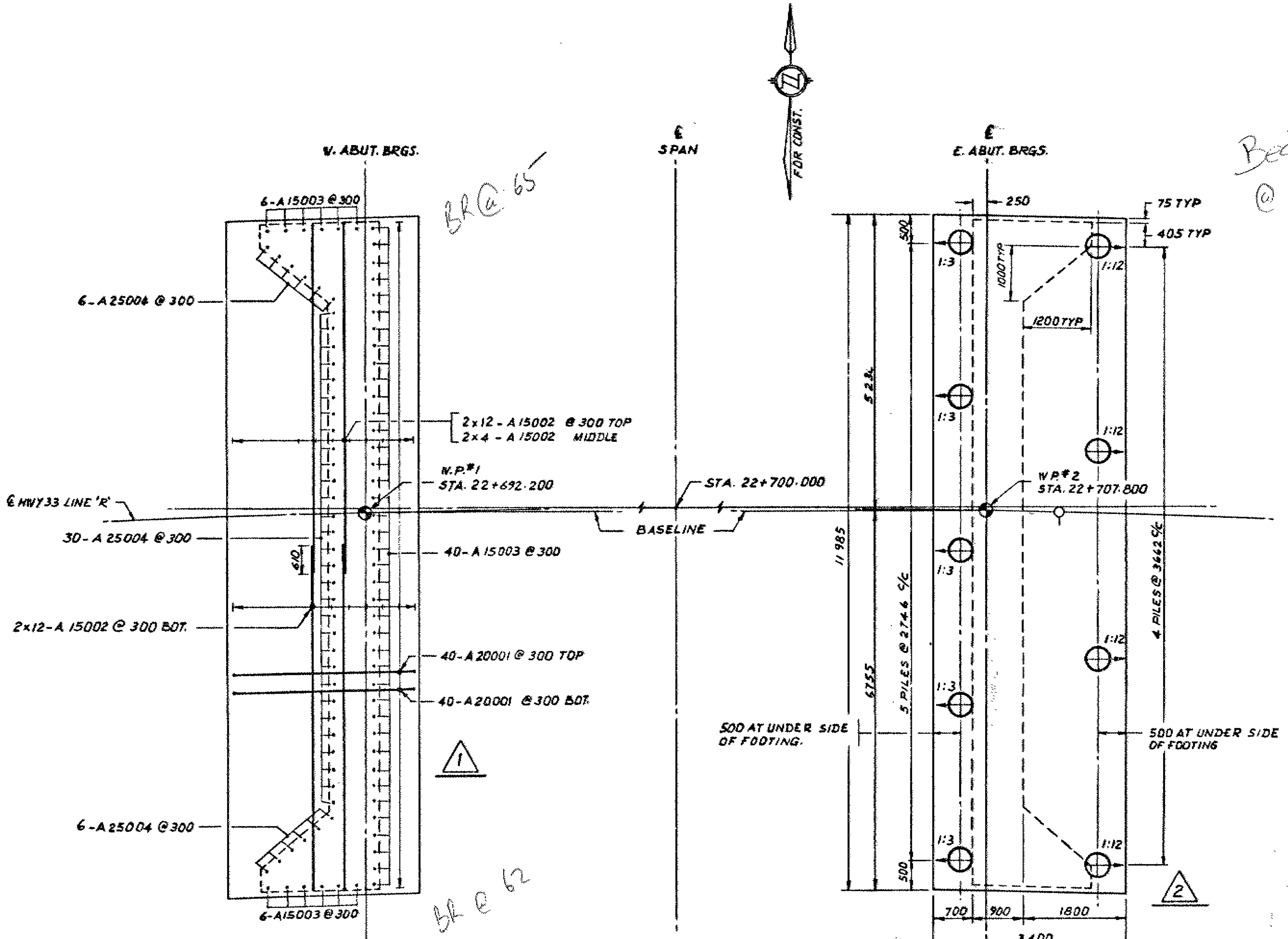
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No
WP No 25-77-04

PARROTT'S BAY BRIDGE

FOOTINGS

SHEET

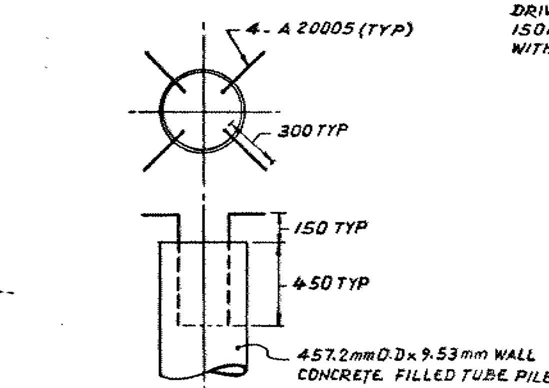


FOOTING - REINFORCEMENT (TYP)

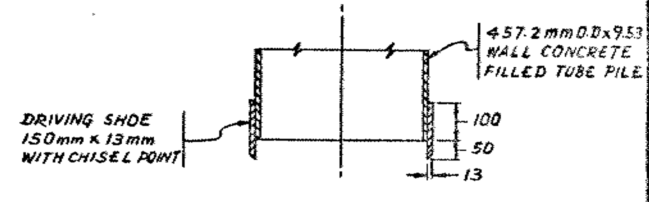
PILE LAYOUT & FOOTING DIMENSIONS (TYP)

