DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 41J-55

DIST. 18 REGION

W.P. No. 29-88-01

CONT. No. 94-228

W. O. No.

STR. SITE No. 38S-177

HWY. No. 548

LOCATION Munroe Island Bridge

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

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

DIST. No. 18
CONT. No. 94-228
WP. No. 29-88-01

MUNROE ISLAND BRIDGE
HIGHWAY 548

GENERAL ARRANGEMENT

SHEET 21

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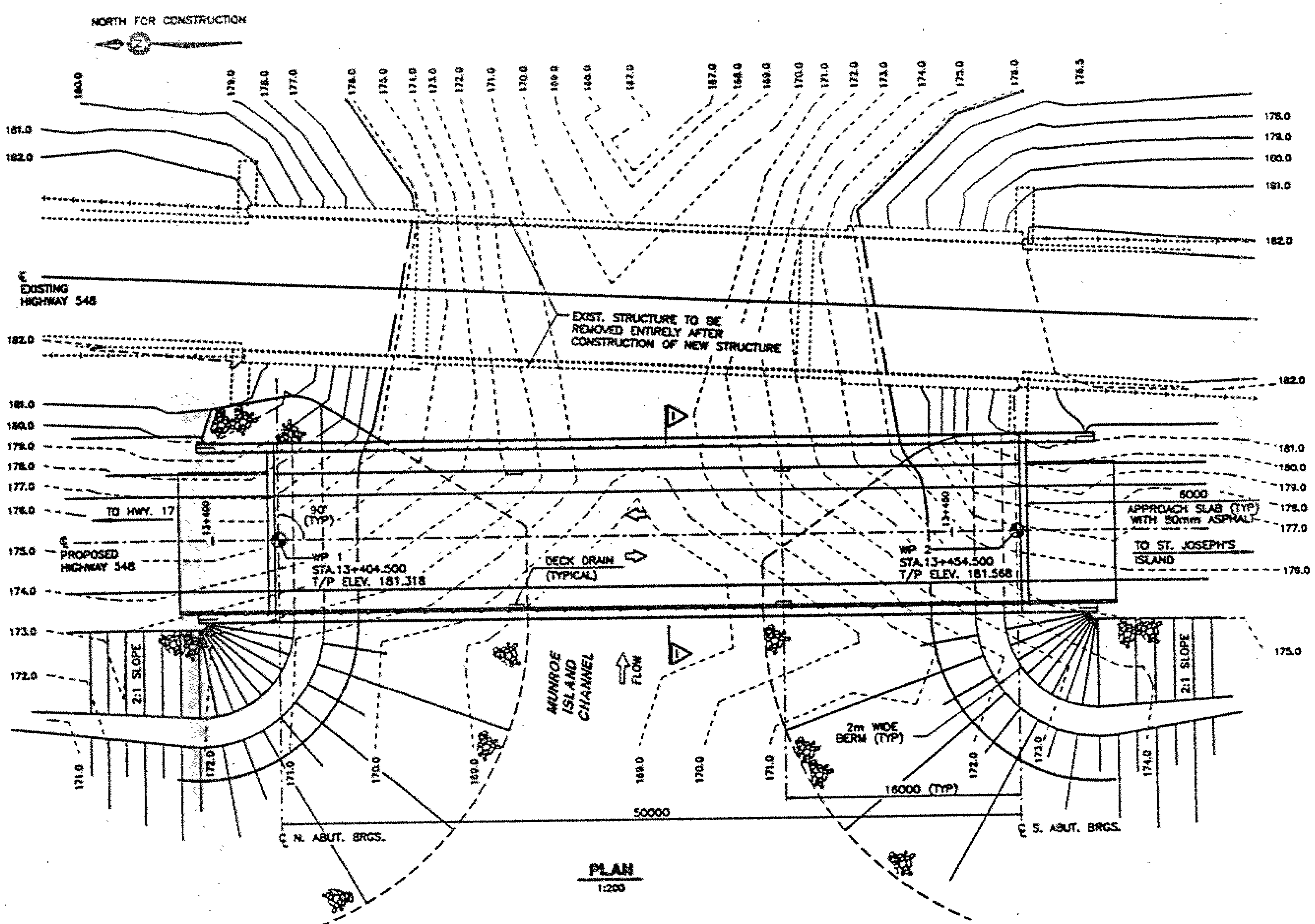
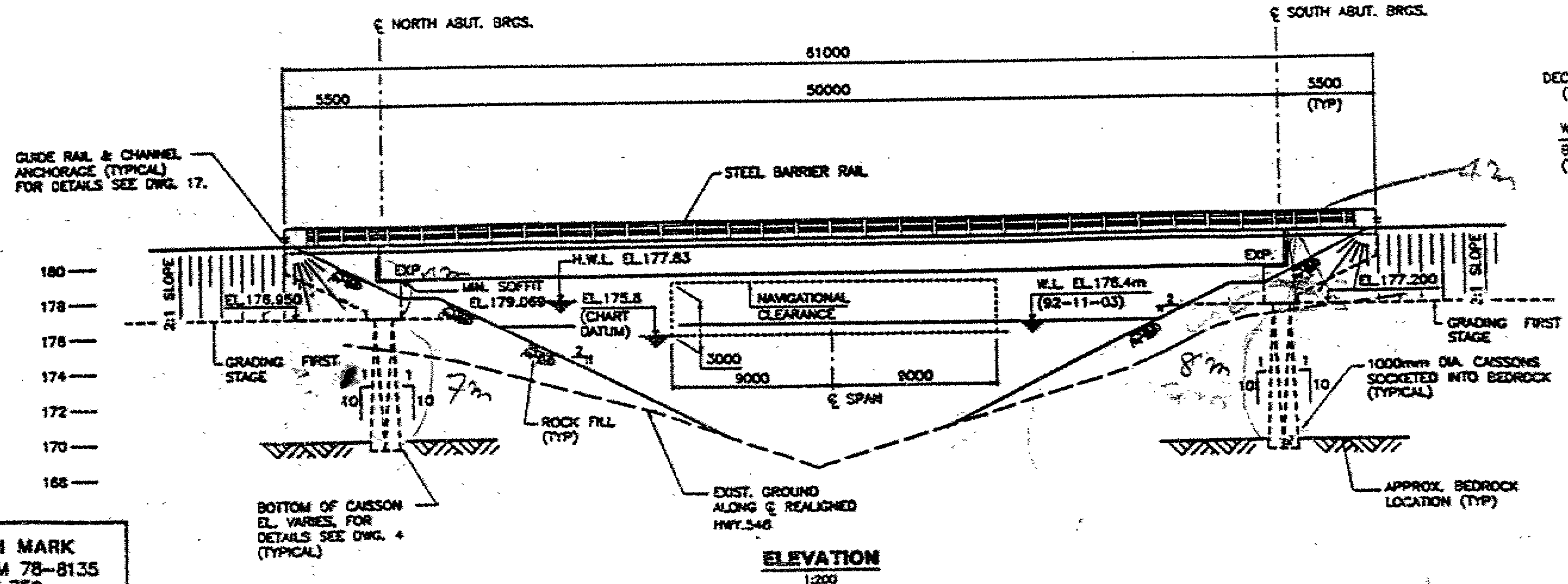


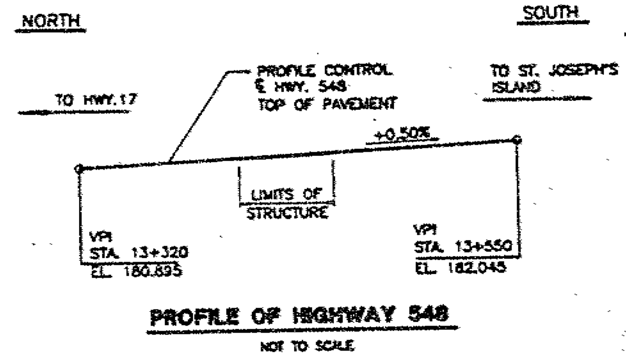
PLAN
1:200



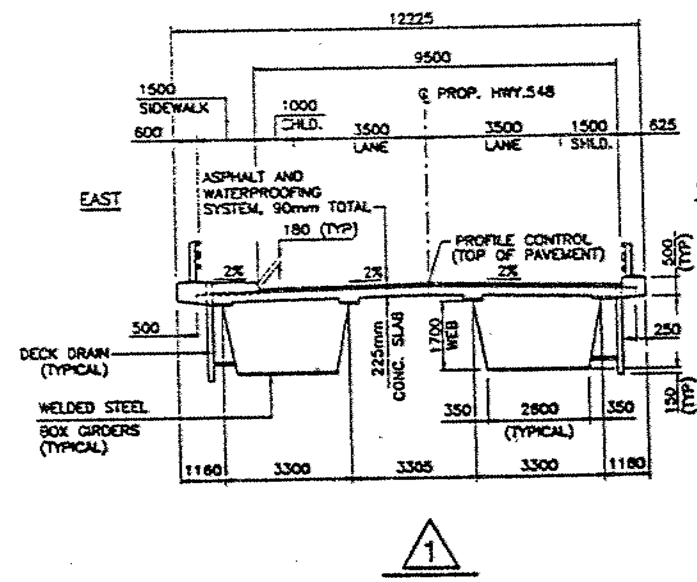
ELEVATION
1:200

CONSTRUCTION NOTES :

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESS FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.



PROFILE OF HIGHWAY 548
NOT TO SCALE



1
1:100

GENERAL NOTES

CLASS OF CONCRETE
- ALL CONCRETE ---------- 30 MPa

REINFORCING STEEL
- REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED.
- BAR MARKS WITH SUFFIX "C" DENOTE COATED BARS.
- WHERE SPLICES ARE NOT SHOWN ON THE DRAWINGS, SPLICES SHALL BE AT LEAST
  15M – 550mm
  20M – 800mm
  25M – 1200mm
  30M – 1650mm
  35M – 2350mm

CLEAR COVER TO REINFORCING STEEL
- CAISSONS ---------- 80 ± 20mm
- ABUTMENTS AND WINGWALLS — FRONT FACE — 80 ± 20mm; BACK FACE — 70 ± 20mm
- DECK — TOP — 70 ± 20mm; BOTTOM — 40 ± 10mm
- REMAINDER UNLESS OTHERWISE NOTED — 70 ± 20mm

LIST OF DRAWINGS
1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS & SOIL STRATA
3. ROADWAY PROTECTION
4. CAISSON LAYOUT
5. NORTH ABUTMENT
6. SOUTH ABUTMENT
7. ABUTMENT DETAILS
8. WINGWALLS
9. STRUCTURAL STEEL I
10. STRUCTURAL STEEL II
11. STRUCTURAL STEEL III
12. DECK LAYOUT
13. DECK REINFORCEMENT
14. STEEL BARRIER RAIL DETAILS
15. 5000mm APPROACH SLAB
16. JOINT ANCHORAGE & ARMOURING
17. STANDARD DETAILS
18. QUANTITIES – STRUCTURE I
19. QUANTITIES – STRUCTURE II

LIST OF ABBREVIATIONS
- WP — DENOTES WORKING POINT
- T/P — DENOTES TOP OF PAVEMENT
- W.L. — DENOTES WATER LEVEL
- H.W.L. — DENOTES HIGH WATER LEVEL

BENCH MARK
MTO BM 78-8135
EL. 181.752

M. RADOLLI 94-10-3
P. K-H. TAM 94-10-3

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS
DATE	BY	DESCRIPTION

DESIGN M.R. | CHK. P.T. | CODE OHBDC-91 | LOAD | DATE AUG 1994

METRIC
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CONT. No. 94-228
WP. No. 29-88-01

MUNROE ISLAND BRIDGE
HIGHWAY 548

ROADWAY PROTECTION

SHEET 23

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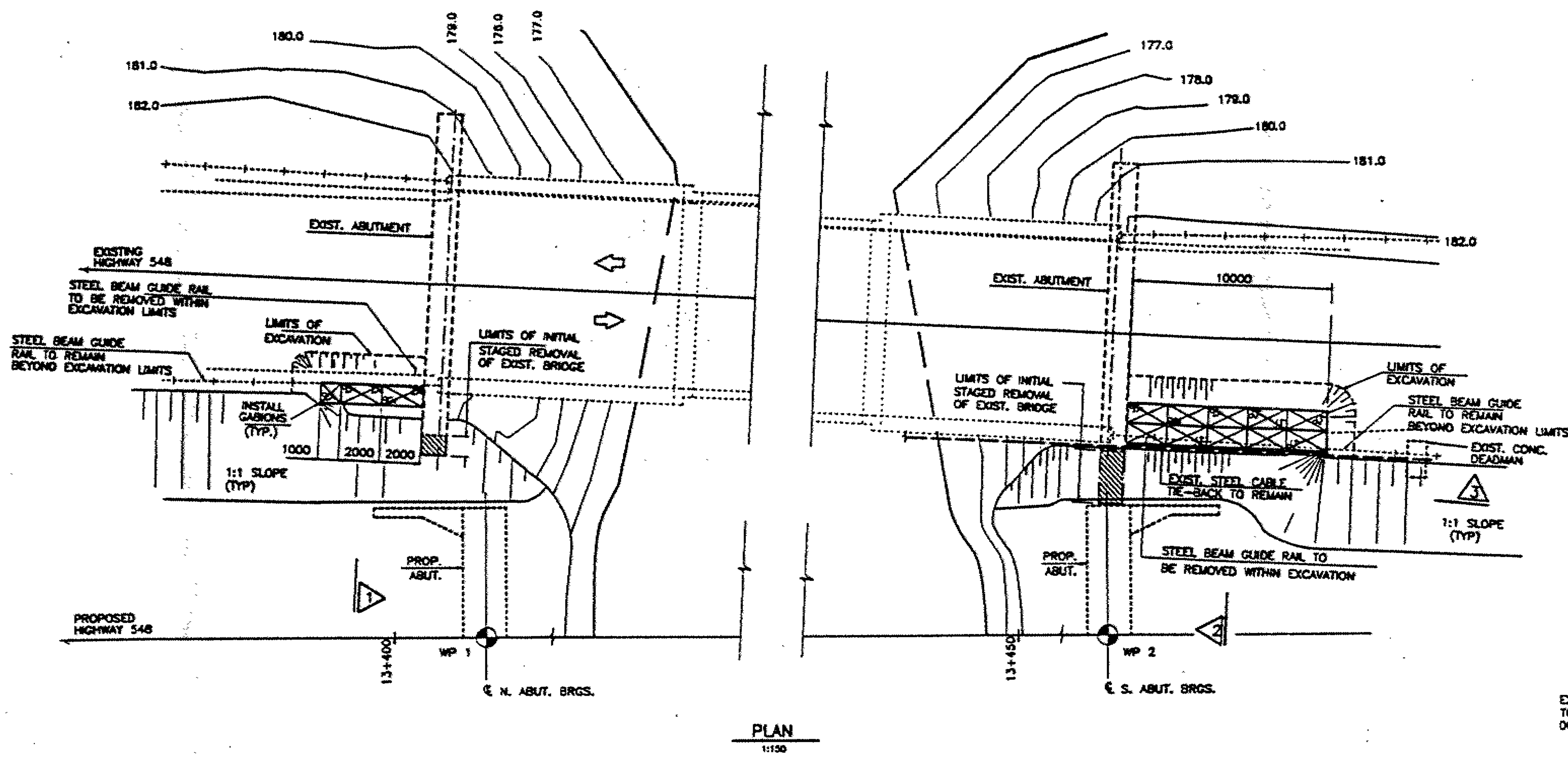
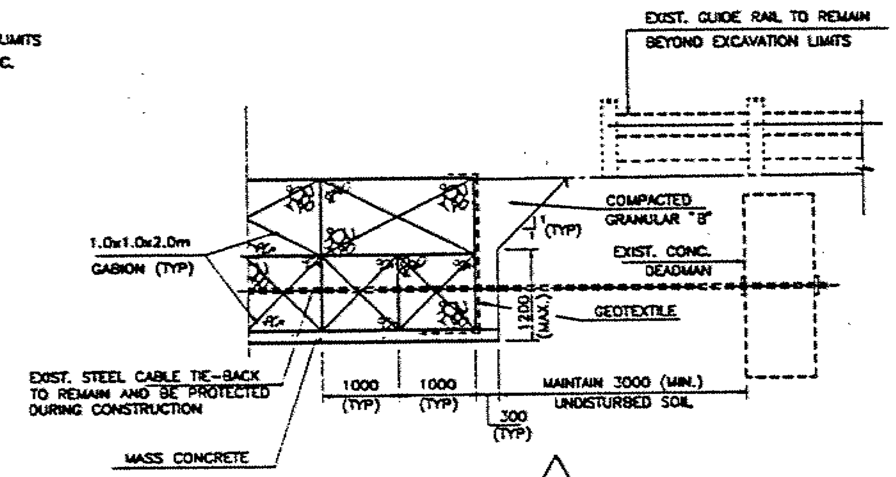
NOTES:

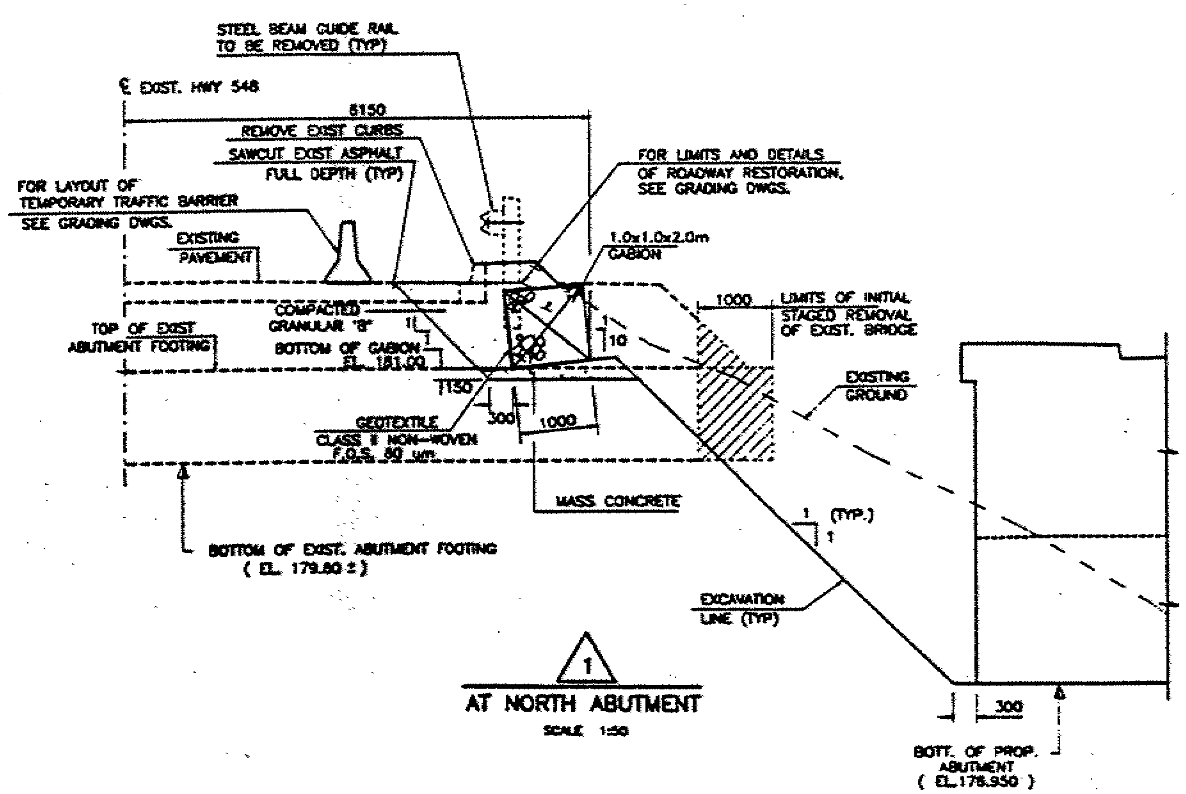
- FOR PAVEMENT DETAILS SEE GRADING DWGS.
- CONTRACTOR TO VERIFY EXISTING GRADE ELEVATIONS AND LOCATION OF STEEL CABLE TIE-BACK AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
- ROADWAY PROTECTION TO BE REMOVED TO 1000mm BELOW FINISHED GRADE AT COMPLETION OF CONSTRUCTION.
- MIN. CLEARANCE BETWEEN EXIST. STEEL CABLE TIE-BACK AND GABION STRUCTURE TO BE 50mm. DEFORM GABION BASKET LOCALLY IF NECESSARY.



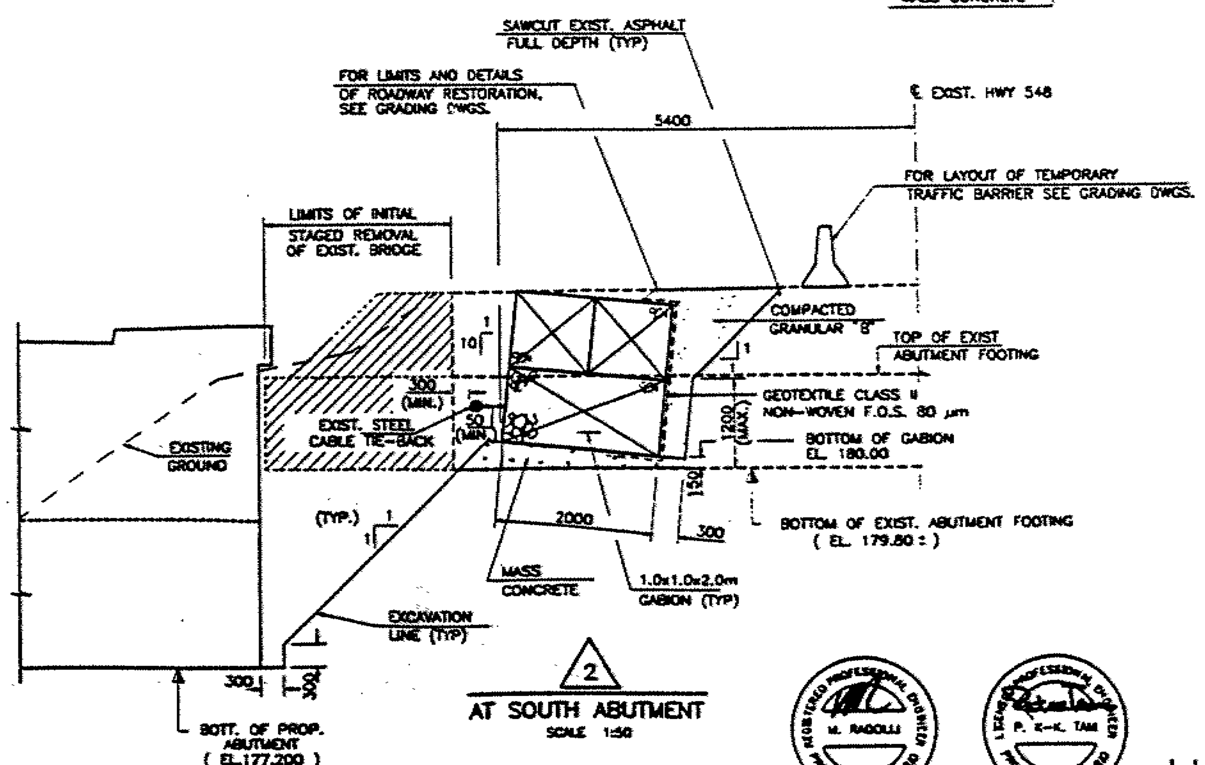
PLAN
1:150



3
SCALE 1:50



1
AT NORTH ABUTMENT
SCALE 1:50



2
AT SOUTH ABUTMENT
SCALE 1:50

REGISTERED PROFESSIONAL ENGINEER M. RAGOLLI PROVINCE OF ONTARIO
94-10-3

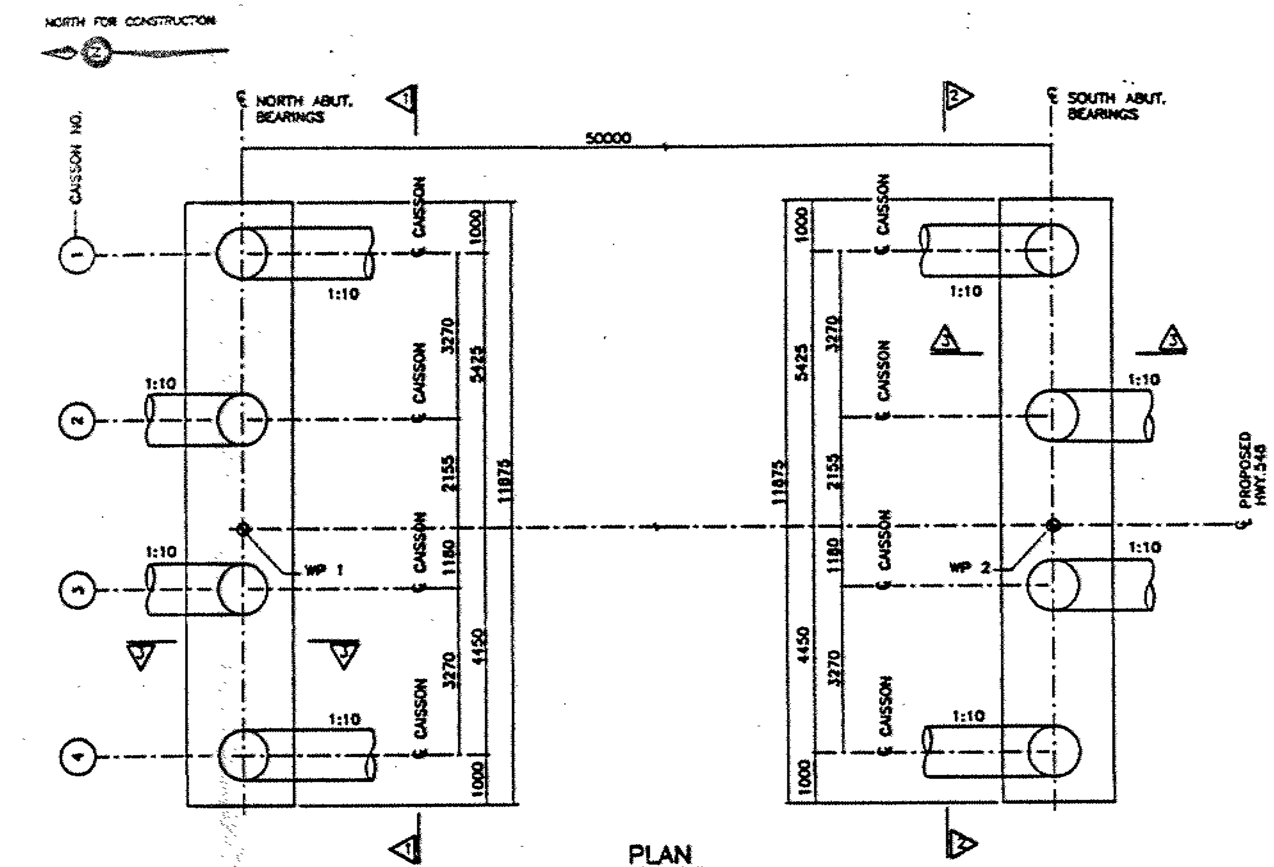
LICENSED PROFESSIONAL ENGINEER P. K.-K. TAM PROVINCE OF ONTARIO
94-10-3

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN P.T.	CHK. M.R.	CODE OHBDC-91	LOAD	DATE AUG. 1994	
DRAWN M.H.	CHK. P.T.	SITE 385-177	STRUCT.	SCHEME	DWG. 3

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

CONT. No. 94-228	
WP. No. 29-88-01	
MUNROE ISLAND BRIDGE HIGHWAY 548	SHEET 24
CAISSON LAYOUT	
DILLON Consulting Engineers • Planners Environmental Scientists	

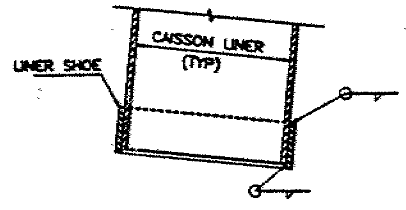


PLAN
SCALE 1:75

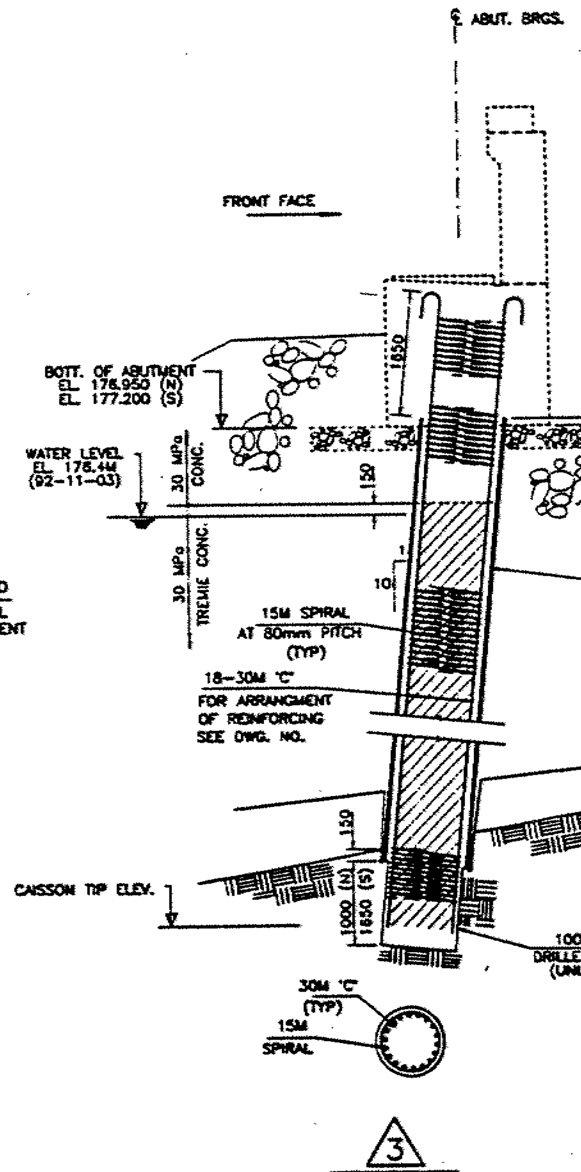
NOTE:
- CAISSON SPACING IS MEASURED AT UNDERSIDE OF ABUTMENT.

CAISSON DATA					
LOCATION	CAISSON NO.	CUT-OFF ELEVATION	ESTIMATED TIP ELEV.	LENGTH	BATTER
NORTH ABUTMENT	1	177.050	171.400	5700	1:10
	2	177.050	170.850	6250	1:10
	3	177.050	170.270	6850	1:10
	4	177.050	169.630	7450	1:10
SOUTH ABUTMENT	1	177.300	169.050	8300	1:10
	2	177.300	168.090	9250	1:10
	3	177.300	167.110	10250	1:10
	4	177.300	166.140	11200	1:10

MAX. FACTORED AXIAL DESIGN LOAD = 4000 KN

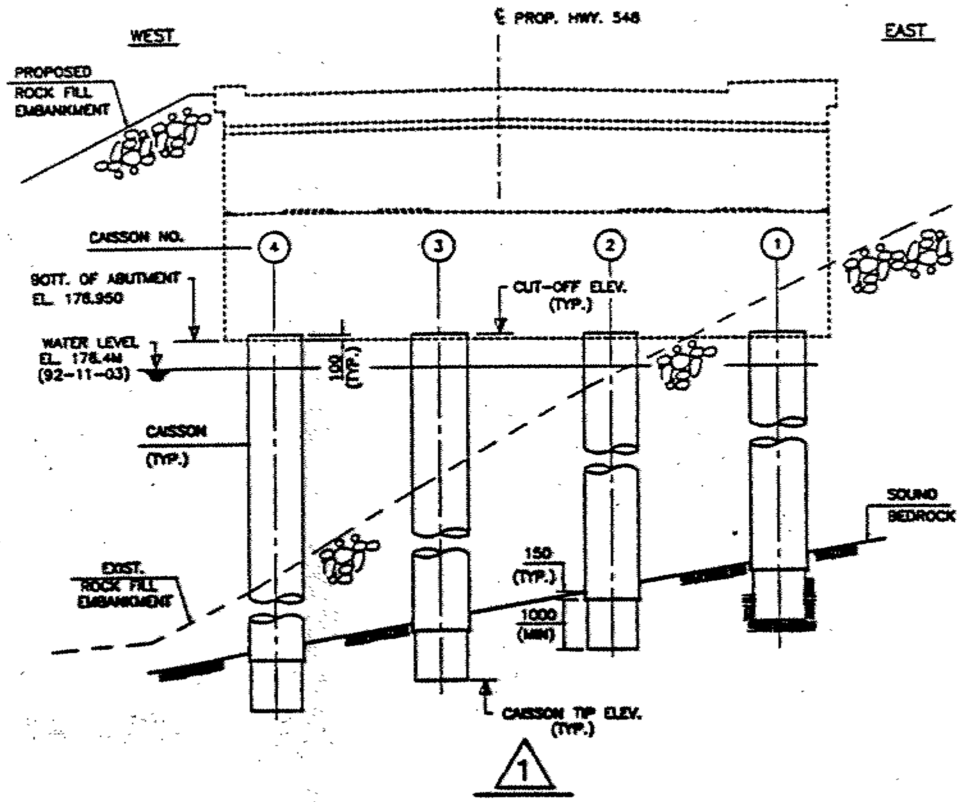


LINER SHOE DETAIL
SCALE 1:25

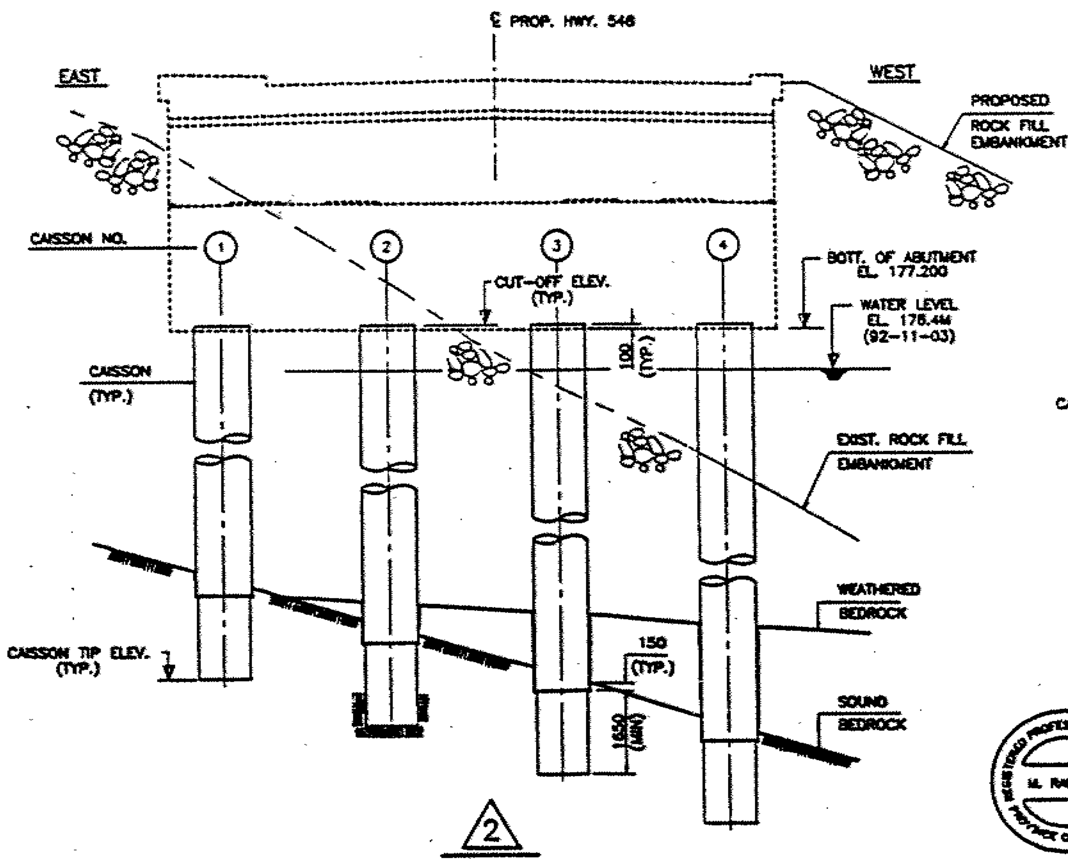


1, lack of what to support liner.
2, Straight vertical
3, [illegible]
(3) or place [illegible] of REN.
(3)

3
SCALE 1:50



1 NORTH ABUTMENT
SCALE 1:75



2 SOUTH ABUTMENT
SCALE 1:75

M. RADOLLI
P. K.-K. TAM
94-10-3

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION			
DESIGN P.T.	CHK. M.R.	CODE OHBDC-91	LOAD	DATE AUG. 1994		
DRAWN	CHK. P.T.	SITE 39S-177	STRUCT.	SCHEME	DWG. 4	

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WP. No. 29-88-01		
MUNROE ISLAND BRIDGE HIGHWAY 548		SHEET 25
NORTH ABUTMENT		

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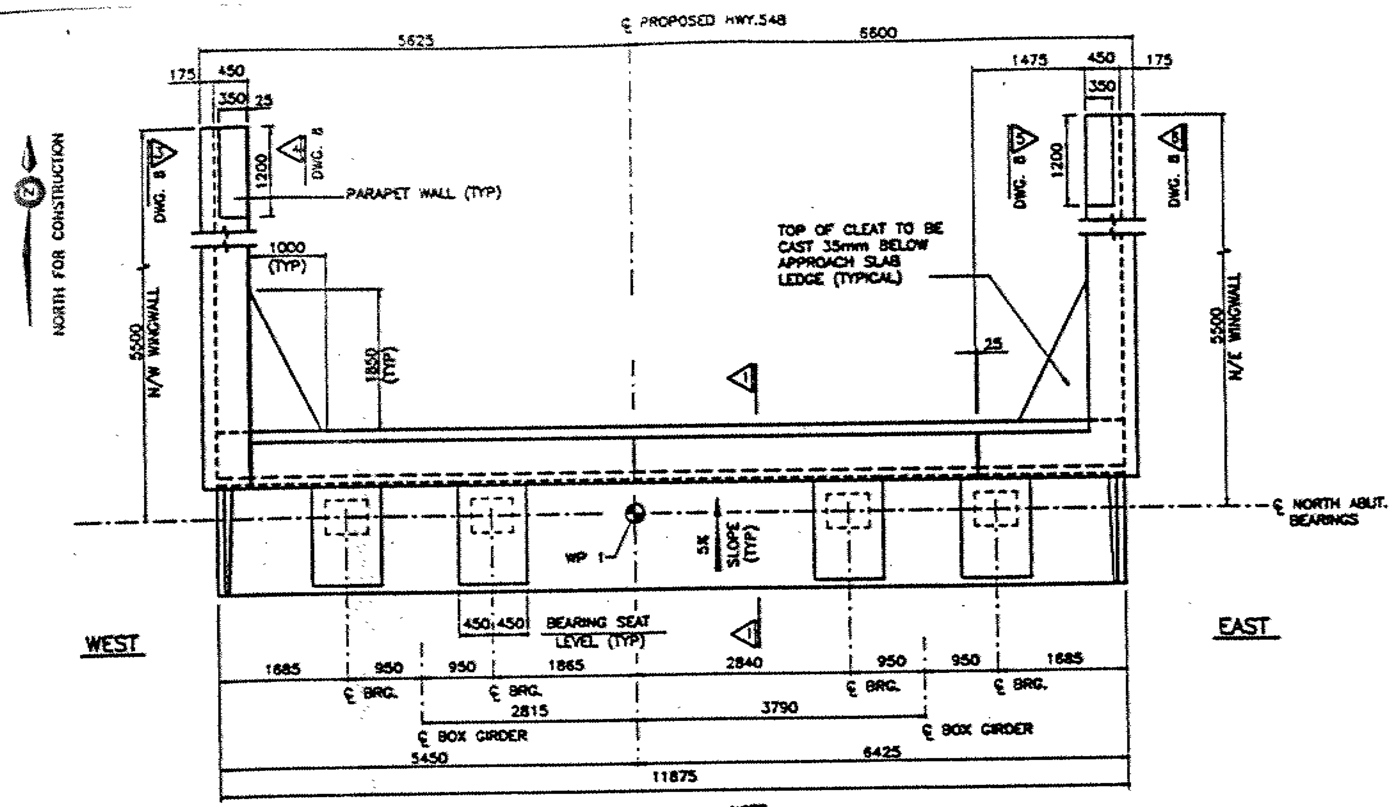
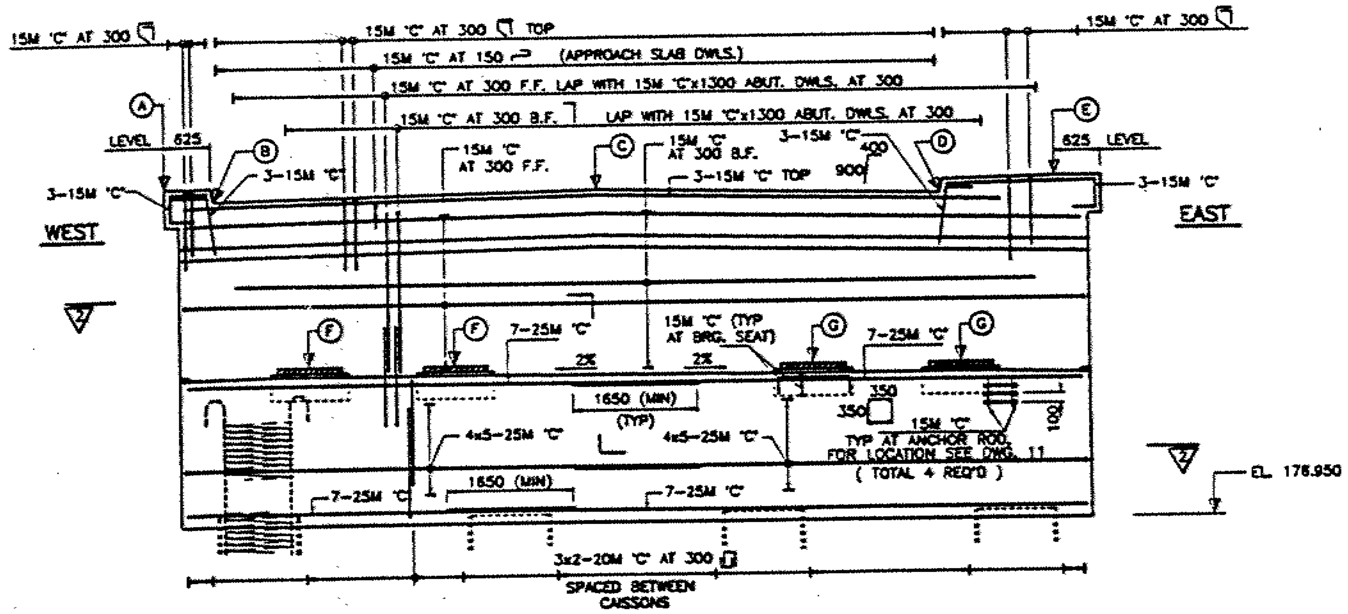
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NOTES:
- FOR BEARING DATA SEE DRAWING 11.
- PROVIDE 20mm CHAMFER AT ALL EXTERIOR CORNERS.
- F.F. DENOTES FRONT FACE.
- B.F. DENOTES BACK FACE.
- SEE DWG. 7 FOR DETAILS OF RETAINED SOIL SYSTEM AND ROCK FILL.