PLAN
1:50

Bedrock @ elev. 65



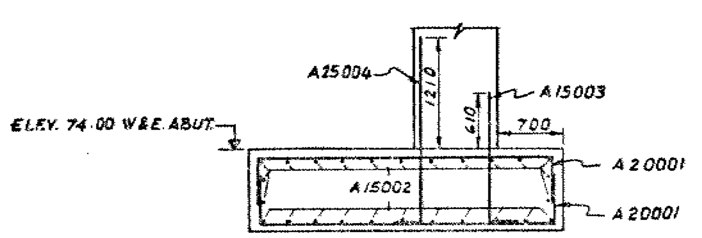
PILE ANCHORAGE DOWEL DETAIL
1:20



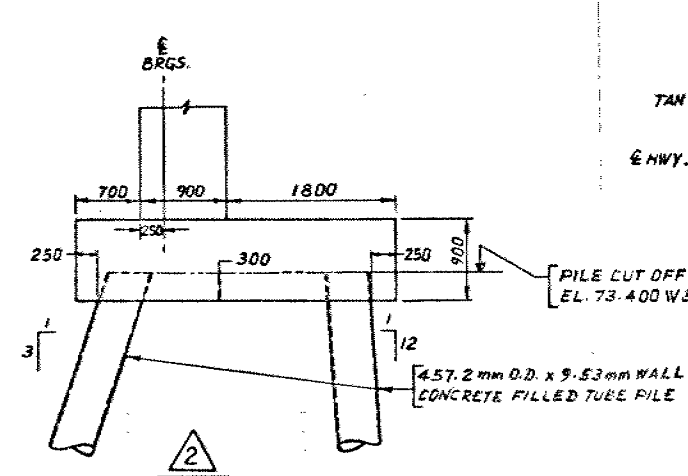
DRIVING SHOE DETAIL
1:10

PILE DATA-457.2mm O.D x 9.53mm TUBE PILE

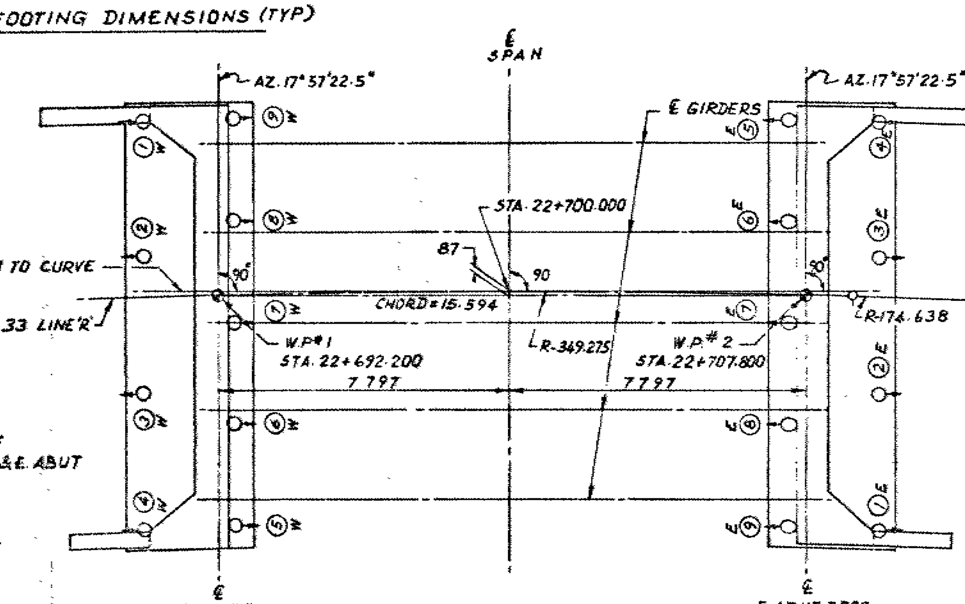
WEST FOOTING				EAST FOOTING			
PILE NO	BATTER	QTY	LENGTH	PILE NO	BATTER	QTY	LENGTH
1 W	1:12	1	7 950	1 E	1:12	1	8 350
2 W		1	8 850	2 E		1	8 550
3 W		1	9 700	3 E		1	8 750
4 W		1	10 600	4 E		1	8 950
5 W	1:3	1	11 250	5 E	1:3	1	9 400
6 W		1	10 500	6 E		1	9 200
7 W		1	9 870	7 E		1	9 100
8 W		1	9 250	8 E		1	8 900
9 W		1	8 500	9 E		1	8 650
CUT OFF ELEV.	73.400			CUT OFF ELEV.	73.400		



1:40



1:40



LOCATION OF WORKING POINTS
1:100

NOTES:

- PILES TO BE DRIVEN TO BEDROCK.
- PILE SPACING TO BE MEASURED AT UNDERSIDE OF FOOTING.
- PILE LENGTH SHOWN ON THE DRAWING IS THEORETICAL LENGTH BELOW CUT-OFF.
- ALL PILES TO HAVE DRIVING SHOES AS SHOWN ON THE CONTRACT DRAWINGS.
- SPLICE SHALL BE AS PER STANDARD DD 3302

Q. M. ISLAM

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN Q.I	CHECK	LOADING OHBDC - A83	DATE APR./87
DRAWING TMN	CHECK Q.I	SITE No 17-103	DWG 3

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 25-77-04 DIST 8

HWY 33 STR SITE 17-103

CONT 91-31

PARROTTS BAY BRIDGE

DISTRIBUTION

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GEOCRES 31C - 142

DATE 86 01 06

FOUNDATION INVESTIGATION REPORT
For
W. P. 25-77-04; Site: 17-103
Parrotts Bay Bridge
Hwy. #33, District #8, Kingston

INTRODUCTION

This report summarizes the results of a foundation investigation required for the above-noted structure replacement.

The fieldwork was conducted between 85 08 28 and 85 09 04 utilizing a continuous flight auger machine equipped with 82 mm I.D. hollow-stem augers and B core barrels.

This work consisted of four sampled boreholes.

SITE DESCRIPTION

The site is located in the County of Lennox and Addington, Township of Ernestown at the crossing of Hwy. 33, Line 'R' over Parrotts Bay. Parrotts Bay is approximately 5 km east of the Hwy. 33/Hwy. 133 intersection.

The limits of this investigation are Sta. 22 + 600 to Sta. 22 + 740 (Line 'R' Chainage).

According to Chapman and Putnum (1984), the site lies within the 'Napanee Plain' physiographic area. This plain is generally characterized by shallow overburden and numerous limestone outcrops. At this specific site, however, the limestone bedrock has been eroded to form Parrotts Bay.

The existing crossing of Hwy. 33 over Parrotts Bay is a 13± m long single span bridge in conjunction with relatively level causeway approaches at a grade of elev. 76± m..

SUBSURFACE CONDITIONS

General

The Record of Borehole Sheets, (Appendix) illustrate the conditions at the borehole locations (refer to BH #1 to BH #4). The locations and elevations of the boreholes and stratigraphical profiles based on the borehole data, are shown on the Borehole Locations and Soil Strata Drawing No. 257704-A.