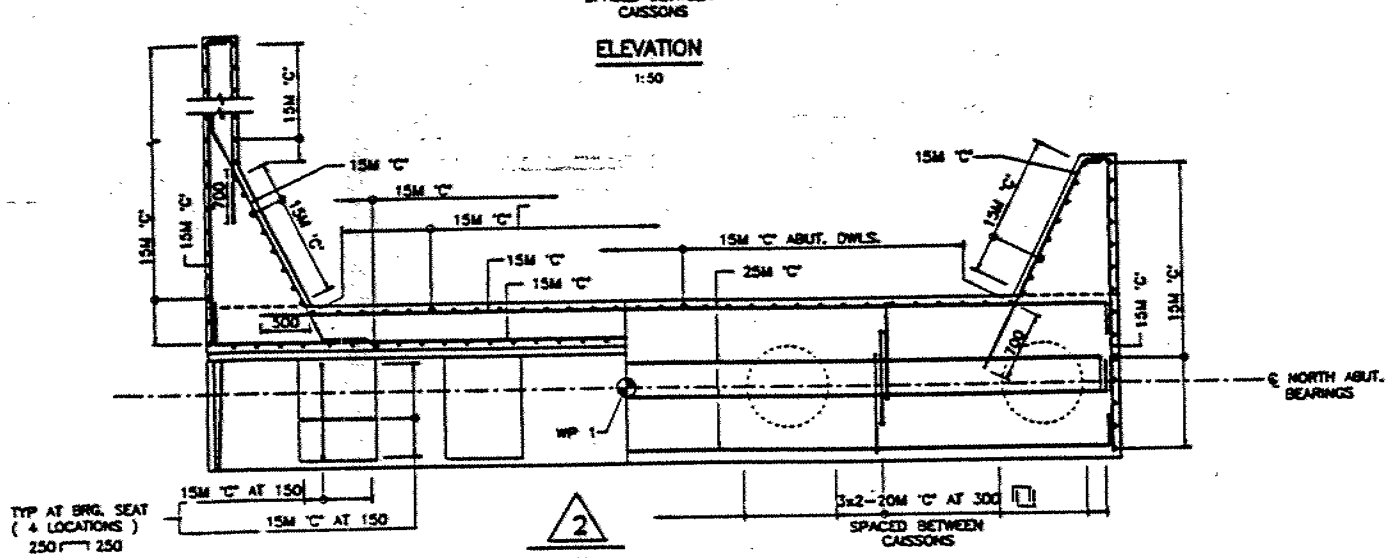


PLAN
1:50

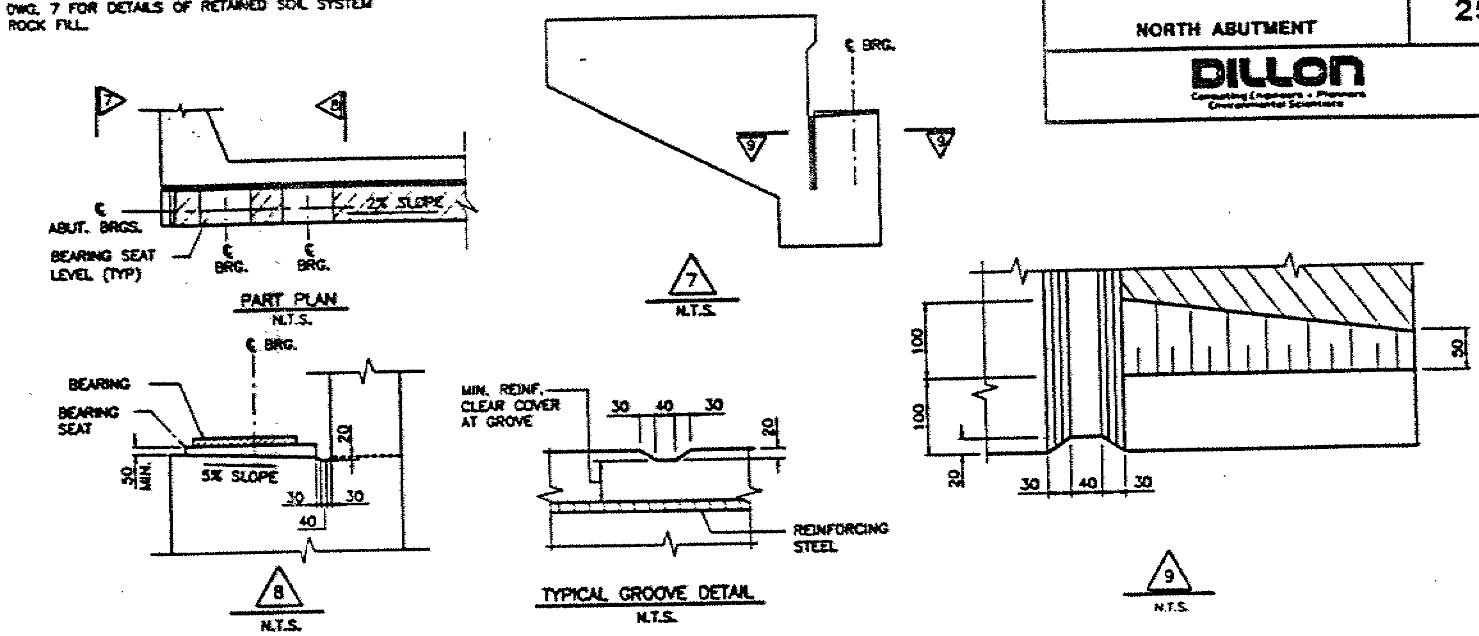
NOTE:
RETAINED SOIL SYSTEM NOT SHOWN FOR CLARITY



ELEVATION
1:50

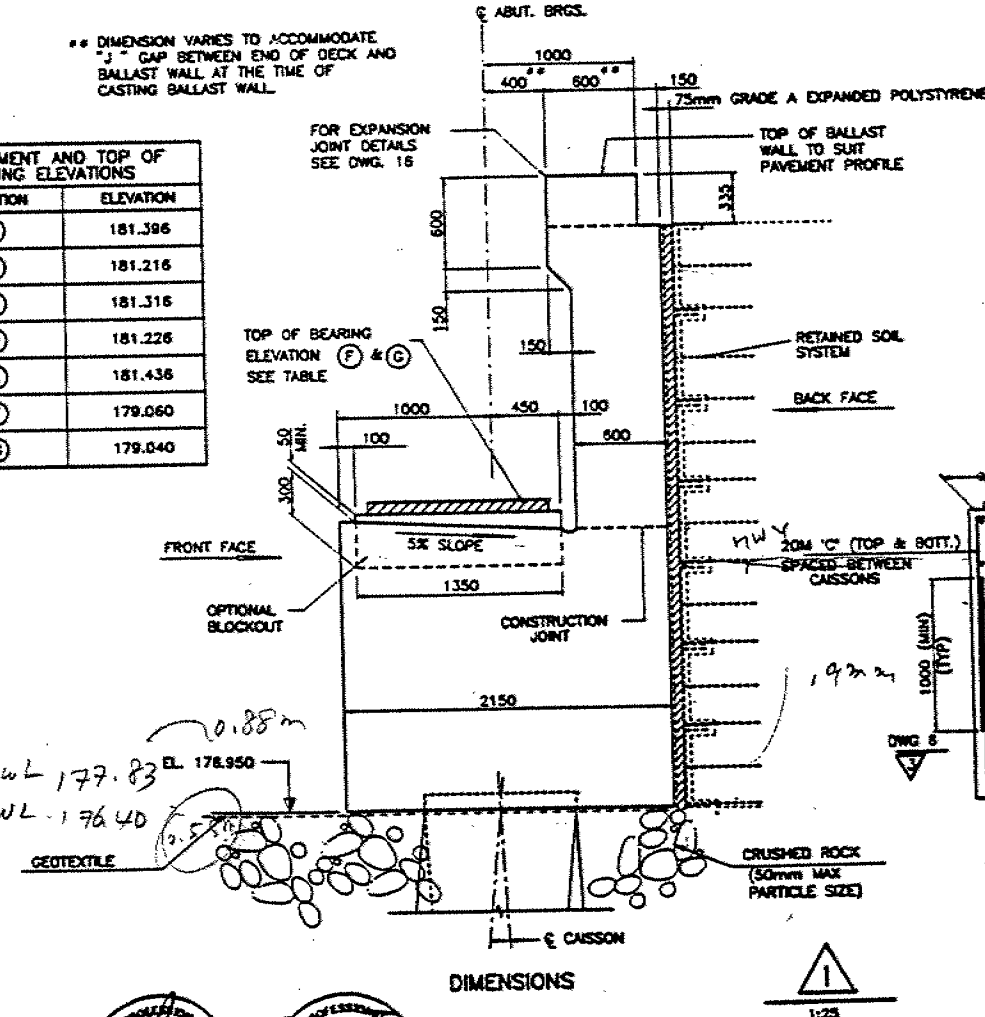


2
1:50



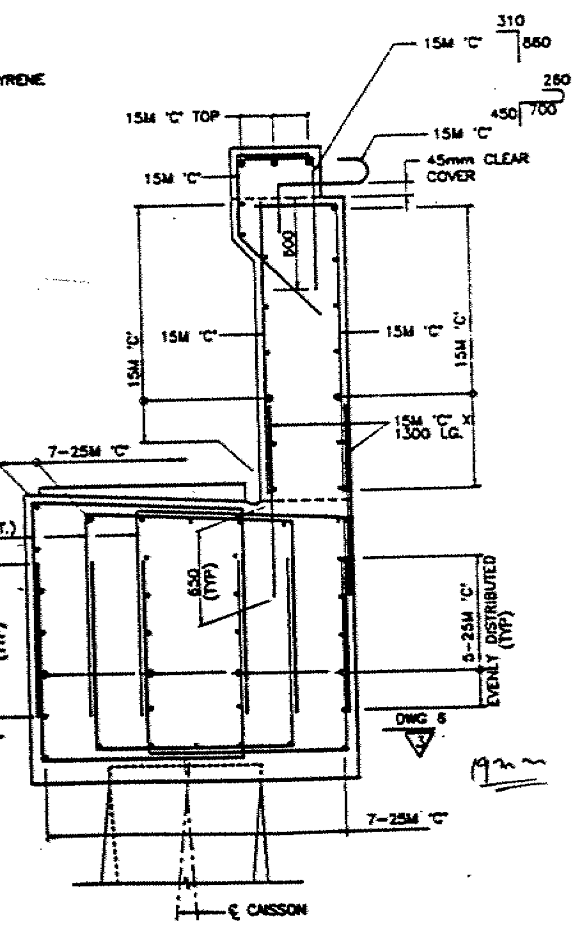
DETAILS OF WEATHERING STEEL RUST STAINS CONTROL AT ABUTMENT

ABUTMENT AND TOP OF BEARING ELEVATIONS	
LOCATION	ELEVATION
A	181.396
B	181.216
C	181.316
D	181.226
E	181.436
F	179.060
G	179.040



DIMENSIONS

1
1:25



REINFORCING STEEL

REGISTERED PROFESSIONAL ENGINEER M. RADOLLI PROVINCE OF ONTARIO 94-10-3

LICENSED PROFESSIONAL ENGINEER P. K-K. TAM PROVINCE OF ONTARIO 94-10-3

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN P.T.	CHK. M.R.	CODE OHBDC-91	LOAD	DATE AUG. 1994	
DRAWN O.C.	CHK. A.H.L	SITE 385-177	STRUCT.	SCHEME	DWG. 5

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

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MUNROE ISLAND BRIDGE
HIGHWAY 548
SOUTH ABUTMENT

SHEET 28

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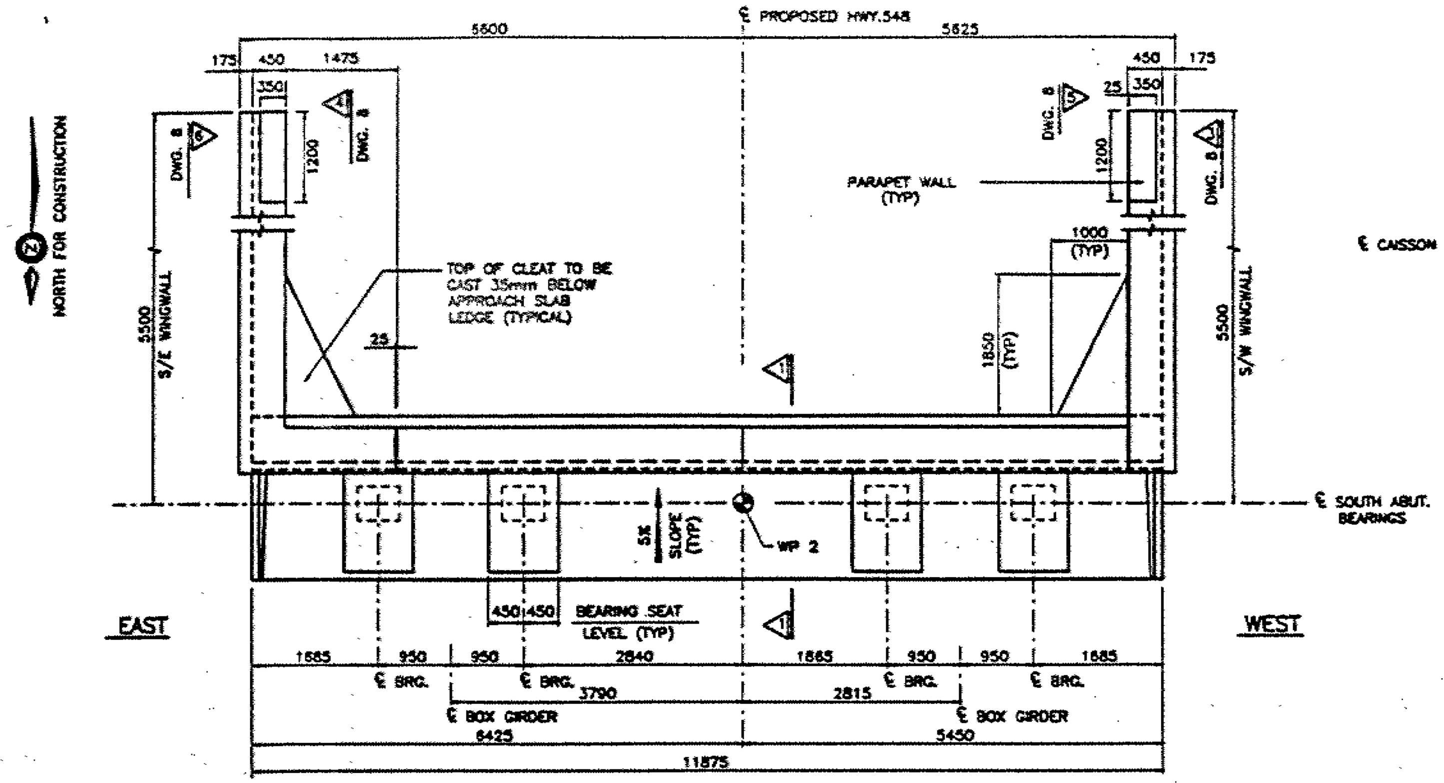
ABUTMENTS AND TOP OF BEARING ELEVATIONS

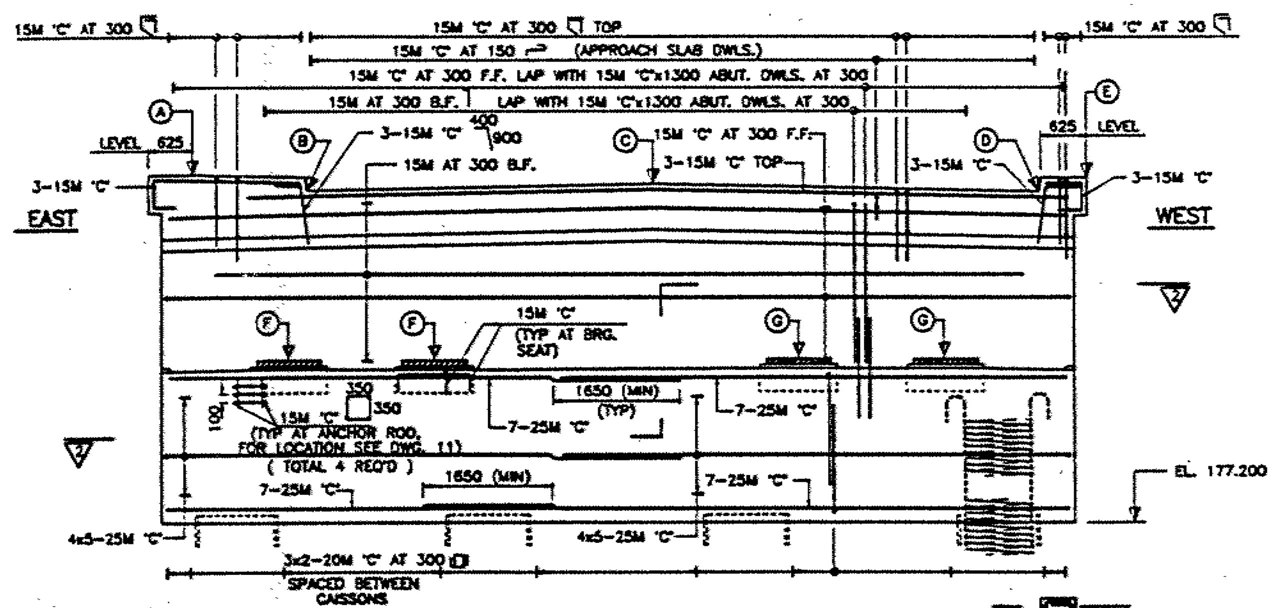
LOCATION	ELEVATION
A	181.690
B	181.480
C	181.570
D	181.470
E	181.650
F	179.258
G	179.308

NOTES:
- FOR BEARING DATA SEE DRAWING 11.
- PROVIDE 20mm CHAMFER AT ALL EXTERIOR CORNERS.
- F.F. DENOTES FRONT FACE.
- B.F. DENOTES BACK FACE.
- SEE DWG. 5 FOR DETAILS OF WEATHERING STEEL RUST STAINS CONTROL AT ABUTMENT.
- SEE DWG. 7 FOR DETAILS OF RETAINED SOIL SYSTEM AND ROCKFILL.

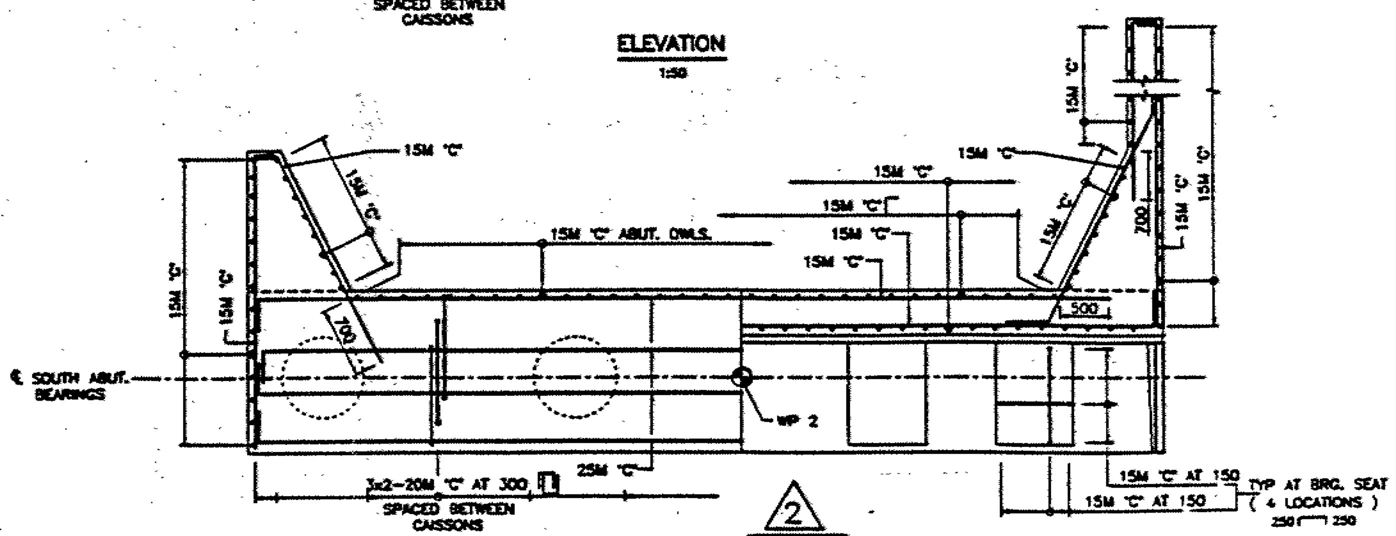


PLAN
1:50

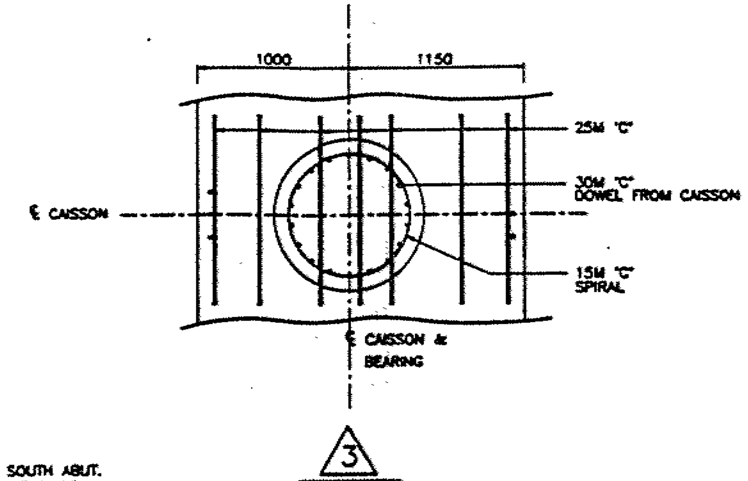
NOTE:
RETAINED SOIL SYSTEM NOT SHOWN FOR CLARITY.