The sequence (from the surface downwards) of subsurface materials at the borehole locations is summarized below:

Material	Thickness
- Boulders, Gravel and Sand	6.7 m to 10.4 m
- Silty Sand to Sandy Silt with Organics	1.2 m to 3.6 m
- Limestone Bedrock	----------

Stratigraphy

Boulders, Gravel and Sand,
trace silt, trace clay,
occasional organic zones

This loose to very dense material was encountered at the surface across the site. The thickness of this material ranges from 6.7 m to 10.4 m at the borehole locations.

The material is interpreted as being a rock fill. It should be noted that diamond drilling was required to advance boreholes through this fill at a number of locations.

Silty Sand to Sandy Silt;
trace/with organics, trace/some clay,
occassional gravel zones, occassional boulders

This very loose to very dense material underlies the rock fill and overlies the bedrock. The thickness of the deposit ranges from 1.2 m to 3.6 m at the borehole locations.

Due to the presence of organics and clay, the material is slightly cohesive.

Physical properties of the material, as determined from the results of field and laboratory tests, are summarized below.

	Range	Average	Median
Natural Moisture Content (w)	15.0-144.0%	76.2%	70.5%
Liquid Limit (w_L)	20.0-104.0%	59.2%	50.0%
Plastic Limit (w_P)	14.5-102.5%	55.2%	41.0%
Organic Content	1.0- 35.0%	18.7%	19.5%

A typical range of grain size distributions is indicated below:

Gravel	0 - 32%
Sand	16 - 49%
Silt	11 - 70%
Clay	8 - 15%

Bedrock

At the borehole locations, bedrock was encountered at elevations ranging from 62.1 m to 65.4 m.

The bedrock is limestone of the Gull River Formation of the Trenton and Black River Group. For detailed descriptions of the bedrock core recovered, refer to the Description of Rock Core in Table 1 of the Appendix.

Groundwater

At the time of the field investigation, the groundwater elevation at the borehole locations was approximately the same as the prevailing level of Lake Ontario (elev. 75 ± m).

DISCUSSION AND RECOMMENDATIONS

It is proposed to replace the existing single span structure carrying Hwy. 33 over Parrotts Bay with a 3-span bridge (8.1 m, 13.5 m, 8.1 m) along a new alignment (Line'R') located less than 1 m north of the existing C/L. The proposal also involves increasing the grade by up to 2.8± m to elev. 78.5± m.

Originally the proposal for the structure replacement involved a single span bridge. However, during preparation of this report the design proposal was changed to a 3-span structure. In view of this change, the need for further foundation investigations is being evaluated, and if it is determined that additional fieldwork is required, the information will be issued in the form of an addendum. However, it is expected that the recommendations will remain essentially the same.

Two foundation problems have been addressed:
1) support for the abutments and piers
2) minimizing the settlement of the approaches.

The recommendations in this report was applicable to the alignment from Sta. 22 + 660 to Sta. 22 + 740.

It is also proposed to construct a temporary detour to the south of the existing alignment.

Structure Design

STRUCTURE FOUNDATIONS

Three alternatives are recommended. The alternative which leads to the least expensive design should be adopted.

Alternative 1: Steel H-Piles Driven to Bedrock

The footings may be supported on 310 HP 110 steel H-piles, equipped with reinforced tips, and driven to bedrock. Please refer to the Record of Borehole Sheets for bedrock elevations at the borehole locations.

Pre-augering will be required to penetrate the bouldery zones. After the piles have been installed, the pre-augered holes should be backfilled with mass concrete placed by tremie methods. It may be neccessary to provide casing to prevent cave-in of the pre-augered holes until the concrete backfill has been placed. These casings may be left in place if desired. In view of these installation difficulties, Alternatives 2 and 3 may be more appropriate for this site. The following design values are recommended:

Pile Type	Factored Capacity at U.L.S.	Capacity of S.L.S. TYPE II
310 HP 110	1600 kN per pile	1150 kN per pile

However, the loading should not exceed the structural capacity of the pile.

Alternative 2: Steel Tube Piles Driven to Bedrock

The footings may be supported on concrete-filled steel tube piles driven to bedrock. Please refer to the Record of Borehole Sheets for bedrock elevations at the borehole locations.

The tube piles should be installed open-ended, as a combination of driving and drilling will be necessary to advance through the bouldery zone. When required, the drilling can be carried out through the tube pile. When the piles have been seated on the bedrock, they should be cleaned out and filled with concrete placed in the dry (after dewatering the liner) or by tremie methods.

The following design values are recommended:

Pile Type	Factored Capacity at U.L.S.	Capacity at S.L.S. TYPE II
Steel Tube 324 mm x 9.5 mm	1600 kN per pile	1150 kN per pile

However, the loading should not exceed the structural capacity of the pile.

Alternative 3: Reinforced Concrete Caissons on Bedrock

The footings may be supported on concrete caissons socketed a minimum of 0.15 m into the bedrock. Please refer to the Record of Borehole Sheets for the bedrock elevations at the borehole locations. The caissons may be constructed by advancing a steel liner through the overburden and socketing it into the bedrock. If additional frictional resistance is required, the caisson can be socketed deeper into the bedrock. This operation will require drilling in order to penetrate the boulders and bedrock. After the liner has been cleaned out and the required reinforcing has been installed, the concrete should be placed in the dry (after dewatering the liner) or by tremie methods. The steel liner should remain in place after construction of the caisson has been completed.

The following design values are recommended:

Caisson Diameter	Factored Capacity at U.L.S.	Capacity S.L.S. TYPE II
0.76 m	3000 kN per caisson	N/A

If larger diameter caissons are considered, please contact this office for design details.

The capacity at S.L.S. Type II will not govern design as the bedrock will not settle appreciably. However, the structural capacity of the caisson should not be exceeded, and its safe geotechnical loading should not exceed the U.L.S. values recommended.

EARTH PRESSURES CALCULATIONS

Backfill to structures should consist of granular materials in accordance with MTC Standard Special Provision #121 (83 10). Computation of earth pressures should be in accordance with Section 6.6.1.2. of the O.H.B.D.C.

For design purposes, the physical properties of the backfill are as follows:

Material	ø	ɣ
Granular 'A'	35°	22.0 kN/m^3
Granular 'B'	30°	21.2 kN/m^3
Rock Fill	35°	20.0 kN/3^3

(If lightweight fill is considered for backfill behind abutments, please contact this office for physical properties of lightweight backfill for earth pressure calculation purposes, and for design details).