ELEVATION
1:50

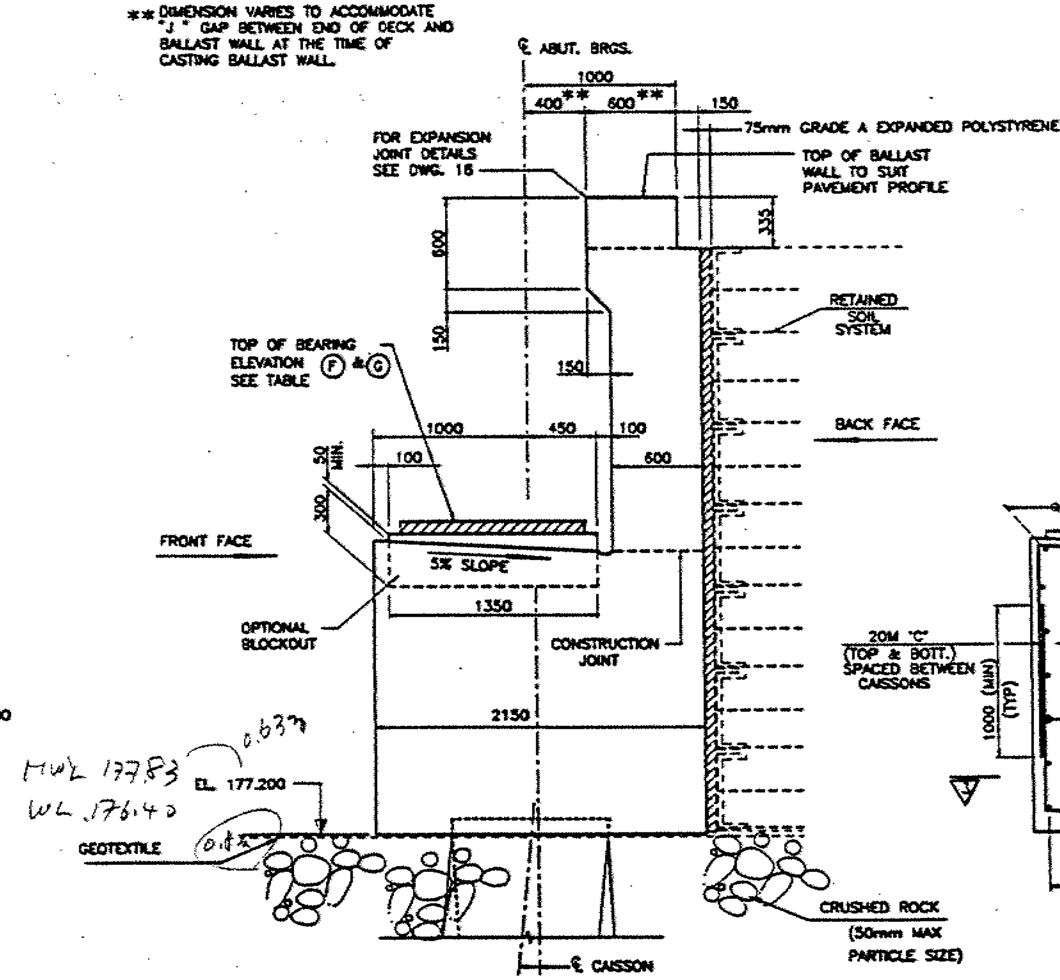


2
1:50

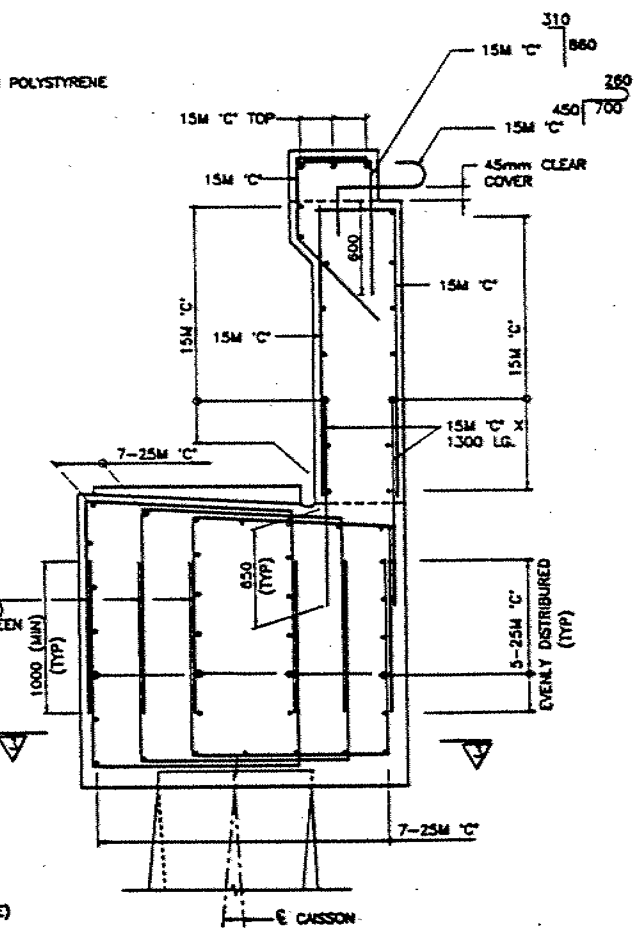


3
1:25

** DIMENSION VARIES TO ACCOMMODATE "J" GAP BETWEEN END OF DECK AND BALLAST WALL AT THE TIME OF CASTING BALLAST WALL.



DIMENSIONS



REINFORCING STEEL

1
1:25

M. RADOLLI 94-10-3
P. K.-K. TAM 94-10-3

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN P.T.	CHK. M.R.	CODE OHBDC-91	LOAD	DATE AUG. 1994	
DRAWN D.C.	CHK. A.K.L.	SITE 385-177	STRUCT.	SCHEME	DWG. 5

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

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WP. No. 29-88-01

MUNROE ISLAND BRIDGE
HIGHWAY 548

ABUTMENT DETAILS

SHEET 27

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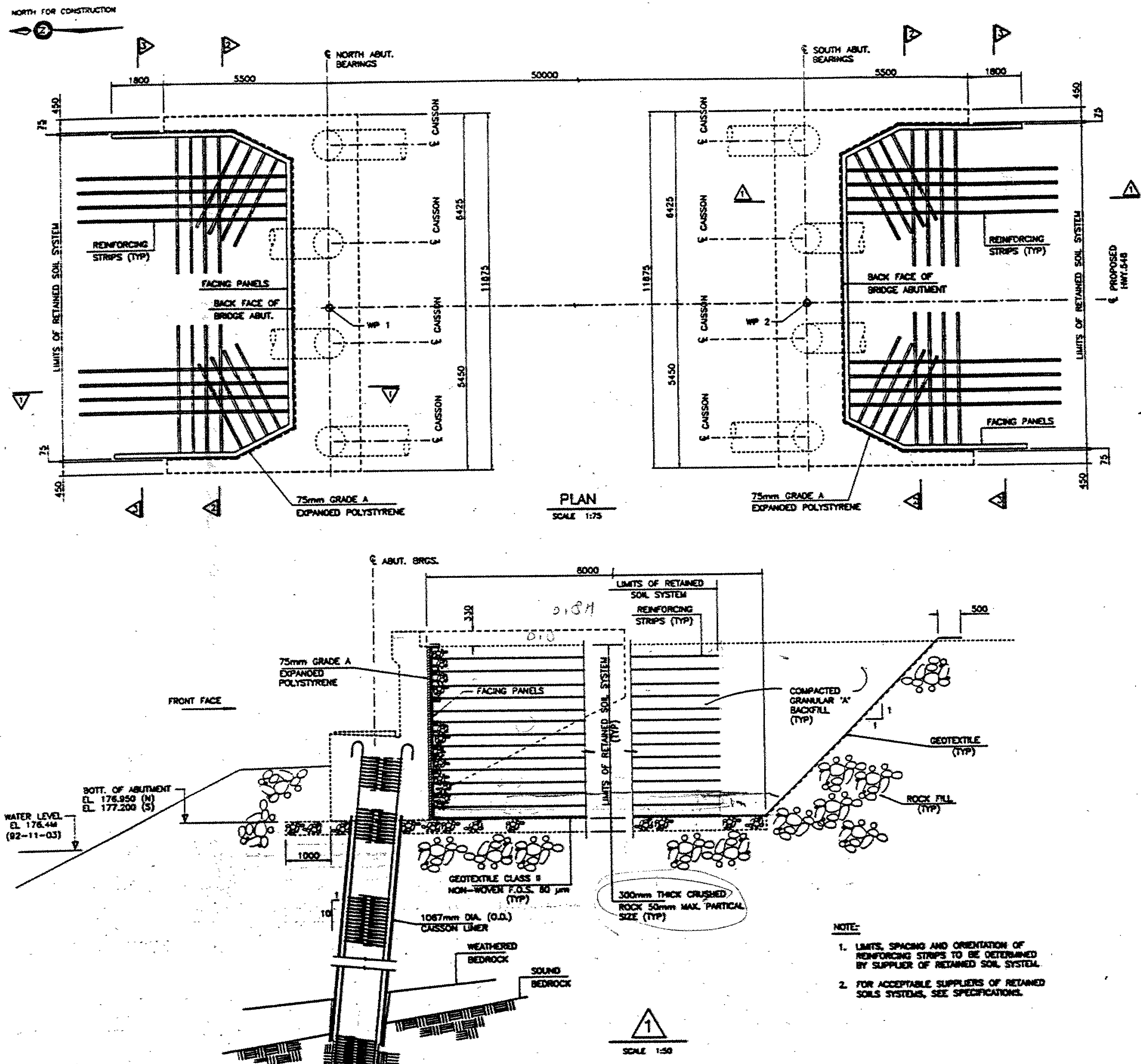
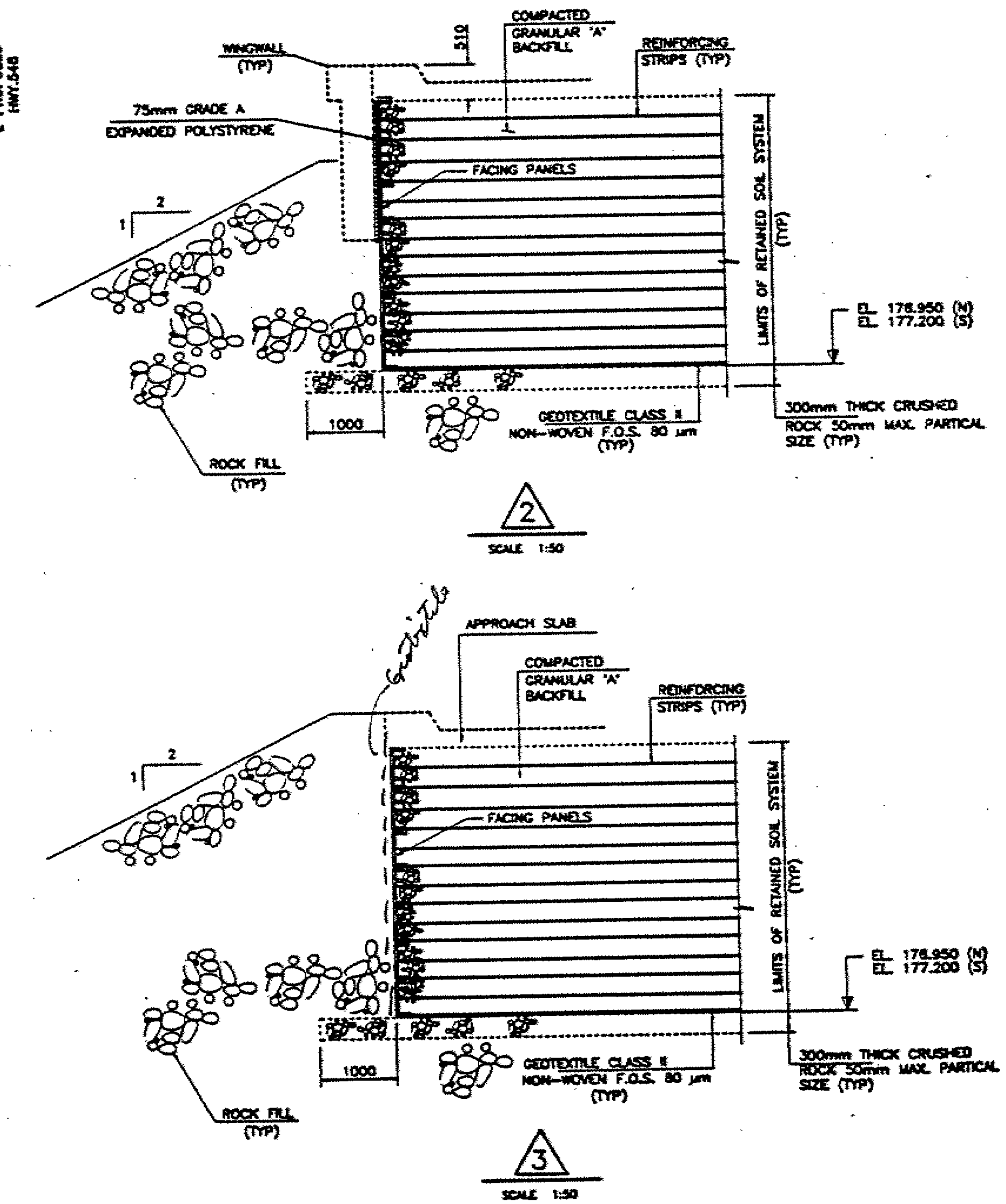
CONSTRUCTION SEQUENCE

1. CONSTRUCT CAISSONS.
2. CONSTRUCT RETAINED SOIL SYSTEM.
3. PLACE ABUTMENT AND WINGWALLS.





NOTE:

1. LIMITS, SPACING AND ORIENTATION OF REINFORCING STRIPS TO BE DETERMINED BY SUPPLIER OF RETAINED SOIL SYSTEM.
2. FOR ACCEPTABLE SUPPLIERS OF RETAINED SOILS SYSTEMS, SEE SPECIFICATIONS.

94-10-3
94-10-3

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN P.T.	CHK. M.R.	CODE OHBDC-91	LOAD	DATE AUG. 1994
DRAWN AKL/MN	CHK. P.T.	SITE 38S-177	STRUCT.	SCHEME	DWG. 7

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

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MUNROE ISLAND BRIDGE
HIGHWAY 548

WINGWALLS

SHEET 28

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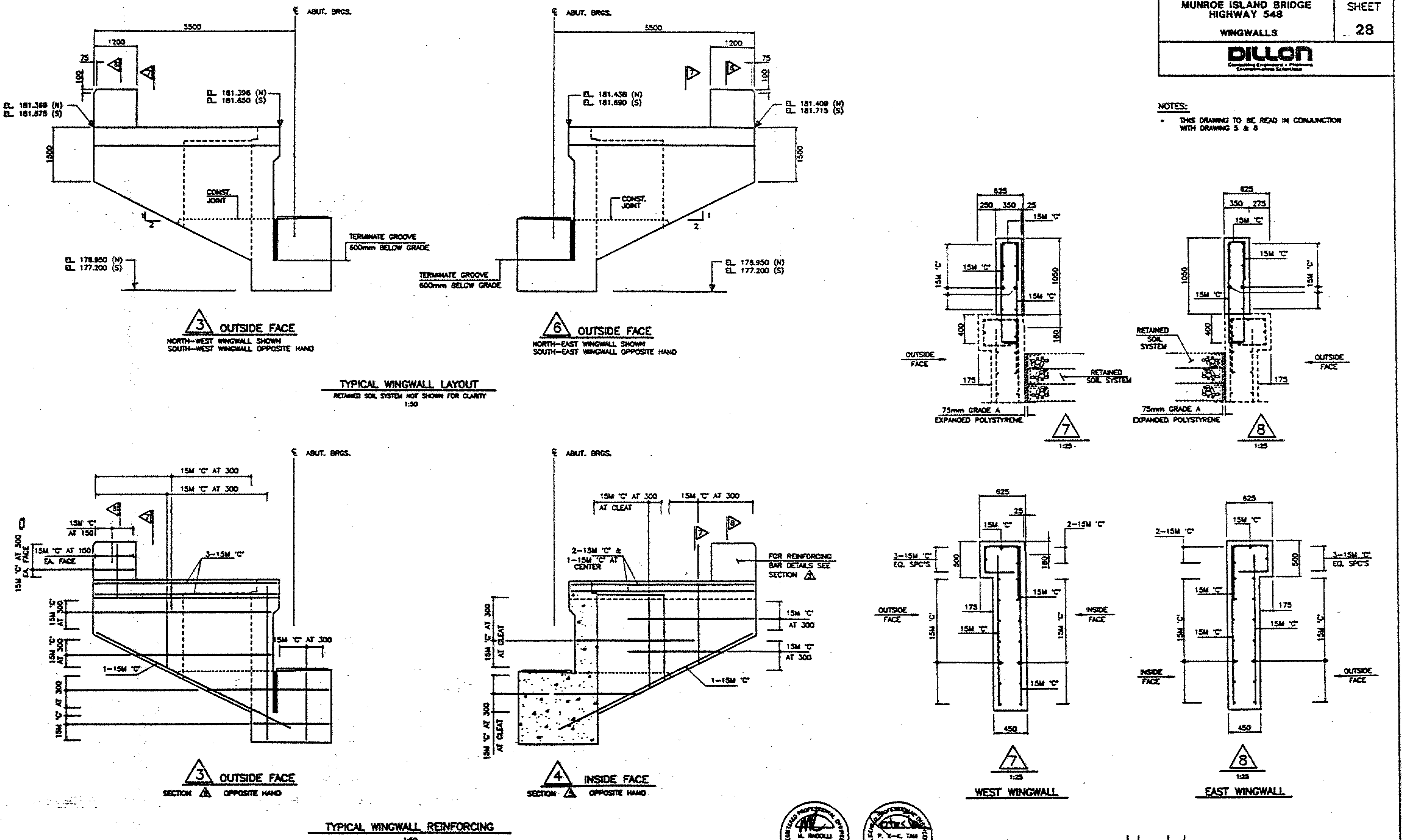
NOTES:
- THIS DRAWING TO BE READ IN CONJUNCTION WITH DRAWING 5 & 6



N. RADOLLI
PROVINCE OF ONTARIO
94-10-3
P. K.-K. TAM
PROVINCE OF ONTARIO
94-10-3

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN P.T.	CHK. M.R.	CODE OHBDC-83	LOAD	DATE AUG. 1994	
DRAWN A.K.L.	CHK. P.T.	SITE 385-177	STRUCT.	SCHEME	DWG. 8

FOUNDATION INVESTIGATION REPORT

CONTRACT NO. 94-228

Ontario

Ministry of Transportation

INDEX

Note: For purposes of the contract, this report supersedes all other Foundation Reports prepared by, or for the Ministry in connection with the above mentioned project.

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 (R Q D), 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

Foundation Investigation Report
For
Munroe Island Bridge at Highway 548
W.P. 29-88-01, Site 38 S-177
District 18, Sault Ste. Marie

INTRODUCTION

This report summarizes the results of the foundation investigation conducted at Munroe Island Bridge of Highway 548. The investigation was carried out upon the request of the Northwestern Region Structural Section for the proposed replacement bridge over Munroe Island channel of St. Mary's River. The field work for the investigation was carried out between 92 11 03 and 92 11 09 and consisted of six (6) sampled boreholes.

This report is applicable to the bridge structure and its immediate approaches (sta 13 + 380 to 13 + 470 approximately)

SITE DESCRIPTION

The site is located on Highway 548 at the southmost crossing from St. Joseph Island to the mainland, in the Township of St. Joseph, District of Algoma. It is about 3.4 km. south of the intersection of Highway 17 and Highway 548. According to Northern Ontario Engineering Geology Terrain Study 97 carried out by Ministry of Natural Resources, the landform of the site is a ground moraine with bedrock below a drift veneer.

The existing Munroe Island Bridge on Highway 548 is part of a causeway link joining St. Joseph Island and the mainland. The bridge was constructed in 1950 as a single span bridge founded on a pad of rockfill which formed part of the original causeway. Since construction, the bridge has undergone a history of distress and various remedial measures have been implemented. The present bridge has a three span configuration with short girder type spans added to each end. The piers are found to be tilting forward and sideways. Tie back cables have been installed between the pier and the south abutment. The existing pavement is in poor conditions with cracks and patches. The steel members of the bridge truss is rusty and some concrete spalling is noted in the structure.

The existing highway embankment at the approaches are up to about 10 m high with about half of it under water. The embankments are granular in nature with large rock armouring blocks on the slope face especially at the approaches. Approach slope gradients are up to 1.4H:1V to 1.5H:1V steep. The slopes appear to be marginally stable in local steep areas. The Munroe Island Channel is about 35 m wide at the bridge location. The sub-aqueous slopes are fairly steep around the bridge and to the east. Some whirlpools can be seen on the east side of the bridge. During the time of the investigation, the water level was at about El.176.4 m.

INVESTIGATION PROCEDURES

Soil data and inherent properties were obtained by in situ and laboratory testing. The procedures employed are discussed below.

Field

The field work for the investigation was carried out between 92 11 03 and 92 11 09 and consisted of six (6) sampled boreholes, which were advanced to depths of 3.1 to 16.3 m.

For the boreholes on land (BH1 and BH2) a track mounted continuous flight auger machine was used to advance the boreholes with conventional hollow stem augering techniques. For the boreholes over water (BH3 to BH6), a diamond drill rig resting on a raft and equipped with N/B size casings was used to advance them.

The sampling program consisted of split spoon samples collected in the overburden. Disturbed subsoil samples were retrieved by a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). They provided Standard Penetration (N) values for assessment of the denseness of the non-cohesive material. All the samples collected were used for identification and laboratory testing purposes. Dynamic core Penetration Test was carried out in BH3, BH4 and BH6. Conventional rock coring methods were applied in retrieving rock core samples using BXL core barrels.

All subsoil samples were identified in the field and returned to the laboratory for further examination and appropriate testing.

Groundwater levels were monitored throughout the duration of the investigation in open boreholes on land. All the land holes were backfilled upon completion of the field work.

Survey information related to the location and elevation of boreholes was provided by the Northwestern Region, Surveys and Plans Section.

Laboratory

The laboratory testing program for select soil samples consisted of:

- grain size distribution
- natural moisture content determinations

Laboratory test results are given in the following section of this report and Record of Borehole sheets included in the Appendix.

SUBSURFACE CONDITIONS

General

The Record of Borehole sheets in the Appendix illustrate the subsurface conditions at the borehole locations. The locations and elevations of the boreholes are shown in DWG. No. 2 (Sheet 22) of the Contract Drawings.

BH1 and BH4 are on or close to the existing embankment. In BH1 to 3, the subsurface material encountered comprised of fill over bedrock. A thin layer (1.3m) of native material consisting of silty sand, some gravel, with boulders was found in BH4 overlying bedrock. In BH5 and BH6 which are further away from at the top 1.3 m, ranging from 30 to 58% The rock at this location is considered to be medium strong.

Groundwater

During the course of the investigation, the water level in the river was at about El.176.4 m. For the boreholes on land, observations of the groundwater level were carried out by measuring the water levels in open boreholes during the investigation. Groundwater levels measured were typically close to the water level in the river.

Water levels are subject to seasonal fluctuations and hence may vary from values given in this report.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of D. Kwok, Project Foundation Engineer. The equipment was owned and operated by Master Soil Investigation Ltd. Bedrock was examined and classified by MTO petrographer, D. Williams.

The project was carried out by D. Kwok under the supervision of Dr. B. Iyer, Senior Foundation Engineer. The report was prepared D. Kwok, reviewed by B. Iyer, and approved by Mr. M. Devata, Chief Foundation Engineer.

LICENSED PROFESSIONAL ENGINEER
TCK
T.-C. KIM
94-11-16
PROVINCE OF ONTARIO

Taechul Kim
T. Kim, P. Eng.
Sr. Foundation Engineer

APPENDIX

RECORD OF BOREHOLE No 1

METRIC

W.P. 29-88-01 LOCATION Sta 13+401.2 12.3 m Lt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE H.S. Auger, BW Casing COMPILED BY DK

DATUM Geodetic DATE 92 11 04 - 92 11 05 CHECKED BY BI

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.2	Ground Surface							
0.0	Silty Sand and Gravel, Trace Clay, Occasional Shale Fragments, Brown and Grey (Fill)					182		
		1	SS	11		180		
		2	SS	6		178		
		3	SS	21	▼*		o	52 24 16 8
		4	SS	12		176		
		5	SS	46		174		
173.2		6	SS	100	/28cm			
9.0	Gabbro Bedrock	7	RC	REC	94%			RQD 26%
		8	RC	REC	100%	172		RQD 67%
		9	RC	REC	100%	170		RQD 41%
167.9		10	RC	REC	100%			RQD 67%
14.3	End of Borehole							

* 92 11 05

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100

Shear Strength kPa: o Unconfined; + Field Vane; ● Quick Triaxial; × Lab Vane — 20 40 60 80 100

Plastic Limit w_P, Natural Moisture Content w, Liquid Limit w_L — Water Content (%) 20 40 60

Unit Weight γ kN/m³

$+^3, \times^5$: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 2 1 OF 1 METRIC

W.P. 29-88-01 LOCATION Sta 13+457.4 8.3 m Lt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE H.S. Auger, BW Casing COMPILED BY DK

DATUM Geodetic DATE 92 11 03 - 92 11 04 CHECKED BY BI

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.3	Ground Surface							
0.0	Silty Sand and Gravel, Occasional Shale Fragments (Fill)					182		
		1	SS	5				
						180		
	Brown / Grey	2	SS	22			o	32 43 18 7
		3	SS	11		178		
		4	SS	20				
		5	SS	11	▼*			
		6	SS	11		176		
		7	SS	4			o	44 49 5 2
	Sand and Gravel, Occasional Cobbles and Boulders	8	SS	7		174		
		9	SS	8				
171.8						172		
10.5	Quartz Sandstone Bedrock	10	RC	REC	60%			RQD 0%
		11	RC	REC	75%	170		RQD 0%
		12	RC	REC	50%			RQD 0%
		13	RC	REC	96%	168		RQD 43%
		14	RC	REC	77%			RQD 0%
		15	RC	REC	88%			RQD 0%
166.0		16	RC	REC	96%			RQD 30%
16.3	End of Borehole							
	* 92 11 03							

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; STRAT PLOT.