For structures supported on piles or caissons founded on bedrock, the foundation is considered to be non-yielding, and the at-rest condition applies for calculations of lateral earth pressures.

For structures supported on rock fill, the foundation is considered to be yielding, and the active condition applies for calculations of lateral earth pressures.

SLOPE STABILITY

Final slopes should be 2H:1V or flatter for earth fill, and 1.5H:1V or flatter for rock fill.

Temporary slopes may be 1.5H:1V or flatter.

FROST PROTECTION

For frost protection, 1.5 m of earth cover, or equivalent, is required.

DE-WATERING

De-watering for pile caps should not be required if they are constructed above the prevailing groundwater elevation.

SETTLEMENT

Differential settlements of the proposed abutments will be negligible if they are constructed in accordance with the recommendations provided in this report.

APPROACH EMBANKMENTS

Settlements of up to 0.5 m are anticipated under the proposed loading conditions at the approach embankments. The majority of this settlement will occur within the Silty Sand to Sandy Silt deposit containing organics, underlying the existing Boulders, Gravel and Sand (rock fill).

In order to minimize the effects of this settlement on the completed alignment, preloading and surcharging of the approach embankments between Sta. 22 + 660 and Sta. 22 + 740 is recommended. The surcharge should be 1 m above the final grade and extend over the plan limits of the proposed embankments.

The preload period should be a minimum of six months. The embankment preload requirements are applicable to the forward direction (including the areas over the proposed abutments) as well as the transverse directions. Although these preload requirements will reduce post-construction settlements (by an estimated 50%), some maintenance may be required due to residual settlements of the approach embankments.

Post construction settlements could be virtually eliminated by using lightweight fill (slag or styrofoam) to construct the approach embankments. This proposal would involve preloading with normal fill, as described above, then subexcavating to the existing grade, and constructing the approach embankments with lightweight fill. The lightweight fill treatment would extend 25± m behind the abutments. If this option is considered, please contact this office for details regarding design and construction. As a preliminary estimate, the following F.O.B. material cost estimates have been determined:

a)	pelletized	'3/8"	Structural Coarse'	slag	$39/m^3$
b)	pelletized		'Old Clinker'	slag	$36/m^3$
c)	air-cooled		'Open-Graded Pit Run'	slag	$28/m^3$
d)	air-cooled	' 1 "	Clear'	slag	$30/m^3$
e)	styrofoam				$85/m^3$

To facilitate pile driving, particle sizes in the fill immediately beneath the pile locations should not exceed 75 mm for steel H-piles and 50 mm for steel tube piles.

EROSION PROTECTION

Where embankments adjacent to the lake/bay are constructed of material other than rock fill, erosion protection, in the form of random rip rap (minimum blanket thickness = 0.6 m) should be placed on the abutment slopes extending from the toe to 0.6 m above the high water level. The rip rap should extend a minimum of 2 m out along the lake/bay bottom.

In a transverse direction, this erosion protection should extend a minimum of 10 m on both sides of the abutments.

DETOUR

It is proposed to construct a temporary detour to the south of the Hwy. 33 alignment. It is our understanding that this proposal involves utilizing the abandoned abutments as foundations for the detour bailey bridge. Alternatively, the detour bailey bridge could be founded on rock fill.

The following design values are recommended for both detour alternatives:

- Factored Bearing Capacity at U.L.S. = 600 kPa
- Bearing Capacity at S.L.S. Type II = 250 kPa

Even at these recommended loadings, small settlements of the detour bridge may occur and periodic maintenance may be required.

MISCELLANEOUS

The fieldwork for this project was carried out under the supervision of Mr. Z. Najak, Student Engineer.

The report was written by Mr. D. Dundas, Senior Foundations Engineer, and reviewed by Mr. M. Devata, Chief Foundatons Engineer.

The equipment used for the field investigation was owned and operated by Marathon Drilling Co. Limited.

D. H. Dundas

D. H. Dundas, P. Eng.
Senior Foundations Engineer

M. Devata

M. Devata, P. Eng.
Chief Foundations Engineer
(East)

A P P E N D I X

TABLE 1 DESCRIPTION OF ROCK CORE - W.P. 25-77-04

BOREHOLE NUMBER				CORE DESCRIPTION	
	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	9.7 - 10.4 - 11.4 - 12.8	96 100 100	0 85 28	9.7 - 10.4 10.4 - 12.8	Dolostone, brown, highly weathered, very closely spaced joints Limestone, brownish grey becoming dark grey, fine grained, medium spaced becoming very closely spaced joints
2	13.4 - 14.9 - 15.6 - 16.2 - 16.5 - 18.0	73 92 67 100 92	0 0 0 0 30	13.4 - 17.1 17.1 - 18.0	Limestone, light grey to greenish grey, slightly to moderately weathered, very closely spaced joints Limestone, grey, slightly weathered, closely spaced joints (Evidence of fault zone from about 14.9 to 17.8; ie calcite veining at about 70° to horizontal; bedding in sediment highly inclined)
3	11.3 - 12.7 - 14.3 - 15.2	79 98 100	44 51 61	11.3 - 11.6 11.6 - 15.2	Core loss for first run assumed to be at top of run. Limestone, grey, unweathered, closely spaced joints

* CR = CORE RECOVERY ; RQD = ROCK QUALITY DESIGNATION

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 63.5kg, 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 475 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 (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 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
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_p	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_p	%	PLASTICITY INDEX = $w_L - w_p$
I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 1 METRIC

W P 25-77-04 LOCATION STA. 22 + 710.4; °/s 11.0 m RT ℄ HWY. 33 LINE 'R' ORIGINATED BY Z.N.

DIST 8 HWY 33 BOREHOLE TYPE H-S Auger, B-Core COMPILED BY D.D.

DATUM Geodetic DATE 85 08 28 CHECKED BY D.D.

SOIL PROFILE ELEV/DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.1	Ground Surface										
0.0	Boulders, Gravel and Sand Trace Silt Trace Clay Occ. Organic Zones Loose to Very Dense		1	SS	26		74				
			2	SS	14						
			3	SS	12		72				
			4	SS	18						
			5	SS	8						
			6	SS	5		70				
			7	SS	23						
68.4			8	SS	24					org= 7.5%	46 35 17 2
6.7	Silty Sand to Sandy Silt Trace/with Organics Trace/some Clay Occ. Gravel Zones Occ. Boulders Slightly Cohesive Very Loose to Very Dense		9	SS	3		68			org= 8.9%	0 17 70 13
65.4			10	SS	100		66				6 16 63 15
9.7	weathered unweathered		11	RC	Rec 96%						
	Bedrock Limestone		12	RC	Rec 100%		64				
62.3			13	RC	Rec 100%						
12.8	End of Borehole										

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2 METRIC

W P 25-77-04 LOCATION STA. 22 + 691.4; °/s 11.0 m RT ℄ HWY. 33 LINE 'R' ORIGINATED BY Z.N.

DIST 8 HWY 33 BOREHOLE TYPE H-S Auger, B-Core COMPILED BY D.D.

DATUM Geodetic DATE 85 09 03 CHECKED BY D.D.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.5	Ground Surface										
0.0	Occ. Silty Clay Zones Boulders, Gravel and Sand Trace Silt Trace Clay Occ. Organic Zones Loose to Very Dense		1	SS	13		74				
			2	SS	4		72				
			3	SS	10		70				
			4	SS	39						
			5	SS	43		68				
			6	SS	8						
65.7			7	SS	38				o ⊢	org= 3.7%	66 23 7 4
9.8	Silty Sand to Sandy Silt Trace/with Organics Trace/some Clay Occ. Gravel Zones Occ. Boulders Slightly Cohesive Loose to Very Dense		8	SS	7		66		W = 131.5 % W_L = 96.0 % W_p = 96.0 %	org= 30.1%	0 48 43 9
			9	SS	5						
			10	SS	6		64		W = 144.0 % W_L = 104.0 % W_p = 102.5 %	org= 35.0%	0 49 41 10
62.1	Frequent Boulders		11	SS	17						
13.4	Bedrock Limestone Unweathered		12	SS	100		62				
			13	RC	Rec 73%						
			14	RC	Rec 92%		60				
			15	RC	Rec 78%						
57.5			16	RC	Rec 100%		58				
18.0	End of Borehole										

+3, ×5 : Numbers refer to Sensitivity

20
15 ◇ 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 3 METRIC

W P 25-77-04 LOCATION STA. 22 + 689.8; °/s 1.5 m LT ℄ HWY. 33 LINE 'R' ORIGINATED BY Z.N.
DIST 8 HWY 33 BOREHOLE TYPE H-S Auger, B-Core COMPILED BY D.D.
DATUM Geodetic DATE 85 09 04 CHECKED BY D.D.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
76.2	Ground Surface										
0.0	Boulders, Gravel, and Sand Trace Silt Trace Clay Occ. Organic Zones Loose to Very Dense		1	SS	24	▼	76				
							74				
			2	SS	17		72				
			3	SS	30		70				
68.0											
8.2	Silty Sand to Sandy Silt Trace/with Organics Trace/some Clay Occ. Gravel Zones Occ. Boulders Slightly Cohesive Compact		4	SS	18		68				
							66		○	org= 1.0%	32 49 11 8
64.9											
11.3	weathered - - - unweathered Bedrock Limestone		5	RC	Rec 79%		64				
			6	RC	Rec 98%		62				
61.0			7	RC	Rec 100%						
15.2	End of Borehole										

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 4 METRIC

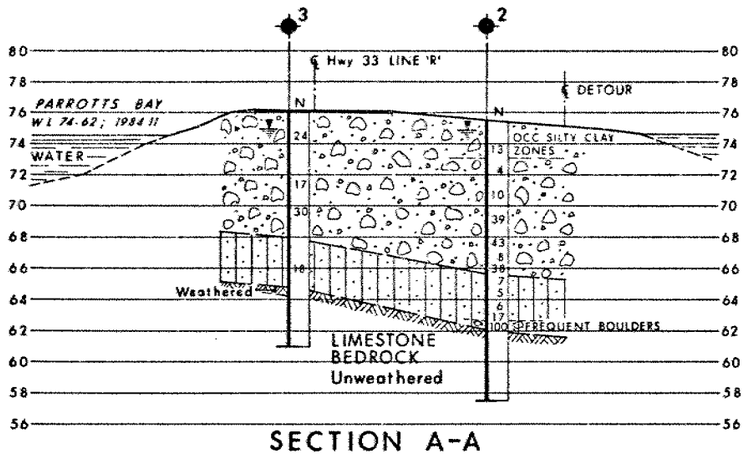
W P 25-77-04 LOCATION STA. 22 + 712.1; °/s 2.9 m LT ℄ HWY. 33 LINE 'R' ORIGINATED BY Z.N.
DIST 8 HWY 33 BOREHOLE TYPE H-S Auger, B-Core COMPILED BY D.D.
DATUM Geodetic DATE 85 08 29 CHECKED BY D.D.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT Wl — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
76.3	Ground Surface										
0.0	Boulders, Gravel and Sand Trace Silt Trace Clay Occ. Organic Zones Loose to Very Dense					*	76				
			1	SS	23						
			2	SS	100/15cm		74				
			3	SS	100/15cm						
			4	SS	117/23cm		72				
			5	RC	Rec		70				
			6	RC	Rec						
			7	SS	8		68				
			8	RC	Rec						
65.9			9	RC	Rec 50%		66				
10.4	Probable Silty Sand to Sandy Silt **										
64.7							64				
11.6	Bedrock Limestone Unweathered		10	RC	Rec 100%						
61.5			11	RC	Rec 100%		62				
14.8	End of Borehole * Groundwater elevation not determined ** Trace/with Organics Trace/some Clay Occ. Gravel Zones Occ. Boulders Slightly cohesive Loose to Very Dense										