$+^3, \times^5$: Numbers refer to Sensitivity   15-◇-5 (20 / 10) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

METRIC

W.P. 29-88-01 LOCATION Sta 13+402.6 3.6 m Rt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE Washboring, BW Casing, Cone COMPILED BY DK

DATUM Geodetic DATE 92 11 06 – 92 11 08 CHECKED BY BI

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L – WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.4	Water Surface									
0.0	Water					176				
						174				
173.0 / 3.4	Sand and Gravel Trace Silt, Trace Organics Occasional Cobbles and Boulders (Possible Fill)	1	SS	12		172				
		2	SS	21						
170.6		3	SS	28			50/ 23cm Bouncing on rock	o		57 30 9 4
5.8	Gabbro Bedrock	4	RC	REC	100%	170				RQD 28%
		5	RC	REC	100%					RQD 33%
		6	RC	REC	80%					RQD 0%
		7	RC	REC	100%					RQD 11%
166.9		8	RC	REC	95%	168				RQD 44%
9.5	End of Borehole									

$+^3$, $\times^5$: Numbers refer to Sensitivity

15-◇-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

1 OF 1 METRIC

W.P. 29-88-01 LOCATION Sta 13+453.9 5.3 m Rt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE Washboring, BW Casing, Cone COMPILED BY DK

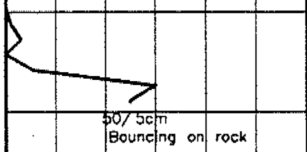
DATUM Geodetic DATE 92 11 03 - 92 11 06 CHECKED BY BI

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.4	Water Surface					176			
0.0	Water					174			
173.2									
3.2	Sand and Gravel (Granular Fill)	1	SS	15		172			
171.3									
5.1	Silty Sand and Gravel	2	SS	100 /10cm			o		51 35 10 4
	Boulders	3	SS	60					
		4	RC	REC	50%				RQD 0%
170.0						170			
6.4	Highly Fractured	5	RC	REC	26%				RQD 0%
		6	RC	REC	28%	168			RQD 0%
	Quartz Kaolinitic Sandstone Bedrock	7	RC	REC	58%				RQD 0%
		8	RC	REC	30%				RQD 0%
		9	RC	REC	92%	166			RQD 50%
		10	RC	REC	83%				RQD 38%
		11	RC	REC	100%				RQD 35%
164.1		12	RC	REC	70%				RQD 45%
12.3	End of Borehole								

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60



$+^3$, $\times^5$: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 5

METRIC

W.P. 29-88-01 LOCATION Sta 13+390.5 12.5 m Rt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE Washboring, BW Casing COMPILED BY DK

DATUM Geodetic DATE 92 11 08 CHECKED BY BI

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.4	Water Surface										
0.0	Water						176 / 174 / 172				
170.9 / 5.5	Sand and Gravel, Some Boulder **		1	SS	**				o		48 45 4 3
170.1	*		2	RC	REC	67%					RQD 0%
6.3	End of Borehole										

* Quartz Sandstone,Gabbro Bedrock

** Bouncing on rock

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ◇ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 6

METRIC

W.P. 29-88-01 LOCATION Sta 13+465.5 15.1 m Rt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE Washboring, Cone COMPILED BY DK

DATUM Geodetic DATE 92 11 08 - 92 11 09 CHECKED BY BI

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P – NATURAL MOISTURE CONTENT w – LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.4	Water Surface										
0.0	Water						176				
174.1 / 2.3	Silty Sand with Organics, Trace Gravel Occasional Cobble and Boulder Dark Brown		1	SS	25 /23cm bouncing on boulder		174				9 46 39 6
173.3 / 3.1	End of Borehole — Probable Bedrock							50/ 8cm bouncing on rock			

$+^3$, $\times^5$: Numbers refer to Sensitivity

$15\overset{20}{\underset{10}{\diamond}}5$ (%) STRAIN AT FAILURE

ROCK CORE DESCRIPTION
WP 29-88-01

	CORE RECOVERY				CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	7	9.02-9.65	94	26	9.02-14.30	**GABBRO** (chloritic), dark greenish grey to greenish black (stained dark reddish brown, 10.84-12.37 m); coarse to fine grained; strong; unweathered to slightly weathered; fractures moderate to extremely close spaced, dipping to near vertical, undulating to planar, smooth.
	8	9.65-11.10	100	67		
	9	11.10-12.70	100	41		
	10	12.70-14.30	100	67		
2	10	10.49-11.51	60	0	10.49-16.31	**QUARTZ SANDSTONE**, medium grey; fine to medium grained; strong; unweathered to slightly weathered; fractures close to extremely close spaced, near vertical to flat, undulating to planar, smooth to rough.
	11	11.51-12.22	75	0		
	12	12.22-12.93	50	0		
	13	12.93-14.35	96	43		
	14	14.35-15.01	77	0		
	15	15.01-15.62	88	0		
	16	15.62-16.31	96	30		
3	4	5.82-6.27	100	28	5.82-9.47	**GABBRO** (chloritic), dark greenish grey to greenish black; coarse to fine grained; strong; unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating, smooth.
	5	6.27-6.73	100	33		
	6	6.73-6.86	80	0		
	7	6.86-7.80	100	11		
	8	7.80-9.47	95	44		

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section

ROCK CORE DESCRIPTION
WP 29-88-01

	CORE RECOVERY				CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
4	4	5.61-6.22	50	0	5.61-6.43	**OVERBURDEN** (till: boulder of **QUARTZ SANDSTONE**, 5.92-6.22 m, light brownish grey; fine to medium grained; strong; unweathered to slightly weathered).
	5	6.43-7.29	26	0		
	6	7.29-8.56	28	0		
	7	8.69-9.30	58	0	6.43-12.32	**QUARTZ SANDSTONE** (kaolinitic), pale red; fine to coarse grained; medium strong; unweathered to slightly weathered; fractures moderate to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
	8	9.30-10.06	30	0		
	9	10.06-10.72	92	50		
	10	10.72-11.46	83	38		
	11	11.46-12.07	100	35		
	12	12.07-12.32	70	45		
5	2	5.97-6.27	67	0	5.97-6.10	**OVERBURDEN** (till: pebble of **QUARTZ SANDSTONE**(feldspathic), 5.97-5.99 m, greyish orange; fine to medium grained; strong; unweathered to slightly weathered).
					6.10-6.27	**GABBRO** (chloritic), dark greenish grey to greenish black; coarse to fine grained; strong; unweathered to slightly weathered; fractures very close to extremely close spaced, dipping to near vertical, undulating to planar, smooth.

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section

Ministry of Transportation of Ontario
Northwestern Region

HIGHWAY 548, MUNROE ISLAND BRIDGE
RICHARDS LANDING, ONTARIO
SITE NO. 38S-177, DISTRICT 18

UNDERWATER INSPECTION REPORT

Agreement No. 4513-6091-626

prepared by:

Proctor & Redfern Limited
Consulting Engineers, Architects, Scientists and Planners
379 Dundas Street
London, Ontario
N1B 1V5

February 24, 1992

EO 91964

PROCTOR & REDFERN LIMITED

February 27, 1992 EO 91964

Mr. Ray Kriscuinas, P.Eng.
Head, Structural Section
MINISTRY OF TRANSPORTATION
P.O. Box 1177
615 South James Street
Thunder Bay, Ontario
P7C 4X9

Attention Mr. Dwain Bennett
Structural Technician

Dear Sir

Re **Highway 11, Noden Causeway**
Site 45W-75, District 20
Kenora Distict, Ontario
Underwater Inspection Report

In accordance with the authorization given by the Ministry of Transportation on October 9, 1991, we are pleased to submit four (4) copies of the final report of our underwater inspection of the above noted bridge.

The report details the visual examination of the underwater components of the structure and an evaluation of lakebed scouring.

The findings of the inspection revealed no evidence of significant deterioration of the underwater portions of the bridge structure.

Thank you for the opportunity afforded our firm in completing this inspection for the Ministry of Transportation. If you require further information or discussion, please do not hesitate to call.

Yours truly

Proctor & Redfern Limited

Greg J. Prichard
Chief Engineering Diver

Paul Schroeder

Paul Schroeder, P.Eng.
Project Engineer

GP:cs

TABLE OF CONTENTS

LIST OF APPENDICES

1. INTRODUCTION

For bridges built across waterways, one of the common potential problems is foundation substructure damage caused by stream currents and ice movements. Erodable riverbed material can be progressively washed away by the river flow, causing local scouring around and under the bridge foundation. Ice and debris impact and freeze-thaw action can damage structural components at or near the water level. If the extent of scouring and damage is severe or deep enough, it can reduce the safety of the bridge structure.

Bridge foundations are generally designed and constructed below estimated depth of scour, or otherwise protected to prevent undermining. Measures to prevent structural damage at water level include shaping, steel nosings, sheet pile and soldier piles. After construction, periodic maintenance inspections should be performed to monitor the extent of scouring and the structure condition.

In the current maintenance program of the Ministry of Transportation of Ontario (MTO), the Northwestern Region has initiated underwater inspections of a group of bridges identified as susceptible to scouring and flood damage.

Proctor & Redfern was retained by the MTO Structural Section, Northwestern Region in October 1991 to carry out and report on the underwater inspection of six bridges in the region.

This report covers the underwater inspection of the **Highway 548, Munroe Island Bridge, MTO Site 38S-177**, including visual examination of the underwater components of the foundation, underwater videotape recordings of the foundations, determination of scouring conditions, and recommendations for remedial work. The Munroe Island Bridge was inspected previously by our Chief Engineering Diver in 1984.

2. DESCRIPTION OF THE EXISTING STRUCTURE

The Munroe Island Bridge, MTO Site No. 38S-177, is located on Highway 548 over the St.Marys River near the Hamlet of Richards Landing, Ontario. The location of the bridge is shown on the key map included on each of the figures following the text of the report.

Based on the construction drawings provided by the MTO, it is apparent that the original bridge, constructed in 1949, was a 2-lane, single-span structure with a span of 28 metres (92 feet). Subsequent alterations have led to the existing 2-lane configuration which consists of 3 spans.

The centre span superstructure is comprised of a cast-in-place concrete deck on longitudinal 375 mm (15 inch) deep stringers spanning between 900 mm (36 inch) deep transverse girders which are in turn supported by riveted steel Warren trusses at the outside edges of the structure. The two end spans consists of cast-in-place concrete deck cast directly on longitudinal steel girders.

The total structure length is approximately 45.7 metres (150 feet), with two end spans of 8.8 metres (29 feet), and one interior span of 28 metres (92 feet). The overall deck width is approximately 10.7 metres (35 feet) including two 4.6 metres (15 foot) traffic lanes and steel railings on each side. The bridge is comprised of two riveted steel trusses with cast-in-place concrete deck supported by concrete abutments and piers.

At the bridge location, the St.Marys River flows from west to east. At the time of inspection, the footings of both Piers 1 and 2 were in the water. The abutments were surrounded by dry ground, well above water level. Photographs 1 & 2 show the general view of the bridge and piers. Photographs 5 to 9 show the general layout of the piers, and Photograph 12 shows the general layout of the abutments.

The concrete piers extend below the steel trusses to the approximate water surface and are supported on cast-in-place spread footings.

The concrete footings are finished square at both the upstream and downstream ends. The north and south abutments were on dry ground, well above water level at the time of the inspection. The general arrangement of the bridge, as taken from the original drawings and the November 1991 photographs, is shown on Figure 1 and in Photographs 1 and 2.

3. UNDERWATER INSPECTION

3.1. General

Inspection Team:	Don Bazely	Specialist Engineer
	Greg Prichard	Chief Engineering Diver
	Greg Potter	Engineering Diver
	Ken Ross	Engineering Diver

Structure Site No.:	38S-177
Inspection Date:	November 14 and 15, 1991
Weather:	Cloudy, 3^{o}C
Water Flow:	Moderate current, 0.8 metres per second
Water Clarity:	Poor underwater visibility (less than 300 mm)
Water Level:	El. 175.96 metres (577.15 ft)
Reference Elevation:	Top of concrete deck, centreline of highway span, El. 182.93 metres (600 ft)

3.2. Inspection

Underwater inspection of the Highway 548, Munroe Island Bridge, was carried out by a team of four members of our engineering staff on November 14 and 15, 1991. The purpose of the investigation was to examine the general condition of the foundations to determine the extent of any river bottom scouring in the vicinity of the piers and abutments and to record deterioration of structural components of the bridge foundations.

All the underwater inspections were carried out in accordance with the Ministry of Labour, Ontario Regulation 634/86 which governs and regulates diving operations in the Province of Ontario.

The underwater inspection was performed by one of our engineering divers, in a surface supplied air diving helmet, who was in constant voice communication with a structural engineer during the inspection. On the Munroe Island Bridge a video camera was used by the engineering diver to record the underwater inspection. The video image was monitored on the surface by an engineering diver and a structural engineer. The video recordings served as a detailed record of portions of the underwater inspection.

The surface backup for our engineering diver was provided by two trained and experienced divers. A fully equipped engineering diver conducting the inspection is shown in Photograph 4.

The condition of concrete components was evaluated by visual inspection and hammering of the concrete surface. The surficial soil type and relative density in the vicinity of the foundations was evaluated by visual examination and probing of the river bottom with a hand-held 15 mm diameter by 800 mm long steel rod. Water depth soundings were obtained with a sounding rod and an electronic depth sounder around the bridge piers in order to accurately record riverbed depths. Photograph 3 shows the inspection team obtaining water depth soundings around a bridge pier.

For the purpose of this report, the elevation on top of the concrete deck at the centre line of the span of Highway 548, elevation 182.93 metres (600 feet) as shown on the original drawings, was used as the reference elevation. The water level on November 14, 1991 was measured at elevation 175.96 metres (577.15 feet), close to the preconstruction water elevation 176.91 metres (580.27 feet) as shown on the original drawings.

3.3. Observation and Inspection Results

3.3.1 General

The results of the underwater soundings are presented in Figure 2 which shows a plan of the riverbed elevations. Cross sections of the riverbed are given in Figure 3 and profiles of the riverbed along Piers 1 and 2 and the river centreline are given in Figure 4. In general, the stream bed was found to consist of loose rock material, concrete sacks and concrete rubble material with occasional sandy areas as shown in Photographs 13 through 17. The river bottom could not be probed due to the coarse nature of the material.

3.3.2 Pier 1

The general layout of Pier 1 is shown in Photographs 5 & 6. At the time of the inspection, most of the concrete pier was above the water level. The concrete footing was mostly below water level. The original construction drawings show that the pier is founded on spread footings, with no indication of pile support.

Above the water level, the concrete pier is in poor to fair condition. Cracking of the concrete surface and deterioration at cold joints in the concrete is evident at several locations. Efflorescence and rust stains were observed at many of the cracks and cold joints found in the abutment concrete.

At the water level, the joint between the footing and the concrete pier is tight and shows no evidence of significant deterioration. The remaining portion of the pier and the concrete footing are generally in good condition. The exposed portions of the footing show minor loss of concrete surface due to erosion. The concrete aggregate in the footing is exposed, however, the footing concrete does not exhibit evidence of major cracks or spalls in the exposed concrete surfaces. Portions of the original footing form work are still in place near the base of the footing (see Photograph 18). Exposed reinforcing steel was not observed at any location on the footing concrete.

From the inspection, overall and differential settlement of the pier has occurred. This is evidenced by the apparent tilt of the pier toward the channel, settlement of the pier at the west end and the numerous shims that have been installed between the pier concrete and the bridge girders. Field measurements of the inclination of the pier faces off of vertical indicate that the top of the pier is inclined toward the channel and toward the west by the following amounts.

- Toward Channel 1 1/4 inch in 48 inches = 1.49^{o} from vertical
- Toward West 3/4 inch in 48 inches = 0.90^{o} from vertical

The soil around the pier consisted of loose rock (Photograph 13) and concrete sacks (Photograph 14). A steel rod was used to probe into the river bottom. However, very little penetration was possible due to the coarse nature of the material. Very little debris was noted on the river bottom around the pier.

3.3.3 Pier 2

The general layout of Pier 2 is shown in Photographs 7 & 8. At the time of the inspection, most of the concrete pier was above the water level. The concrete footing was entirely below the water level. The original construction drawings show that the pier is founded on spread footings, with no indication of pile support.

Above the water level, the concrete pier is in poor to fair condition. Photographs 7 and 9 show the general condition of the pier. Cracking of the concrete surface and deterioration at cold joints in the pier concrete is evident at several locations. Efflorescence and rust staining was observed at many of the cracks and cold joints.

Below the water level, the remaining portion of the pier and the concrete footing are generally in good condition. The joint between the footing and the concrete pier is tight and shows no evidence of significant deterioration (see Photograph 19). The footing concrete surface is eroded and the concrete aggregate in the footing is exposed as shown in Photograph 20 & 21, however, the footing concrete does not exhibit evidence of major cracks or spalls in the exposed concrete surfaces. Exposed reinforcing steel was not observed at any location on the footing concrete.

From the inspection, overall and differential settlement of the pier has occurred. This is evidenced by the apparent tilt of the pier toward the channel, settlement of the pier at the west end and the numerous shims that have been installed between the pier concrete and the bridge girders. Field measurements of the inclination of the pier faces off of vertical indicate that the top of the pier is inclined toward the channel and toward the west by the following amounts.

- Toward Channel 2 1/2 inch in 48 inches = 2.98^{o} from vertical
- Toward West 1/4 inch in 48 inches = 0.30^{o} from vertical

In one area toward the upstream end of Pier 2, a small gap about .75 metres in length, 5 cm high and 15 cm deep, was found between what appeared to be the bottom of the footing and the riverbed. The gap, shown in Photographs 22 and 23, is the result of either localized undermining erosion, or localized slope failure in the riverbed.

3.3.4 Abutments

Both the north and south abutments were on dry ground at the time of the inspection. The abutment slope on the south side is protected by random sized rip rap and armour stone material. The protection is concentrated at the toe of the slope leaving some of the upper, less vegetated portions of the slope vulnerable to soil erosion.

The abutment slope on the north side of the crossing is protected with rip rap and armour stone on the west side and is virtually unprotected on the east side as shown in Photograph 2. The unprotected portion of the slope is steep and bare and unable to sustain any vegetative cover. Lack of topsoil, minor slope failures and surface erosion appear ongoing, thus preventing the development of a protective vegetative cover.

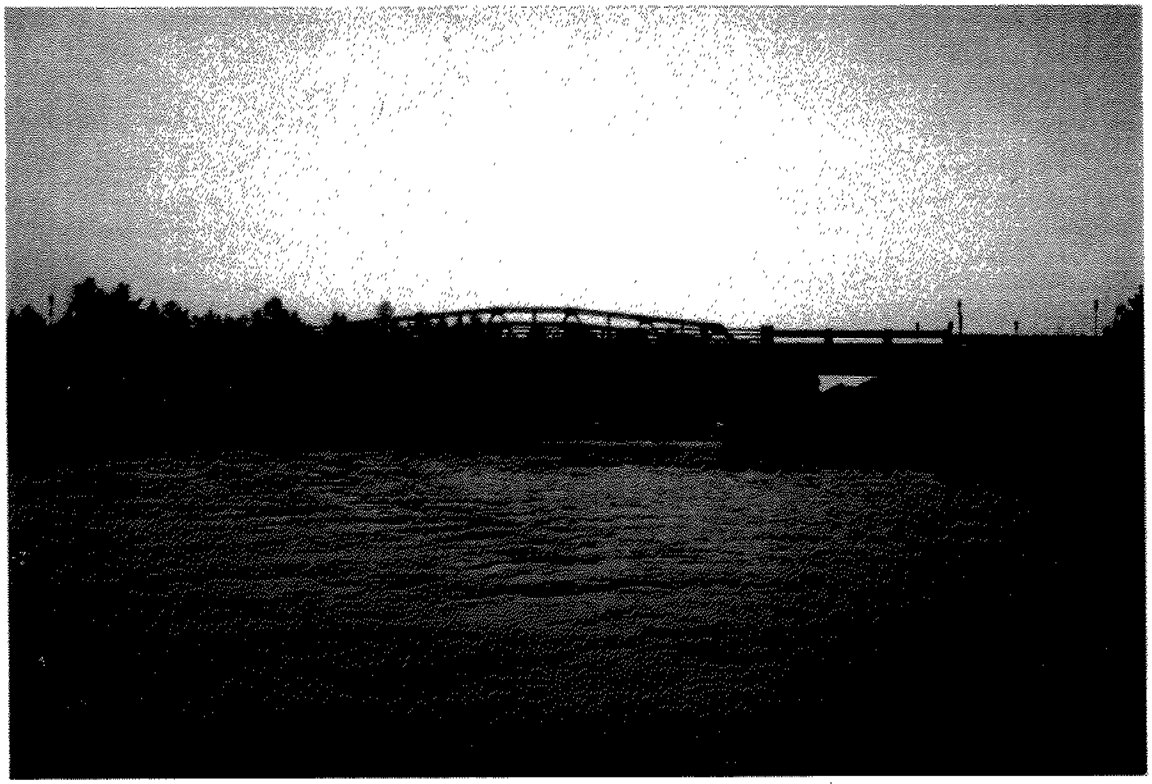
4. CONCLUSIONS AND RECOMMENDATIONS

This investigation has confirmed the results of previous site inspections carried out at the Munroe Island Bridge. Settlement and tilting of the bridge foundations has occurred, reducing the stability of the structure. There visually appears to be little change in the underwater condition of the structure since the 1984 inspection.