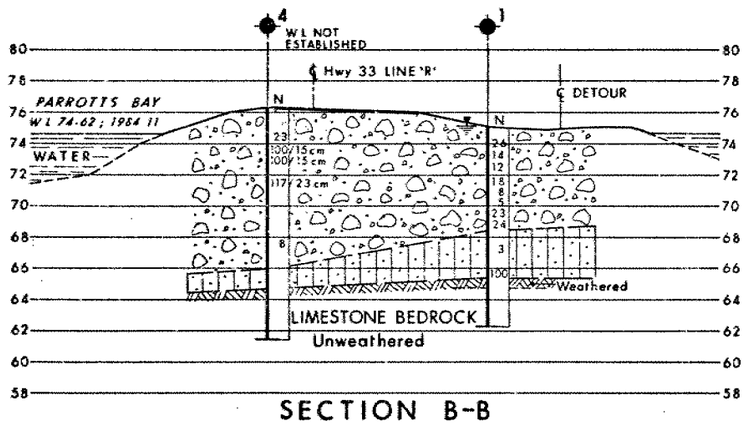
+3, ×5 : Numbers refer to Sensitivity

15 20 10 5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION



SECTION A-A



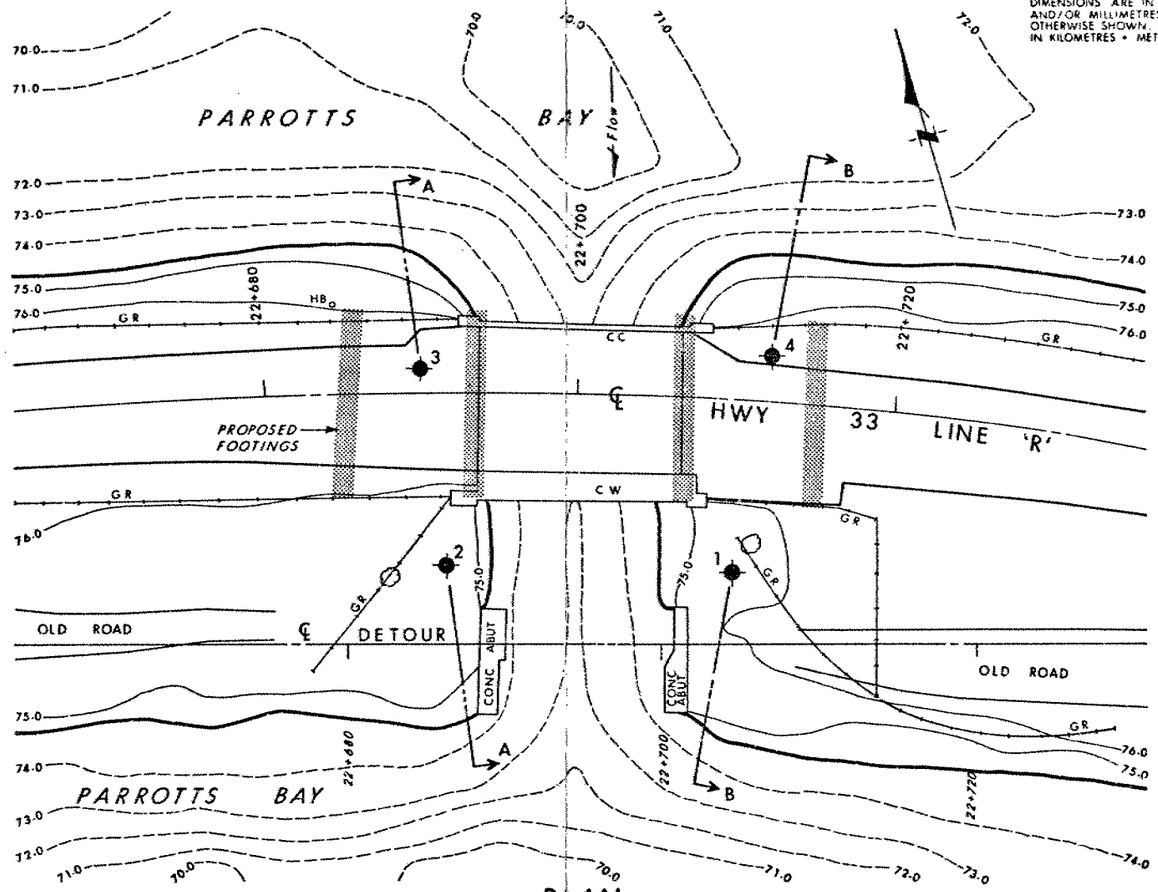
SECTION B-B

SCALE
4m 2 0 4m

SOIL STRATIGRAPHY LEGEND

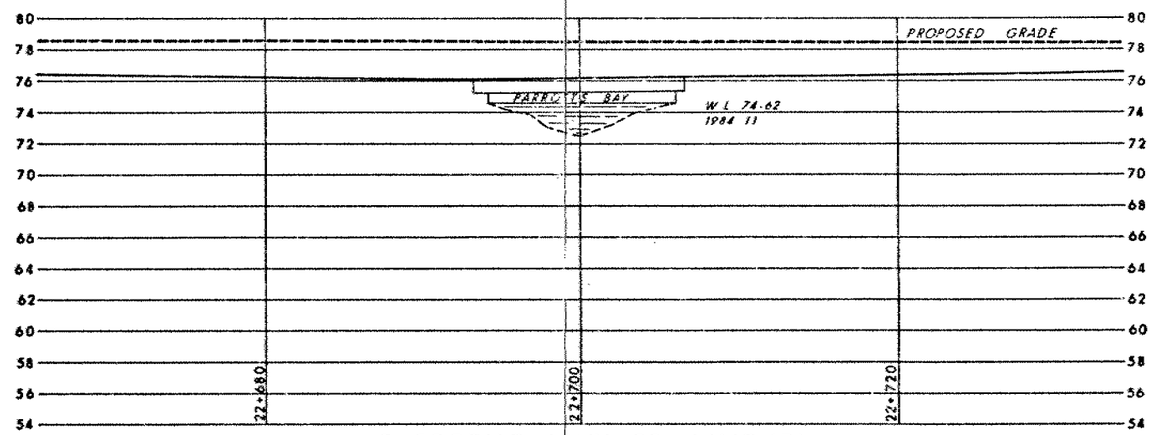
BOULDERS, GRAVEL & SAND
TRACE OF SILT, TRACE OF CLAY
OCC ORGANIC ZONES
Loose to Very Dense

SILTY SAND TO SANDY SILT
TRACE / WITH ORGANICS
TRACE / SOME CLAY, OCC GRAVEL ZONES
OCC BOULDERS (Slightly Cohesive)
Very Loose to Very Dense



PLAN

SCALE
4m 2 0 4m



℄ PROFILE HWY 33 LINE 'R'

SCALE
4m 2 0 4m

REF No E-7013-1; 1984 12

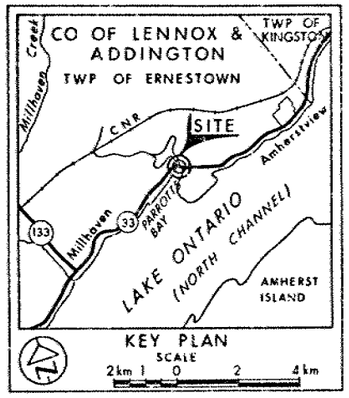
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No
WP No 25-77-04

PARROTTS BAY BRIDGE

BORE HOLE LOCATIONS & SOIL STRATA

SHEET



KEY PLAN
SCALE
2km 1 0 2 4km

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)
- WL at time of investigation 1985 08 and 09

No	ELEVATION	STATION	OFFSET
1	75.1	22+710.4	11.0m Rt
2	75.5	22+691.4	11.0m Rt
3	76.2	22+689.8	1.5m Lt
4	76.3	22+712.1	2.9m Lt

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV.	DATE	BY	DESCRIPTION

Geocres No 31C-142

HWY No 33			DIST 9
SUBMD D D	CHECKED	DATE 1985 12 30	SITE 17-103
DRAWN S	CHECKED	APPROVED	DWG 257704-A

memorandum

Ontario

Tel: (416) 235-3731
Date: 1987 08 07

To: E.C. Lane
Head, Structural Section
Eastern Region

Attention: Q.M. Islam

From: Foundation Design Section
Rm. 315, Central Region

RE: W.P. 25-77-04, Site 17-103
Parrott's Bay Bridge
Highway 33, District 8, Kingston

We have reviewed the final drawings and Special Provisions for the above-noted project. Our comments are as follows:

1) We have not been provided with information pertaining to the detour and therefore can not comment on detour foundations.

2) It is our understanding that, due to concerns of the Regional Maintenance office, the approach embankments will not be preloaded. In this case, some total and differential settlements should be expected in the approach embankments.

3) The estimated pile lengths for the West Footing in the pile data chart on Dwg. #3 of the Contract drawings should be reviewed. A stratigraphical profile illustrating approximate bedrock elevations at the west footing is provided in Section A-A on the Borehole Location and Soil Strata Drawing in the Foundation Report. At BH #3, near the north end of the footing the bedrock elevation is estimated at 65±m while at the south end of the footing, bedrock is estimated at elev. 62±m.

4) It is noted that the pile caps are below the lake level and therefore dewatering will be required for construction in the dry.

If there are any questions, please contact this office.

D.H. Dundas,

D.H. Dundas, P. Eng.
Sr. Foundations Engineer

DHD/pb

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

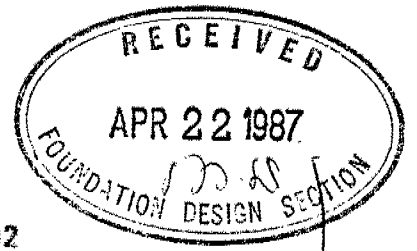
Copy for the information of

545-4712

E. Lin
Design Engineer
Design Section
Structural Office
Downsview, Ontario

April 16, 1987

From: Structural Section
Eastern Region
Kingston, Ontario



D.D
→ File

Re: K.P 25-77-03, -04 and -05
Millhaven Creek Bridge, site 17-102
Parrott's Bay Bridge, site 17-103
Collins Creek Bridge, site 7-53
Highway 33, District 8 - Kingston
Loyalist Parkway Aesthetic Consideration

Further to my memo of March 23, 1987, to R.W. Franks, a meeting was held on April 3, 1987, in the office of R.C. Wycliffe, District Engineer, District 8, to finalize the above noted aesthetic requirements. The meeting was attended by R.C. Wycliffe, E.C. Lane, A. Hall, Ted Phillips and the undersigned.

It was resolved that the limestone endwall treatment as presented by the Structural Section earlier would be withheld so that it does not impose limitations on the development of overall treatments for the hard landscape elements of the Loyalist Parkway.

It was decided that the vertical fluting (groove) should be provided on the outside face of the concrete barrier walls, but the exposed face of the wingwalls should receive a rough textured finish instead of imitation limestone finish as detailed by the Structural Section earlier.

It was agreed that the specification for the barrier walls and the abutment walls would call for tinting of the concrete to simulate natural limestone colour.

It was also agreed that the rounding of pavement edge near the end of the barrier walls would have enough space for future endwall treatment. Therefore, the rounding of pavement edge shown earlier with the proposed stonewall details should be maintained.

The slope treatment as proposed earlier to be changed from rip-rap and sodding to rip-rap and rockfill.

It was agreed that the Planning and Design Section would look after the rounding of pavement edge near the end of barrier walls and the slope treatment with rip-rap and rockfill.

It was decided that a flat area parallel to the wingwalls would be provided along the side slope of the pavement to

provide the space for walkway up to the waterline. This area would be treated with flat stones. The details for the walkway would be finalized by the Planning and Design Section. This detail would have to be shown on the Structural General Arrangement Plans of all three structures when it is finalized.

I am sending you herewith the original tracings of the Loyalist Parkway Aesthetic Consideration for Millhaven Creek Bridge, site 17-102 and Collins Creek Bridge, site 7-53 for inclusion with your contract drawings. The Millhaven Creek Bridge drawing may have to be transferred by photograph to a proper size contract drawing mylar.

The fluting (groove) details shown on the outside face of the barrier walls has dictated the locations of barrier wall construction joints which are as shown on the attached drawings. The construction joint locations are different than what was submitted in your package to the region. Would you please make necessary revisions on your barrier wall drawings resulting from the relocation of construction joints.

We have not finalized the rough texture finish pattern for the exposed faces of the wingwalls yet. However, this detail could be specified in the non-standard special provision when it is finalized.

The Region will look after the similar details and revisions for Parrott's Bay structure.