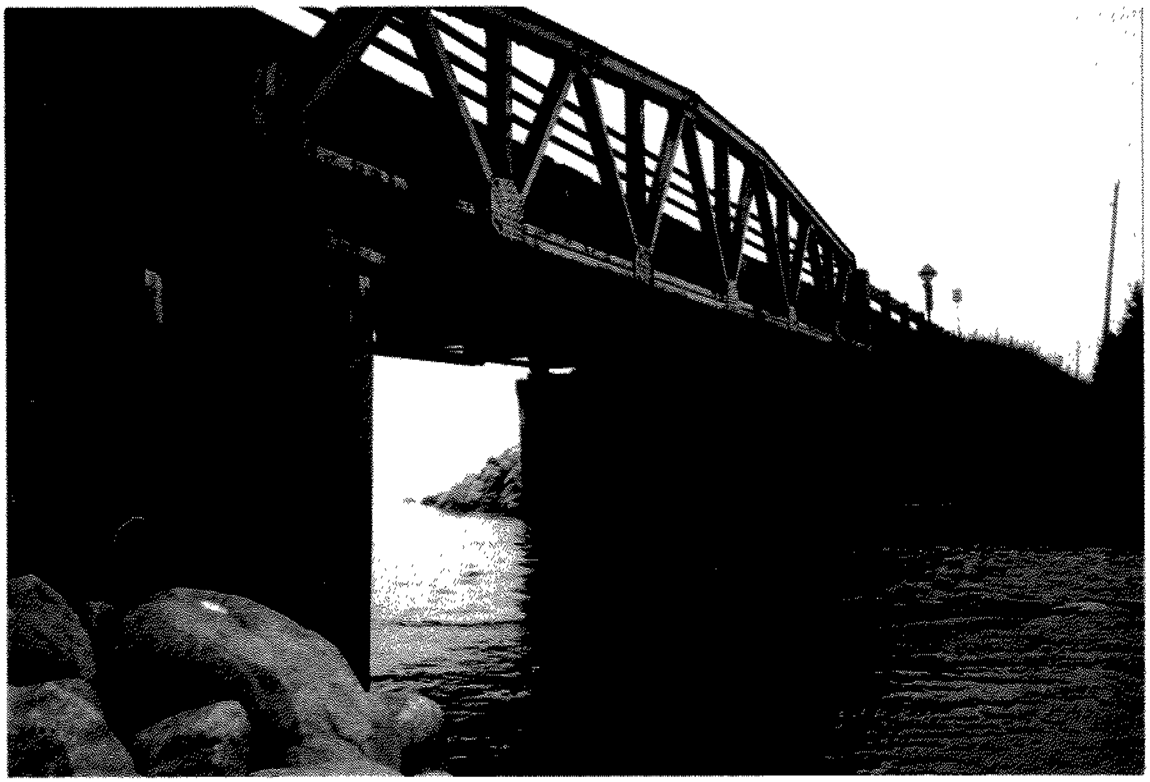
Due to the piers being founded on spread footings which have settled and remain susceptible to riverbed erosion, we recommend that the extent of undermining be monitored on a regular basis until such time as an effective repair is undertaken or the bridge replaced.

Appendix A

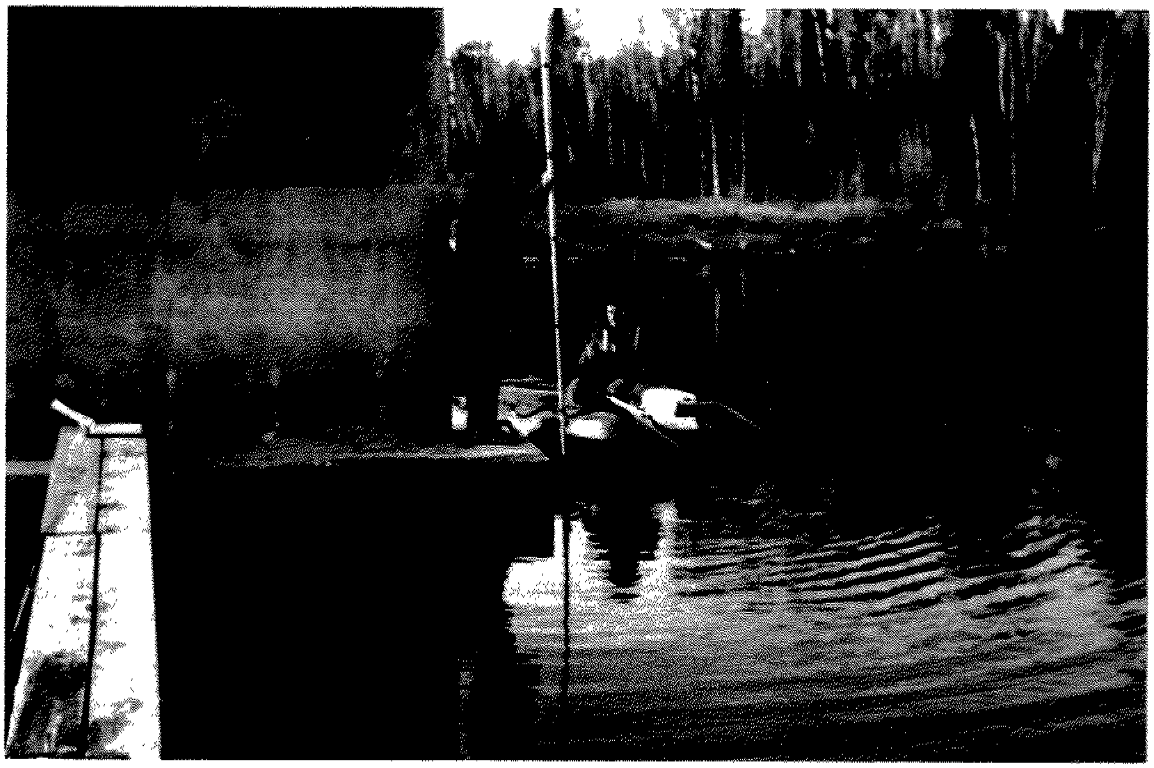
PHOTOGRAPHS



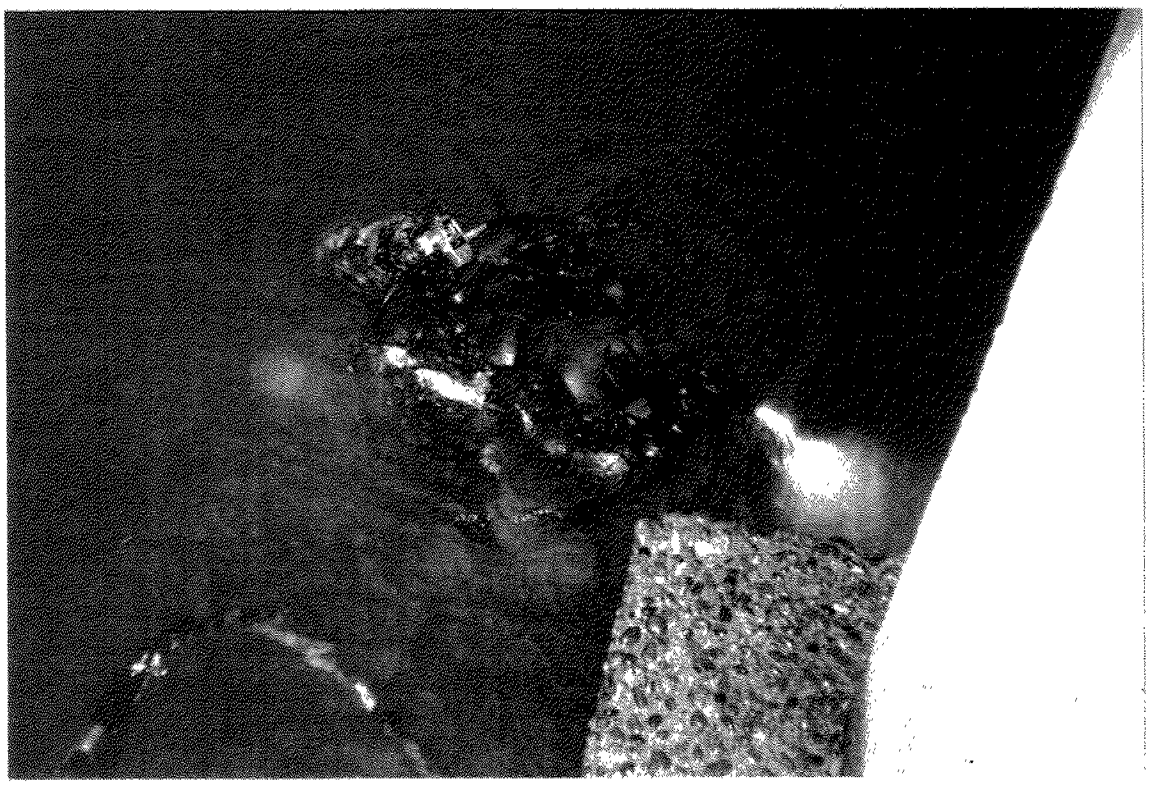
PHOTOGRAPH 1 General arrangement of bridge structure. Photograph taken looking west.



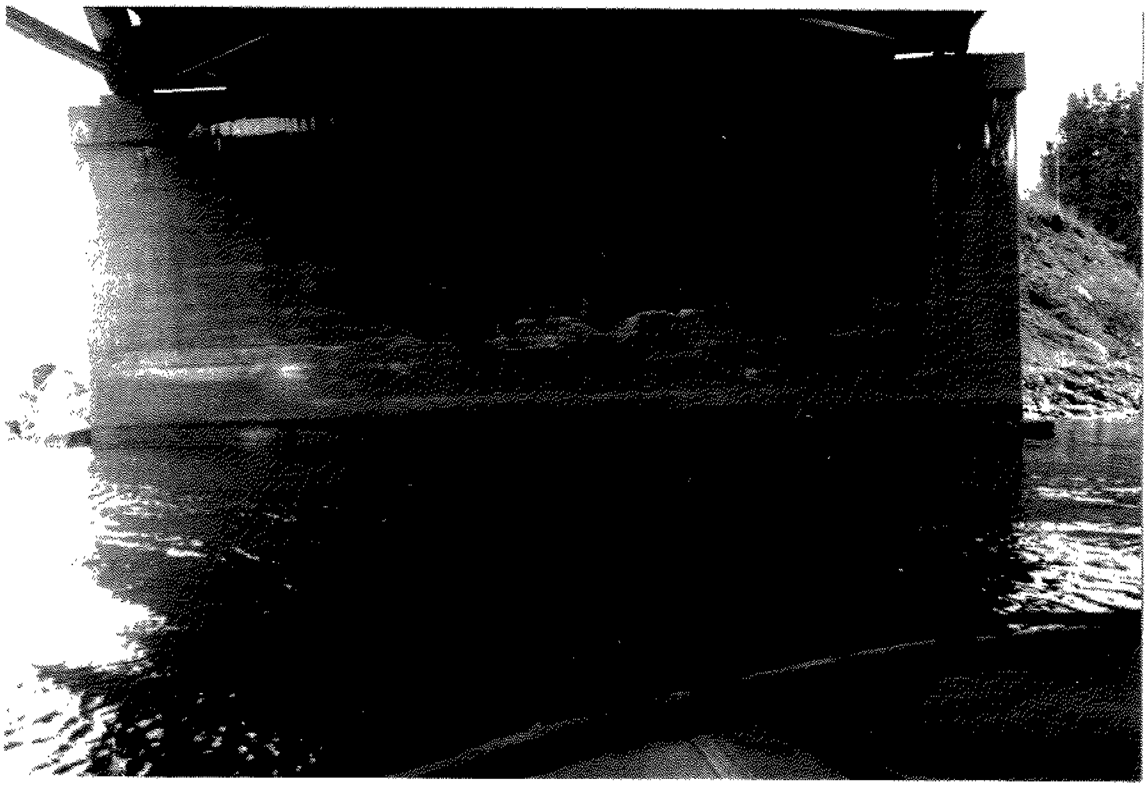
PHOTOGRAPH 2 General arrangement of bridge structure. Photograph taken looking north.



PHOTOGRAPH 3 Obtaining water depth soundings around bridge pier.



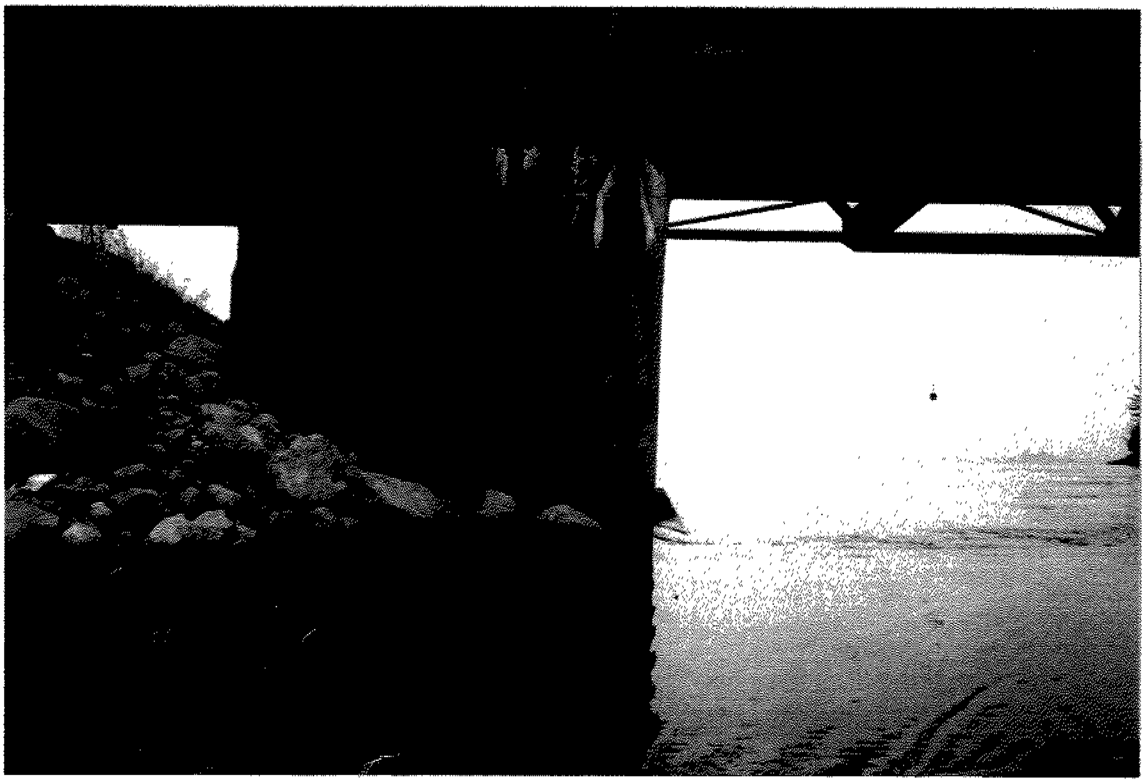
PHOTOGRAPH 4 Engineering diver conducting the inspection.



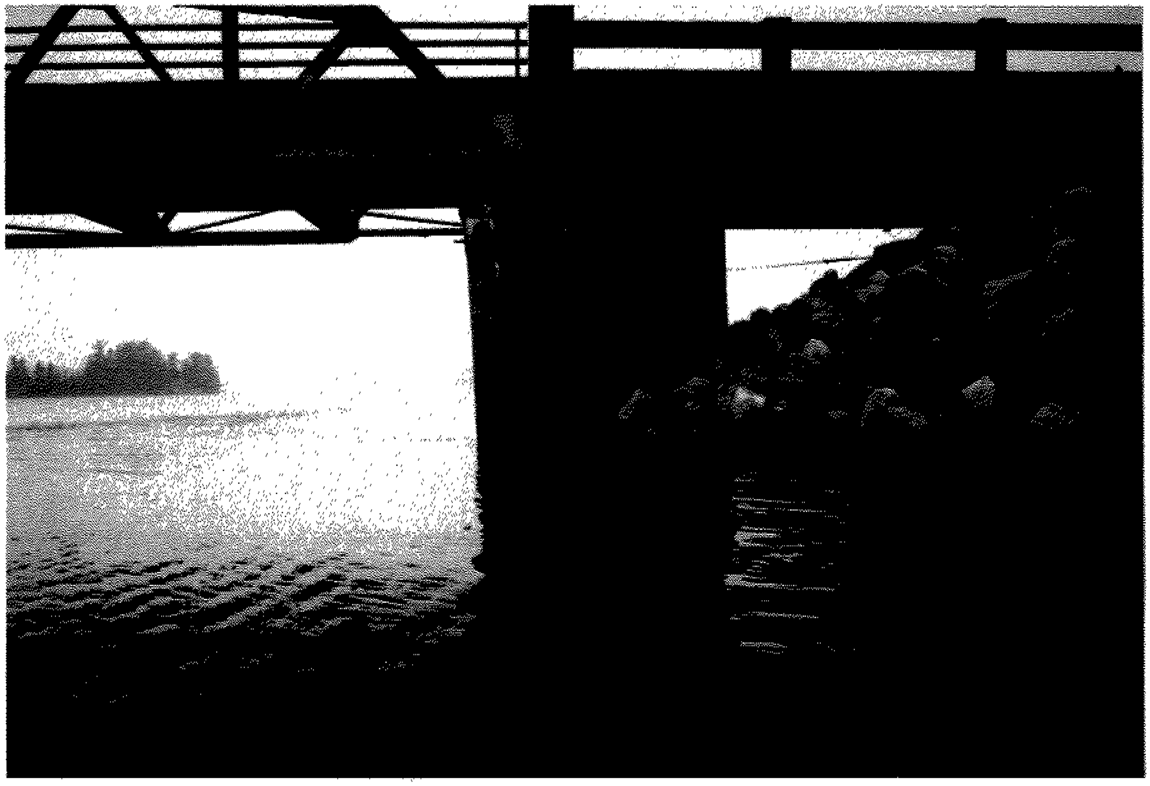
PHOTOGRAPH 5 South face of Pier 1.

PHOTOGRAPH 6 South face of Pier 1. Note differential settlement of pier evidenced by varied exposure of the concrete footing.

PHOTOGRAPH 7 North face of Pier 2.



PHOTOGRAPH 8 Upstream face of Pier 1. Note lean of pier toward the channel.



PHOTOGRAPH 9 Upstream face of Pier 2. Note lean of pier toward the channel.

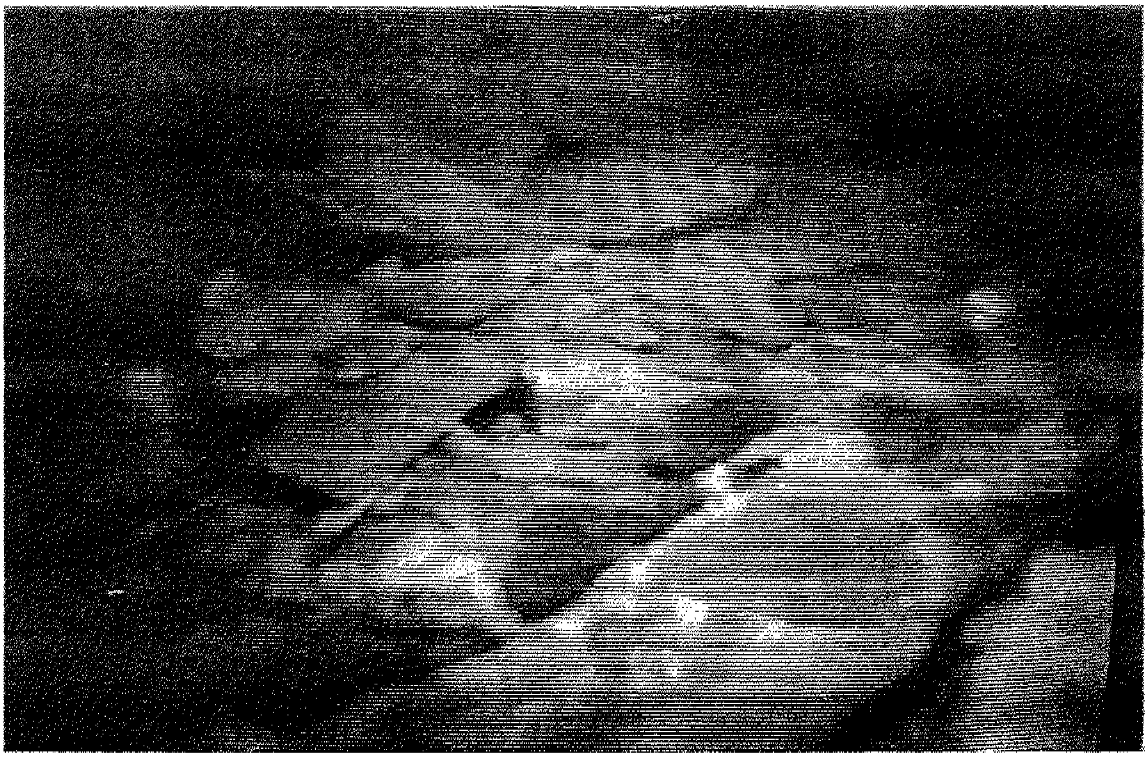
PHOTOGRAPH 10 Note deck settlement on east side of bridge above Pier 2.

PHOTOGRAPH 11 Note deck settlement on west side of bridge above Pier 2.

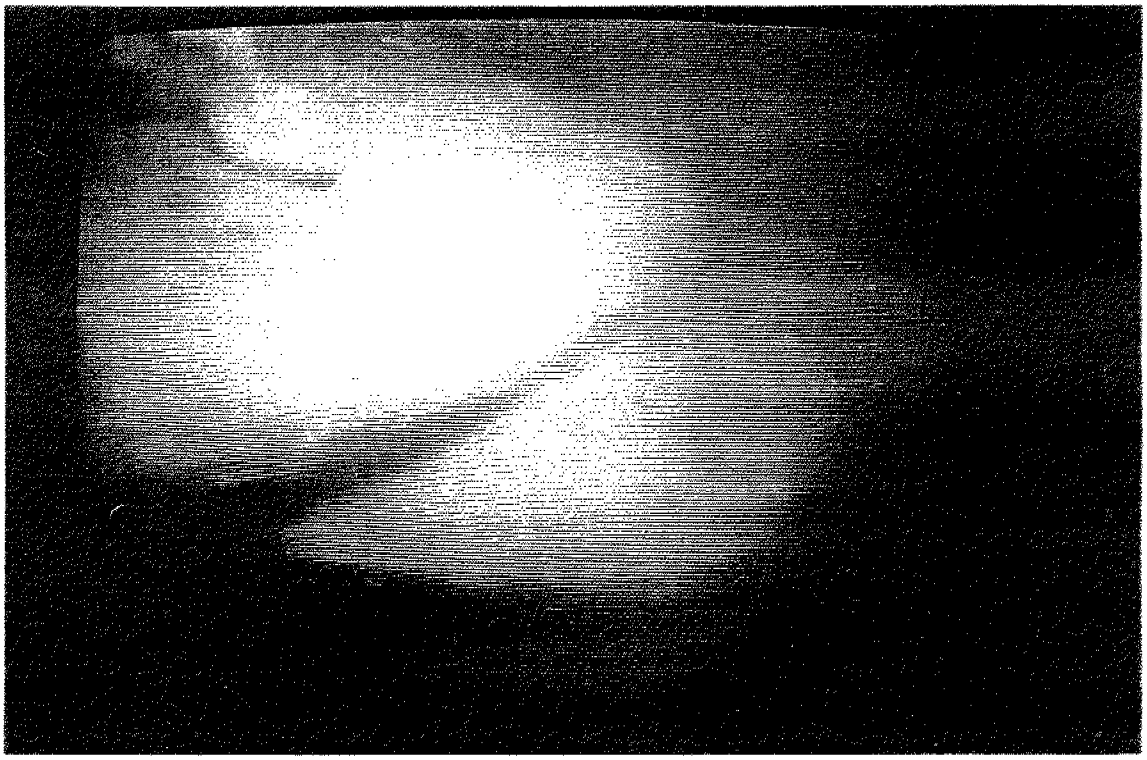
PHOTOGRAPH 12 General arrangement of bridge abutment.