Q.M. Islam
Q.M. Islam
Senior Structural Engineer

cc R.C. Wycliffe
cc J.W. Reid (Attn: B. Tarini)
cc Ted Phillips
cc S. Ng
cc M. Devata

QMI:tbm

memorandum

Ontario

Tel: 3731

To: Q.M. Islam
Sr. Structural Engineer
Structural Section
Kingston

Date: 1987 04 01

From: Foundation Design Section
Room 315, Central Building

RE: W.P. 25-77-03/04/05
Millhave Creek Bridge, Site 17-102
Parrott's Bay Bridge, Site 17-103
Collins Creek Bridge, Site 7-53
Hwy. 33, District 8, Kingston

Further to your memo dated March 23, 1987, and our telephone conversation of March 31, 1987, following are our recommendations for foundations for the proposed 1 m high above-noted structures:

1) Provided that some differential settlements can be tolerated, especially between the parapet wall and the abutments, the proposed parapet walls may be founded on spread footings.

2) The base of the spread footings should be founded below the depth of frost penetration (1.5 m) on 1 m thick pads of engineered fill.

3) The engineered fill should extend down at a 1 H : 1 V slope from the edge of the footing

4) The engineered fill should consist of:

   a) compacted Granular 'A'

or b) rock fill (with maximum dimension less than 150 mm) if rock fill has been used as backfill to the abutments, and for the immediate approaches.

If there are any questions, please contact this office.

D.H. Dundas

D.H. Dundas, P. Eng.
Sr. Foundations Engineer

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

Copy for the information of

545-4712

R.W. Franks, Manager
Engineering & R.O.W. Office
Kingston, Ontario

March 23, 1987

From: Structural Section
Kingston, Ontario

Re: W.P. 25-77-03, -04 and -05
Millhaven Creek Bridge, Site 17-102
Parrott's Bay Bridge, Site 17-103
Collins Creek Bridge, Site 7-53
Highway 33, District 8 - Kingston
Loyalist Parkway Aesthetic Consideration

Further to our meeting of March 12th, 1987, in your office with R.C. Wycliffe, Ted Phillips and S.N. Chen, we have prepared drawings for the above noted structures showing the details of limestone end walls and the grooves on the outside face of concrete wingwalls and barrier walls. All the dimensions were confirmed by S. Ng and Ted Phillips in another meeting on March 17th, 1987, in E.C. Lane's office.

It was agreed in our first meeting that the limestone end walls for the above noted structures, and some other structures on Highway 33, would be constructed under a separate contract after completion of the first contract for the construction of the above mentioned structures. The site for the limestone end walls for other structures will be determined at a later date.

It was also agreed that we would try to incorporate the necessary concrete foundation work for the end stone walls, for the above noted three structure, with the first contract. It was mentioned that the area in front of all wingwalls would be sodded and the toe of the slope treated with rip-rap. It was further agreed that we would try to match the bluish hue of natural limestone by tinting the concrete for the wingwalls and barrier walls.

We are sending you herewith one copy each of the drawings showing the above mentioned details for your review and comments. We would appreciate your comments by March 31, 1987.

.../2

For your consideration.

Q.M. Islam
Q.M. Islam
Sr. Structural Engineer

QMI:bd

Encl.

c.c. R.C. Wycliffe
Ted Phillips
J.W. Reid (Att'n: B. Tarini)
S. Ng. (Landscape Planning Unit)
✓M. Devata (For Foundation Recommendation)
W. Lin

memorandum

Ontario

Tel: 3731

To: E.C. Lane
Head, Structural Section
Kingston

Date: 1987 03 10

Atten: Q. Islam

From: Foundation Design Section
Room 315, Central Building

RE: W.P. 25-77-03, Millhaven Creek
✓ W.P. 25-77-04, Parrotts Bay
W.P. 25-77-05, Collins Creek
Hwy. 33, District 8 - Kingston

Further to your recent discussions with M. Devata, we have reviewed the foundation requirements for the above-noted three structures. We are satisfied with the recommendations given for Millhaven Creek and Collins Creek.

For Parrotts Bay, we are still of the opinion that the use of tube piles as discussed in the Foundation Report, is the most feasible solution. However, in order to facilitate the installation, the design should involve 450 mm diameter concrete-filled tube piles equipped with driving shoes as per M.T.C. standards. For such a diameter, loads of 2200 kN/pile at the U.L.S. and 1500 kN/pile at the S.L.S. II could be used.

If you require additional information, please do not hesitate to contact this Section.

L. Politano
Project Foundations Engineer

for

M. Devata
Chief Foundations Engineer
(East)

LP/MD/mmj

E.C. Lin

memorandum

Ontario

Tel: 3282

To: E.C. Lane
Head, Structural Section
Kingston

Date: 1987 01 15

From: Foundation Design Section
Central Building

RE: Parrott's Bay Bridge
W.P. 25-77-04; Site 17-103
Hwy. 33, District 8, Kingston

We have reviewed Preliminary General Arrangement Drawing No. 17-103-P1 (dated Dec. 86) for the above-noted structure, and make the following comments:

1. Please note that we recommend that the tube piles be installed open-ended, as a combination of driving and drilling will be necessary to advance through the bouldery zone. When required, the drilling can be carried out through the tube pile. When the piles have been seated on the bedrock, they should be cleaned out and filled with concrete placed in the dry (after dewatering the liner) or by tremie methods.

2. Since the excavation for the construction of the pile caps will be carried out below the prevailing groundwater level and in a non-cohesive material, de-watering will be required.

3. Erosion protection should extend to a minimum of 10 m both north and south of the structures.

4. We note that the final slope will be 1.5H:1V. This leads us to believe that a rock fill is to be used. However, if an earth fill is planned, the slopes should be 2H:1V or flatter.

5. Please note that we recommend preloading and surcharging of the approach embankments between Sta. 22 + 660 and Sta. 22 + 740 for a minimum period of six months, in order to minimize the effects of settlements. The surcharge should be 1 m above the final grade and extend over the plan limits of the proposed embankments.

6. To facilitate pile driving, particle sizes in the fill immediately beneath the pile locations should not exceed 50 mm for steel tube piles.

7. Since no details of the proposed temporary detour have been provided to us, we cannot make any comments at this time.

Igo Steblynsky

I. Steblynsky
Project Foundations Engineer

for

M. Devata
Chief Foundations Engineer
(East)

7540-1318 (10/78) IS/MD/mmj