NOTE

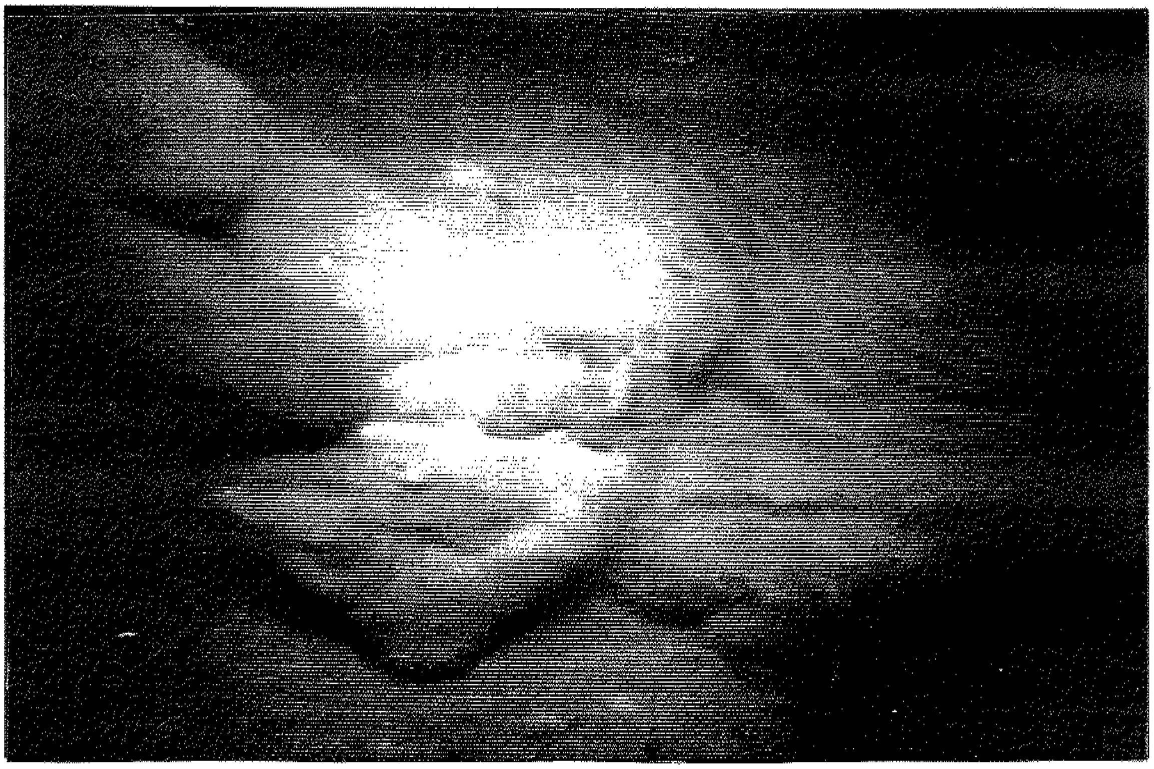
Photographs 13 to 23 are complied from an underwater video recording of the site as still photography during the inspection was unsuccessful.



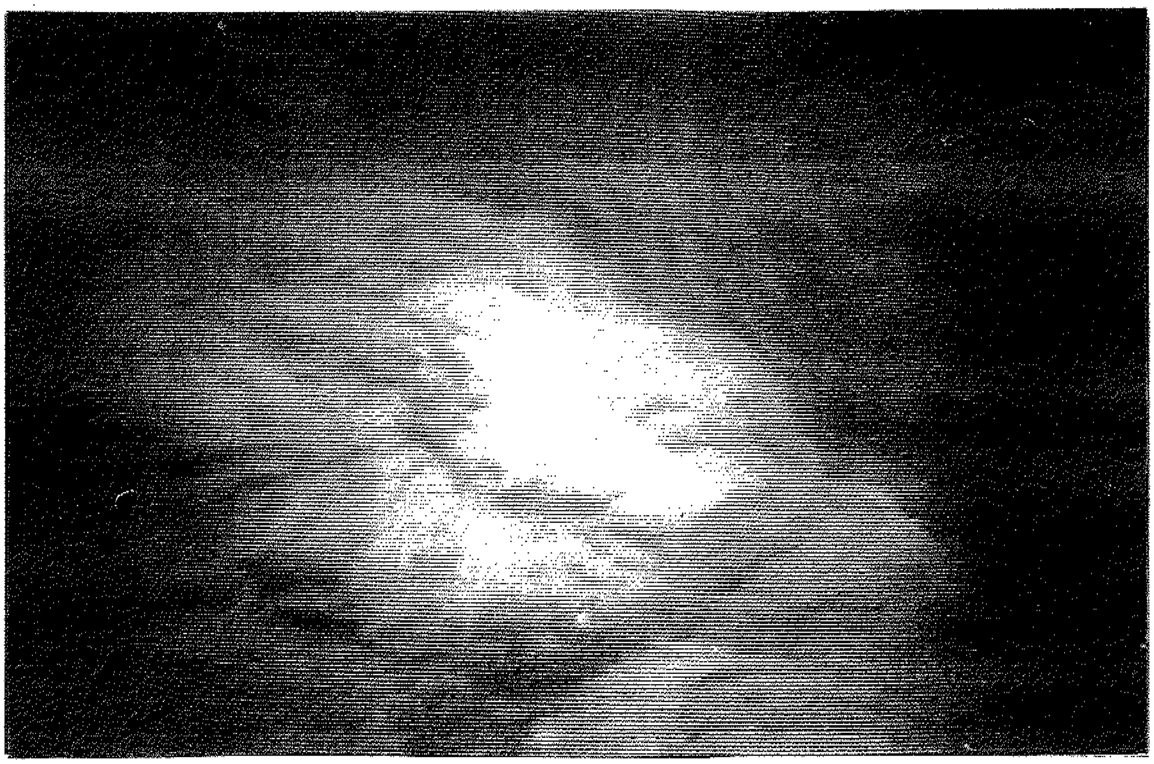
PHOTOGRAPH 13 Loose rock and concrete rubble located around Pier 1.



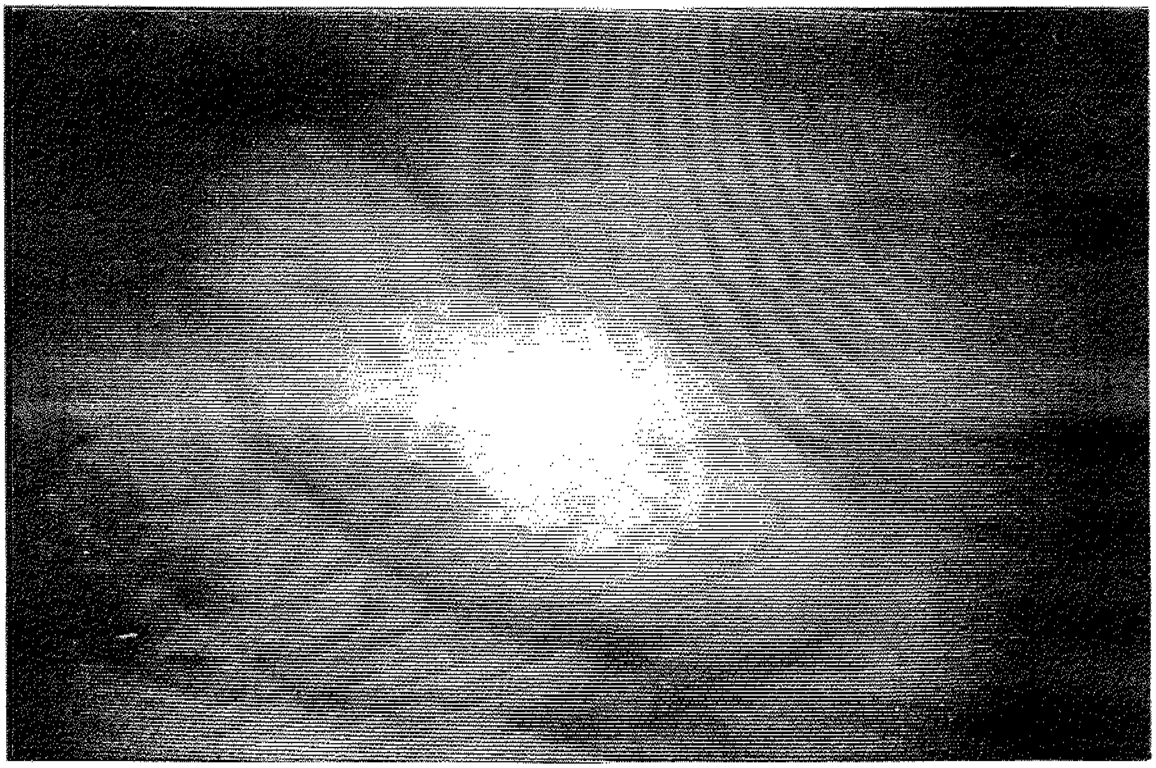
PHOTOGRAPH 14 Concrete sacks as found around Pier 1 and on the river bed slope below Pier 1.



PHOTOGRAPH 15 Concrete sacks located below the footing of Pier 1.

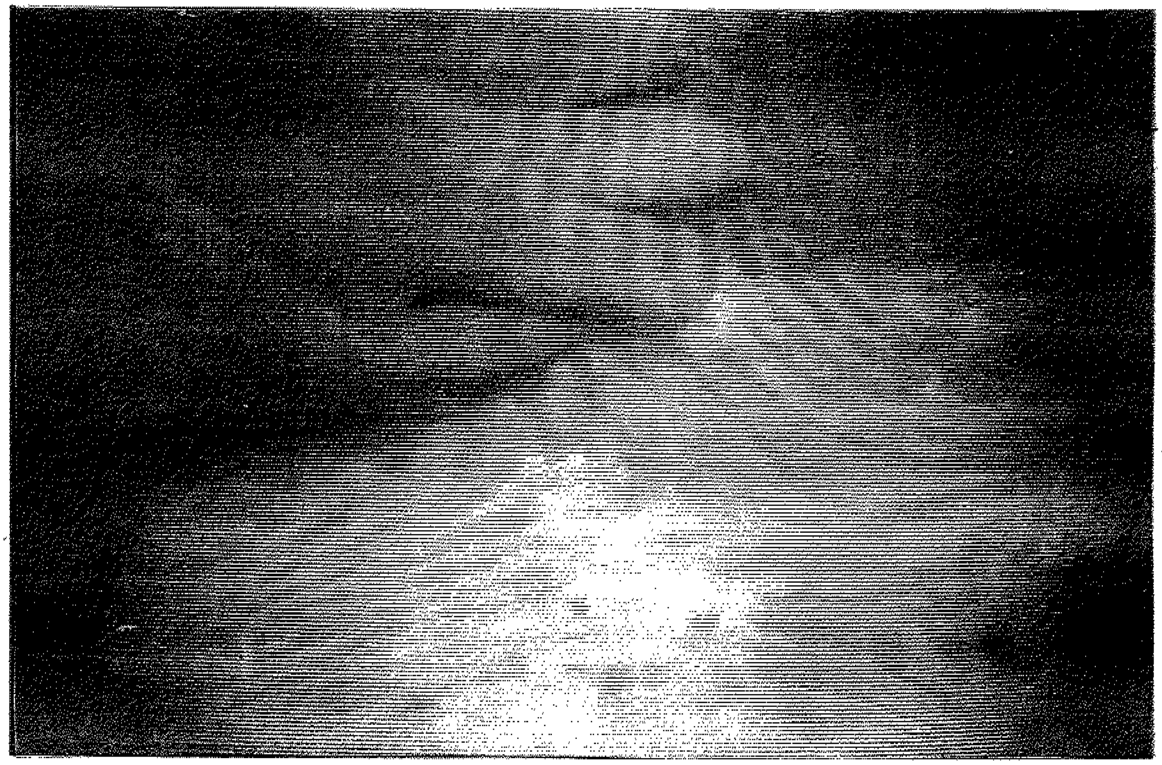


PHOTOGRAPH 16 Concrete sacks located beside the footing of Pier 2.



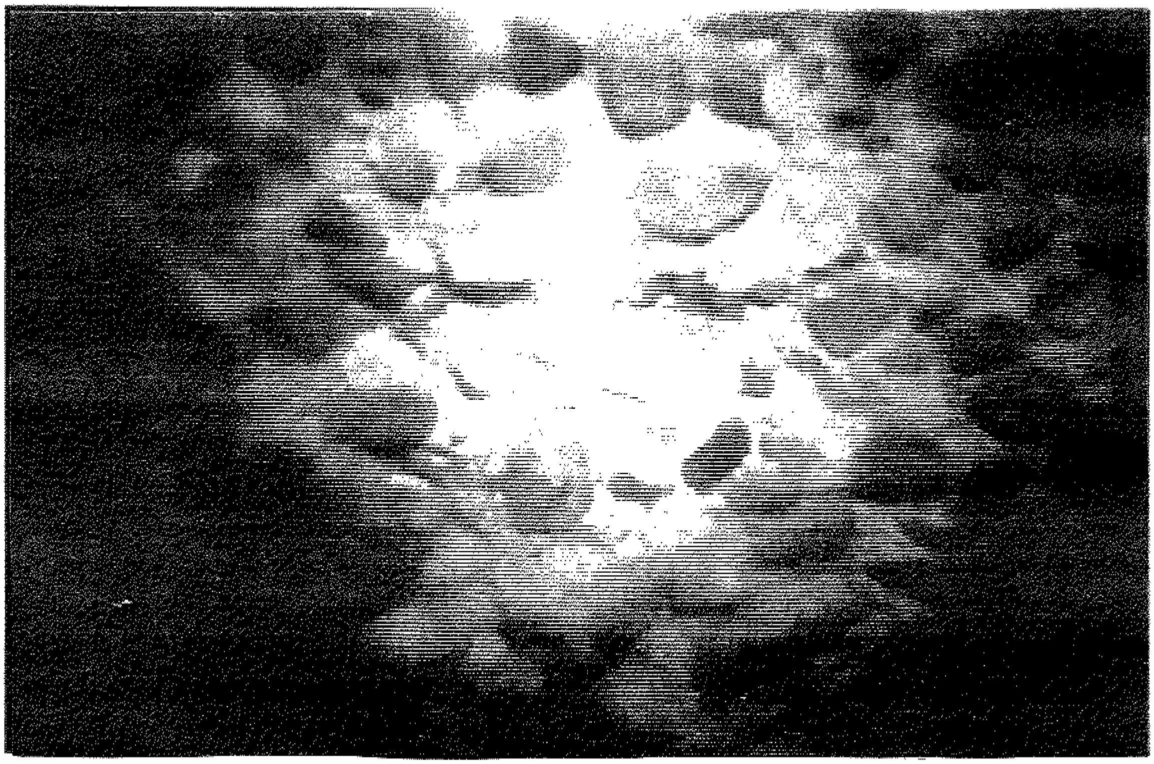
PHOTOGRAPH 17 Concrete sacks located on the riverbed slope below Pier 2.

PHOTOGRAPH 18 Remnant 2 x 4 planks from original footing form work near the base of the Pier 1 footing.



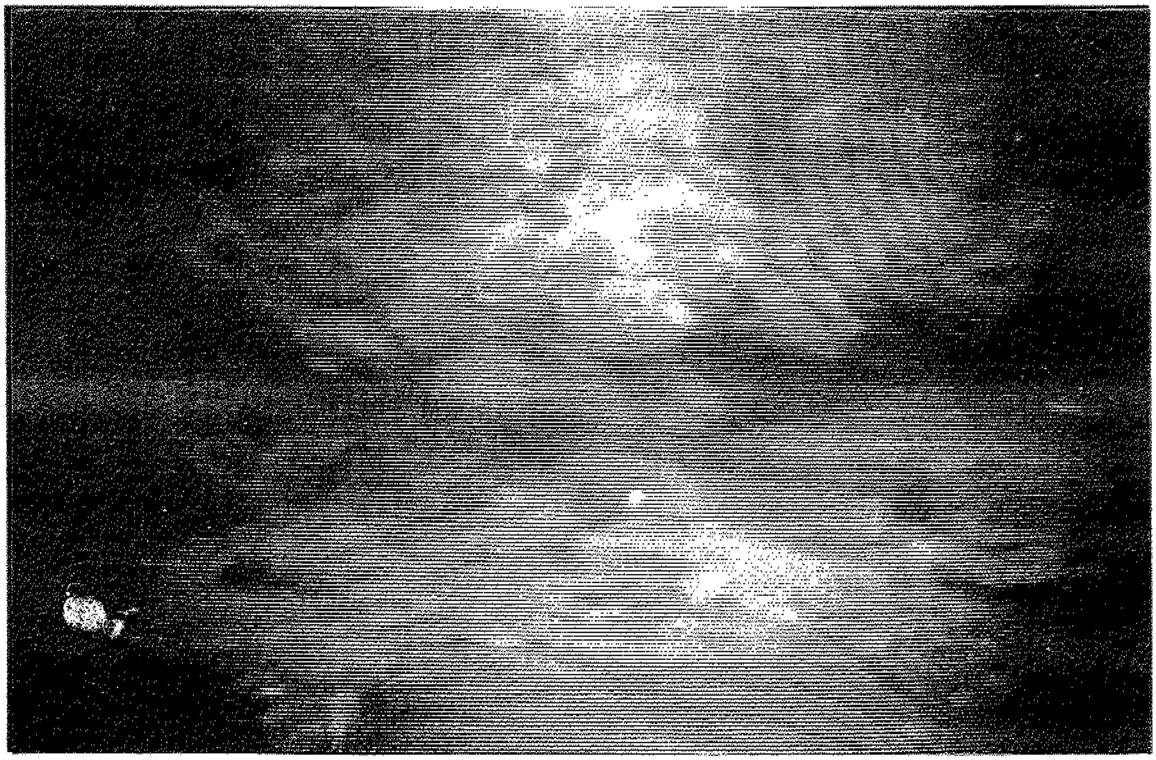
PHOTOGRAPH 19 Joint between the footing and the concrete of Pier 2.

PHOTOGRAPH 20 Concrete surface of footing (Pier 2). Note exposed aggregate.
For scale note divers hand on right.



PHOTOGRAPH 21 Concrete surface of footing (Pier 2). Note exposed aggregate 20 to 30 mm in diameter.

PHOTOGRAPH 22 Gap below footing of Pier 2. Gap evident at right of photograph.



PHOTOGRAPH 23 Gap below footing of Pier 2.

Appendix B

FIGURES

OVERSIZE DRAWING

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Ontario

FOUNDATION DESIGN SECTION

foundation investigation and design report

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 29-88-01 DIST 18

HWY 548 STR SITE 38 S-177

Munroe Island Bridge at Hwy. 548

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Foundation Investigation Report
For
Munroe Island Bridge at Highway 548
W.P. 29-88-01, Site 38 S-177
District 18, Sault Ste. Marie

INTRODUCTION

This report summarizes the results of the foundation investigation conducted at Munroe Island Bridge of Highway 548. The investigation was carried out upon the request of the Northwestern Region Structural Section for the proposed replacement bridge over Munroe Island channel of St. Mary's River. The field work for the investigation was carried out between 92 11 03 and 92 11 09 and consisted of six (6) sampled boreholes.

This report is applicable to the bridge structure and its immediate approaches (sta 13 + 380 to 13 + 470 approximately)

SITE DESCRIPTION

The site is located on Highway 548 at the southmost crossing from St. Joseph Island to the mainland, in the Township of St. Joseph, District of Algoma. It is about 3.4 km. south of the intersection of Highway 17 and Highway 548. According to Northern Ontario Engineering Geology Terrain Study 97 carried out by Ministry of Natural Resources, the landform of the site is a ground moraine with bedrock below a drift veneer.

The existing Munroe Island Bridge on Highway 548 is part of a causeway link joining St. Joseph Island and the mainland. The bridge was constructed in 1950 as a single span bridge founded on a pad of rockfill which formed part of the original causeway. Since construction, the bridge has undergone a history of distress and various remedial measures have been implemented. The present bridge has a three span configuration with short girder type spans added to each end. The piers are found to be tilting forward and sideways. Tie back cables have been installed between the pier and the south abutment. The existing pavement is in poor conditions with cracks and patches. The steel members of the bridge truss is rusty and some concrete spalling is noted in the structure.

The existing highway embankment at the approaches are up to about 10 m high with about half of it under water. The embankments are granular in nature with large rock armouring blocks on the slope face especially at the approaches. Approach slope gradients are up to 1.4H:1V to 1.5H:1V steep. The slopes appear to be marginally stable in local steep areas. The Munroe Island Channel is about 35 m wide at the bridge location. The sub-aqueous slopes are fairly steep around the bridge and to the east. Some whirlpools can be seen on the east side of the bridge. During the time of the investigation, the water level was at about El.176.4 m.

INVESTIGATION PROCEDURES

Soil data and inherent properties were obtained by in situ and laboratory testing. The procedures employed are discussed below.

Field

The field work for the investigation was carried out between 92 11 03 and 92 11 09 and consisted of six (6) sampled boreholes, which were advanced to depths of 3.1 to 16.3 m.

For the boreholes on land (BH1 and BH2) a track mounted continuous flight auger machine was used to advance the boreholes with conventional hollow stem augering techniques. For the boreholes over water (BH3 to BH6), a diamond drill rig resting on a raft and equipped with N/B size casings was used to advance them.

The sampling program consisted of split spoon samples collected in the overburden. Disturbed subsoil samples were retrieved by a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). They provided Standard Penetration (N) values for assessment of the denseness of the non-cohesive material. All the samples collected were used for identification and laboratory testing purposes. Dynamic core Penetration Test was carried out in BH3, BH4 and BH6. Conventional rock coring methods were applied in retrieving rock core samples using BXL core barrels.

All subsoil samples were identified in the field and returned to the laboratory for further examination and appropriate testing.

Groundwater levels were monitored throughout the duration of the investigation in open boreholes on land. All the land holes were backfilled upon completion of the field work.

Survey information related to the location and elevation of boreholes was provided by the Northwestern Region, Surveys and Plans Section.

Laboratory

The laboratory testing program for select soil samples consisted of:

- grain size distribution
- natural moisture content determinations

Laboratory test results are given in the following section of this report and Record of Borehole sheets included in the Appendix.

SUBSURFACE CONDITIONS

General

The Record of Borehole sheets in the Appendix illustrate the subsurface conditions at the borehole locations. The locations and elevations of the boreholes are shown in DWG. No. 298800-A.

BH1 and BH4 are on or close to the existing embankment. In BH1 to 3, the subsurface material encountered comprised of fill over bedrock. A thin layer (1.3m) of native material consisting of silty sand, some gravel, with boulders was found in BH4 overlying bedrock. In BH5 and BH6 which are further away from the shore, the bedrock is overlain by a thin layer of river bed material. The material is a silty sand and gravel, some boulders in BH5 and a silty sand with organics, trace gravel, occasional cobble and boulder in BH6.

Following are the specific descriptions of the material encountered in the investigation.

Silty Sand with Gravel, Occasional Shale Fragments (Granular Fill)

This is the embankment fill material and is contacted to a depth of 9.0 to 10.5m in BHI and BH2, and 5.1 to 5.8m below water surface in BH3 and BH4. It is described as silty sand with gravel, occasional shale fragments. The material typically becomes sand and gravel below water. The Standard Penetration Resistance "N" value ranged from 4 to more than 100 blows/0.3m. They may not be indicative of the denseness of the stratum due to the erratic nature of fill.

Typical properties of the material, as determined by laboratory tests on representative samples are summarized as follows:

Property	Range	No. of Test
Natural Moisture Content (W)	9.5 - 19.0	4
Grain Size Distribution (%)		4
Gravel	32 - 57	
Sand	24 - 49	
Silt and Clay	7 - 25	

Silty Sand and Gravel, Some Boulders

This native material is encountered in BH4 and BH5 to a depth of about 6 m below water surface. The thickness of this layer is about 0.4 to 1.3 m, it is described as silty sand and gravel, some boulders. The Standard Penetration Resistance "N" values are typically greater than 100 blows/0.3m due to the gravel content and boulders.

Typical properties of the material, as determined by laboratory tests on representative samples are summarized as follows:

Property		Range	No. of Test
Natural Moisture Content (W)		12.0 - 16.0	4
Grain Size Distribution (%)			4
	Gravel	48 - 50	
	Sand	36 - 45	
	Silt and Clay	7 - 14	

Silty Sand with Organics, Trace Gravel, Occasional Cobble and Boulder

This 0.8 m thick organic layer is contacted in BH6. The material is described as silty sand with organics, trace gravel, occasional cobble and boulder. Standard Penetration Resistance "N" value obtained does not represent the denseness of the material as the sampler encountered boulders during driving. Laboratory test on the sample indicates a high moisture content of 50.5% due to the presence of organics. Grain size distribution determined in the laboratory is 9% gravel, 56% sand and 45% silt and clay.

Bedrock

Bedrock is encountered at elevations of 170.0 to 173.3m. Length of rock drilled ranges from 0.3m in BH5 to 6.71 m in BH4. They are used for rock quality determination and classification.

Detailed descriptions of the rock are attached in the Appendix entitled "Rock Core Description". Two types of bedrock are found at the site. At the north abutment location, the bedrock is a Gabbro of igneous origin. The rock is considered strong with core recoveries ranging from 80 to 100%. At the south abutment location, the bedrock is a Quartz Sandstone of sedimentary origin. Rock cores from BH2 have core recoveries of 65 to 96%, and the rock is generally strong. At BH4, the rock is kaolinitic and shattered with poor core recoveries at the top 1.3 m, ranging from 30 to 58% The rock at this location is considered to be medium strong.

Groundwater

During the course of the investigation, the water level in the river was at about El.176.4 m. For the boreholes on land, observations of the groundwater level were carried out by measuring the water levels in open boreholes during the investigation. Groundwater levels measured were typically close to the water level in the river.

Water levels are subject to seasonal fluctuations and hence may vary from values given in this report.

DISCUSSION AND RECOMMENDATION

General

The existing structure on Highway 548 at the Munroe Island River crossing was constructed in 1950. It was originally constructed as a single span structure with abutments resting on rock fill which formed part of the original causeway. Since construction, serious settlement problems and foundation movement have occurred throughout the life of the bridge. In 1953, the material behind the abutments was cut back and short spans were added to each end of the main span, resulting in a three span configuration. In 1960s, various attempts including grouting the rockfill and rejacking the girders have been made to remedy the distressed situation. In 1974, tie-back cables were installed between the pier and the south abutment. Further monitoring and inspections indicate continual foundation movements.

The project comprises construction of a replacement bridge to the west of the existing one. The proposed centerline is 17.5 ± m offset from the existing at the north and 15.0 ± m at the south. The clear distance between the two bridges is about 3 m at the south end and 7 m at the north. The proposed structure is a 50 m long single span bridge. The proposed grades are similar to the existing structure, at EL. 182 ± m. The proposed width of the structure is 11 m with one lane and a shoulder each way and a 1.5 m wide sidewalk on one side. The existing bridge will serve as detour during construction, and will be removed after construction of the new bridge.

Foundation

According to the investigation results, the subsurface stratigraphy at abutment locations typically comprises fill over bedrock. Since the existing bridge will remain in service during construction, excavation of existing fill has to be kept a minimum to avoid undermining the existing embankment.

The structure can be supported by caissons socketed into bedrock. Liners should be used during excavation to stabilize the hole and surrounding ground. Boulder obstructions are expected when the excavation advances through the existing rock armouring layer.

Caissons should be socketed at least 300 mm into bedrock. Lateral loads and uplift on caissons may be taken by caisson sockets. Alternatively, dowels may be provided at the base of the caissons to provide the resistance. The recommended minimum diameter for the caissons is 910 mm to allow for downhole inspection and hand-cleaning. Before concrete is poured, the loose material at the bottom of the caisson hole should be hand-cleaned or air-lifted. Tremie concrete is recommended for under water concreting. The following factored end bearing values are recommended for the purpose of O.H.B.D.C. Depending upon the final design caisson sizes and socket lengths, we are pleased to recommend upon contributions from shaft friction.

	Factored Capacity at U.L.S.
North Abutment (Gabbro)	7500 kPa
South Abutment (Kaolinitic Quartz Sandstone)	3000 kPa

S.L.S. type II does not govern for unyielding founding material.

The caissons should be founded into sound bedrock. For preliminary design purpose, the estimated caisson bottom elevations are as follows:

	Estimated Caisson Bottom Elevation (M)
North Abutment (West Side)	170.0
North Abutment (East Side)	171.5
South Abutment (West Side)	167.5
South Abutment (East Side)	171.0

Alternatively, driven piles may also be considered. For steel H-piles driven to bedrock, the following design capacities in accordance with the O.H.B.D.C. are recommended.

	HP310 X 79	HP310 X 110
Factored Axial Capacity at U.L.S.	1150 kN	1600 kN
Axial Capacity at S.L.S. Type II	890 kN	1150 kN

Again, it is anticipated that the piles would encounter boulder obstructions from the existing rock armouring layer. The obstructions at pile locations will have to be removed by pre-augering or other means and replaced with granular materials.

The new fill material at pile locations should have a maximum grain size limited to 75 mm. Piles should be equipped with driving shoes to minimize tip damage. Lateral loads may be taken by the horizontal component of batter piles.

For preliminary design purpose, the estimated pile tip elevations are as follows:

	Estimated Pile Tip Elevation (M)
North Abutment (West Side)	170.5
North Abutment (East Side)	172.5
South Abutment (West Side)	170.0
South Abutment (East Side)	171.5

Due to the anticipated obstructions, pile driving should be closely monitored to ensure that the piles are founded on bedrock. If this alternative is chosen, please contact our office for further recommendations on pile driving.

Alternatively, consideration may also be given to shallow foundation. However, since the new alignment is close to the existing, there are limitations regarding excavation on existing embankment in order not to undermine it. Accordingly, the footings will have to be supported on a granular pad perched over existing fill. The existing fill is poorly graded with a rock armouring layer along the shore. It is imperative that all voids be chinked with well graded granular material or mass concrete. There are concerns on the integrity of the existing fill and river erosion, in view of the history of distress of the existing bridge. If this alternative is adopted, please contact our office for details.

Earth Pressure

Backfill to abutments should consist of granular material in accordance with MTO Standard Special Provision No. 121 (83 10).

Computation of earth pressures should be in accordance with Section 6.6.1.2.1. of the O.H.B.D.C. The active condition will govern earth pressure design for the yielding case while the at-rest condition will govern for the unyielding case. The follow properties for backfill are recommended.

Material	ϕ	γ	Ka	Ko
Granular "A"	35°	22.8 kN/m^3	0.27	0.43
Granular "B"	30°	21.2 kN/m^3	0.33	0.50
Rockfill	35°	18.0 kN/m^3	0.27	0.43

Slope Stability

The Munroe Island Channel has a V shaped river bed with fairly steep sub-aqueous slopes. For design purpose, the following slope geometries may be used. Rockfill slopes may be formed at 1.5H:1V up to 6 m and with a 2 m wide berm every 6 m up to a maximum height of 14 m so that no uninterrupted slope is greater than 6 m. If earth fill is used, the slopes should be formed at 2H:1V gradient with a 2 m wide berm incorporated at 8 m for embankment heights above 8 m to a maximum height of 14 m.

Only relatively free draining granular material or rockfill should be used underwater. Substantial underwater construction and filling will be required for the abutment forward slopes due to the relatively steep existing river bank gradient. This will affect the navigation channel and may also create hydrological and environmental concerns.

Alternatively, a closed abutment may be used. Consideration should be given to reinforced earth abutment construction. This support system enables the vertical loads be taken by the caissons/piles and the lateral loads by the reinforced earth wall. It has higher flexibility and does not require extensive shoring requirements.

Construction Considerations

Based on the investigation results, some organic material is found at the river bed. It is recommended that the organic material, if encountered within the area of construction, should be removed prior to filling. All embankment slopes should be protected from erosion by means of rock armouring as per hydrological requirements.

Provisions should be made in the contract document to take into account possible obstructions during pile driving or caisson construction.

CLOSING REMARKS

In the recommendations given above, we have provided a number of foundation alternatives as well as options on the approach abutment construction. In our opinion, pile driving concept is not the most viable alternative and caisson foundation appears to be a more feasible solution for this particular site.

Prior to the development of general arrangement drawing and during the design stage, technical discussion between this office and the structure office and Northwestern Region Structural Section is warranted to achieve the most cost-effective solution.

MISCELLANEOUS

The field work for this investigation was carried out under the supervision of D. Kwok, Project Foundation Engineer. The equipment was owned and operated by Master Soil Investigation Ltd. Bedrock was examined and classified by MTO petrographer, D. Williams.

The project was carried out by D. Kwok under the supervision of B. Iyer, Senior Foundation Engineer. This report was prepared by D. Kwok, reviewed by B. Iyer, and approved by M. Devata, Chief Foundation Engineer.

LICENSED PROFESSIONAL ENGINEER
W. D. KWOK
93 02 01
PROVINCE OF ONTARIO

D. Kwok, P. Eng.
Project Foundation Engineer

M. Devata, P. Eng.
Chief Foundation Engineer

APPENDIX

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

RECORD OF BOREHOLE No 1

METRIC

W.P. 29-88-01 LOCATION Sta 13+401.2 12.3 m Lt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE H.S. Auger, BW Casing COMPILED BY DK

DATUM Geodetic DATE 92 11 04 - 92 11 05 CHECKED BY BI

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.2	Ground Surface					182		
0.0	Silty Sand and Gravel, Trace Clay, Occasional Shale Fragments, Brown and Grey (Fill)	1	SS	11		180		
		2	SS	6		178		
		3	SS	21	▼*		o	52 24 16 8
		4	SS	12		176		
		5	SS	46		174		
173.2		6	SS	100	/28cm			
9.0	Gabbro Bedrock	7	RC	REC	94%			RQD 26%
		8	RC	REC	100%	172		RQD 67%
		9	RC	REC	100%	170		RQD 41%
167.9		10	RC	REC	100%			RQD 67%
14.3	End of Borehole							
	* 92 11 05							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ◇ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W.P. 29-88-01 LOCATION Sta 13+457.4 8.3 m Lt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE H.S. Auger, BW Casing COMPILED BY DK

DATUM Geodetic DATE 92 11 03 - 92 11 04 CHECKED BY BI

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.3	Ground Surface					
0.0	Silty Sand and Gravel, Occasional Shale Fragments (Fill)	1	SS	5		
	Brown / Grey	2	SS	22		32 43 18 7
		3	SS	11		
		4	SS	20		
		5	SS	11	▼*	
		6	SS	11		
		7	SS	4		44 49 5 2
	Sand and Gravel, Occasional Cobbles and Boulders	8	SS	7		
		9	SS	8		
171.8 / 10.5	Quartz Sandstone Bedrock	10	RC	REC	60%	RQD 0%
		11	RC	REC	75%	RQD 0%
		12	RC	REC	50%	RQD 0%
		13	RC	REC	96%	RQD 43%
		14	RC	REC	77%	RQD 0%
		15	RC	REC	88%	RQD 0%
166.0		16	RC	REC	96%	RQD 30%
16.3	End of Borehole					
	* 92 11 03					

Elevation scale: 182, 180, 178, 176, 174, 172, 170, 168

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

$+^3$, $\times^5$: Numbers refer to Sensitivity

15◇5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3

METRIC

W.P. 29-88-01 LOCATION Sta 13+402.6 3.6 m Rt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE Washboring, BW Casing, Cone COMPILED BY DK

DATUM Geodetic DATE 92 11 06 - 92 11 08 CHECKED BY BI

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.4	Water Surface									
0.0	Water					176				
						174				
173.0 / 3.4	Sand and Gravel Trace Silt, Trace Organics Occasional Cobbles and Boulders (Possible Fill)	1	SS	12		172				
		2	SS	21						
		3	SS	28			50/ 23cm Bouncing on rock	o		57 30 9 4
170.6 / 5.8	Gabbro Bedrock	4	RC	REC	100%	170				RQD 28%
		5	RC	REC	100%					RQD 33%
		6	RC	REC	80%					RQD 0%
		7	RC	REC	100%					RQD 11%
		8	RC	REC	95%	168				RQD 44%
166.9 / 9.5	End of Borehole									

Dynamic cone penetration resistance plot: 20 40 60 80 100. Shear strength kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100.

Water content: PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; 20 40 60.

$+^3$, $\times^5$: Numbers refer to Sensitivity

15-20-5 over 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 4

METRIC

W.P. 29-88-01 LOCATION Sta 13+453.9 5.3 m Rt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE Washboring, BW Casing, Cone COMPILED BY DK

DATUM Geodetic DATE 92 11 03 - 92 11 06 CHECKED BY BI

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
176.4	Water Surface
0.0	Water
173.2 / 3.2	Sand and Gravel (Granular Fill)
171.3 / 5.1	Silty Sand and Gravel, Boulders
170.0 / 6.4	Highly Fractured Quartz Kaolinitic Sandstone Bedrock
164.1 / 12.3	End of Borehole

SAMPLES

NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	15			
2	SS	100	/10cm	o	51 35 10 4
3	SS	60			
4	RC	REC	50%		RQD 0%
5	RC	REC	26%		RQD 0%
6	RC	REC	28%		RQD 0%
7	RC	REC	58%		RQD 0%
8	RC	REC	30%		RQD 0%
9	RC	REC	92%		RQD 50%
10	RC	REC	83%		RQD 38%
11	RC	REC	100%		RQD 35%
12	RC	REC	70%		RQD 45%

ELEVATION SCALE: 176, 174, 172, 170, 168, 166

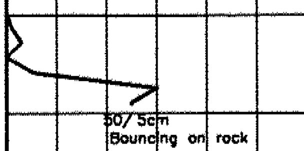
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3



$+^3, \times^5$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 5

METRIC

W.P. 29-88-01 LOCATION Sta 13+390.5 12.5 m Rt ORIGINATED BY DK

DIST 18 HWY 548 BOREHOLE TYPE Washboring, BW Casing COMPILED BY DK

DATUM Geodetic DATE 92 11 08 CHECKED BY BI

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.4	Water Surface										
0.0	Water						176, 174, 172				
170.9 / 5.5	Sand and Gravel, Some Boulder		1	SS	**				o		48 45 4 3
170.1	*		2	RC	REC	67%					RQD 0%
6.3	End of Borehole										

* Quartz Sandstone,Gabbro Bedrock

** Bouncing on rock

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

$+^3$, x^5: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 6

METRIC

W.P. 29-88-01 LOCATION Sta 13+465.5 15.1 m Rt ORIGINATED BY DK
DIST 18 HWY 548 BOREHOLE TYPE Washboring, Cone COMPILED BY DK
DATUM Geodetic DATE 92 11 08 - 92 11 09 CHECKED BY BI

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.4	Water Surface										
0.0	Water						176				
174.1											
2.3	Silty Sand with Organics, Trace Gravel Occasional Cobble and Boulder Dark Brown		1	SS	25 /23cm bouncing on boulder		174		o		9 46 39 6
173.3											
3.1	End of Borehole Probable Bedrock							50/ 8cm bouncing on rock			

$+^3$, $\times^3$: Numbers refer to Sensitivity

15 ⟡ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

ROCK CORE DESCRIPTION
WP 29-88-01

	CORE RECOVERY				CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	7	9.02-9.65	94	26	9.02-14.30	**GABBRO** (chloritic), dark greenish grey to greenish black (stained dark reddish brown, 10.84-12.37 m); coarse to fine grained; strong; unweathered to slightly weathered; fractures moderate to extremely close spaced, dipping to near vertical, undulating to planar, smooth.
	8	9.65-11.10	100	67		
	9	11.10-12.70	100	41		
	10	12.70-14.30	100	67		
2	10	10.49-11.51	60	0	10.49-16.31	**QUARTZ SANDSTONE,** medium grey; fine to medium grained; strong; unweathered to slightly weathered; fractures close to extremely close spaced, near vertical to flat, undulating to planar, smooth to rough.
	11	11.51-12.22	75	0		
	12	12.22-12.93	50	0		
	13	12.93-14.35	96	43		
	14	14.35-15.01	77	0		
	15	15.01-15.62	88	0		
	16	15.62-16.31	96	30		
3	4	5.82-6.27	100	28	5.82-9.47	**GABBRO** (chloritic), dark greenish grey to greenish black; coarse to fine grained; strong; unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating, smooth.
	5	6.27-6.73	100	33		
	6	6.73-6.86	80	0		
	7	6.86-7.80	100	11		
	8	7.80-9.47	95	44		

***CR** = CORE RECOVERY *(NOTE: Depths are approximated where core recovery is less than 100%)*
***RQD** = ROCK QUALITY DESIGNATION

Logged by: DAW, Soils and Aggregates Section

ROCK CORE DESCRIPTION
WP 29-88-01

	CORE RECOVERY				CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
4	4	5.61-6.22	50	0	5.61-6.43	**OVERBURDEN** (till: boulder of **QUARTZ SANDSTONE**, 5.92-6.22 m, light brownish grey; fine to medium grained; strong; unweathered to slightly weathered).
	5	6.43-7.29	26	0		
	6	7.29-8.56	28	0		
	7	8.69-9.30	58	0	6.43-12.32	**QUARTZ SANDSTONE** (kaolinitic), pale red; fine to coarse grained; medium strong; unweathered to slightly weathered; fractures moderate to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
	8	9.30-10.06	30	0		
	9	10.06-10.72	92	50		
	10	10.72-11.46	83	38		
	11	11.46-12.07	100	35		
	12	12.07-12.32	70	45		
5	2	5.97-6.27	67	0	5.97-6.10	**OVERBURDEN** (till: pebble of **QUARTZ SANDSTONE**(feldspathic), 5.97-5.99 m, greyish orange; fine to medium grained; strong; unweathered to slightly weathered).
					6.10-6.27	**GABBRO** (chloritic), dark greenish grey to greenish black; coarse to fine grained; strong; unweathered to slightly weathered; fractures very close to extremely close spaced, dipping to near vertical, undulating to planar, smooth.

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

METRIC

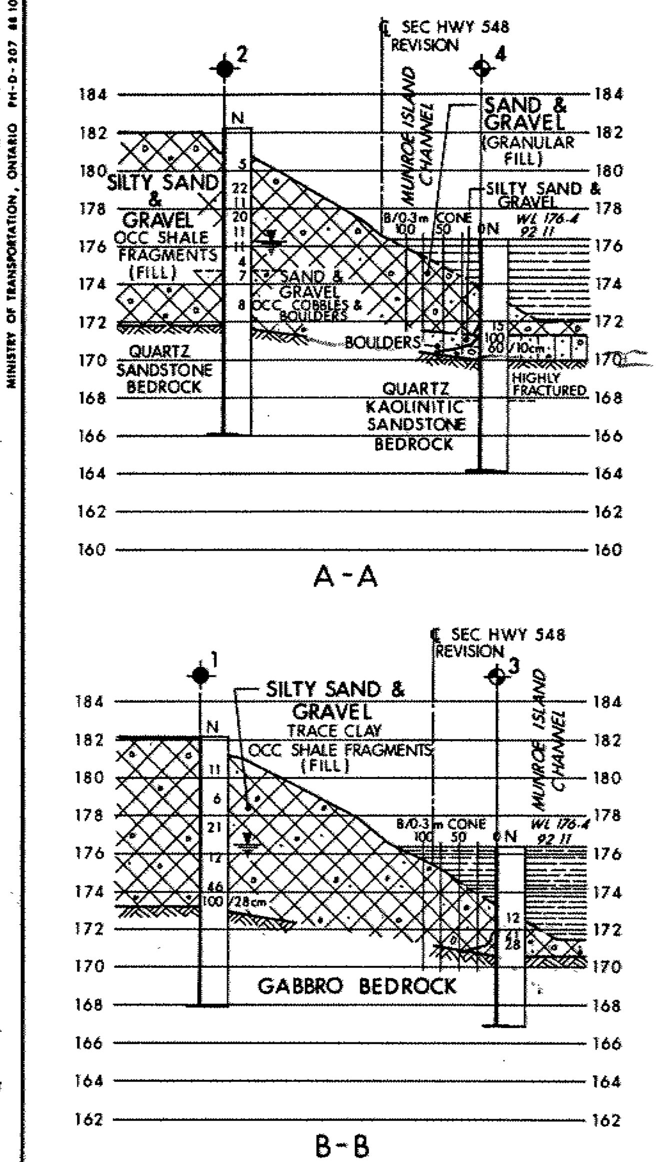
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No
WP No 29-88-01

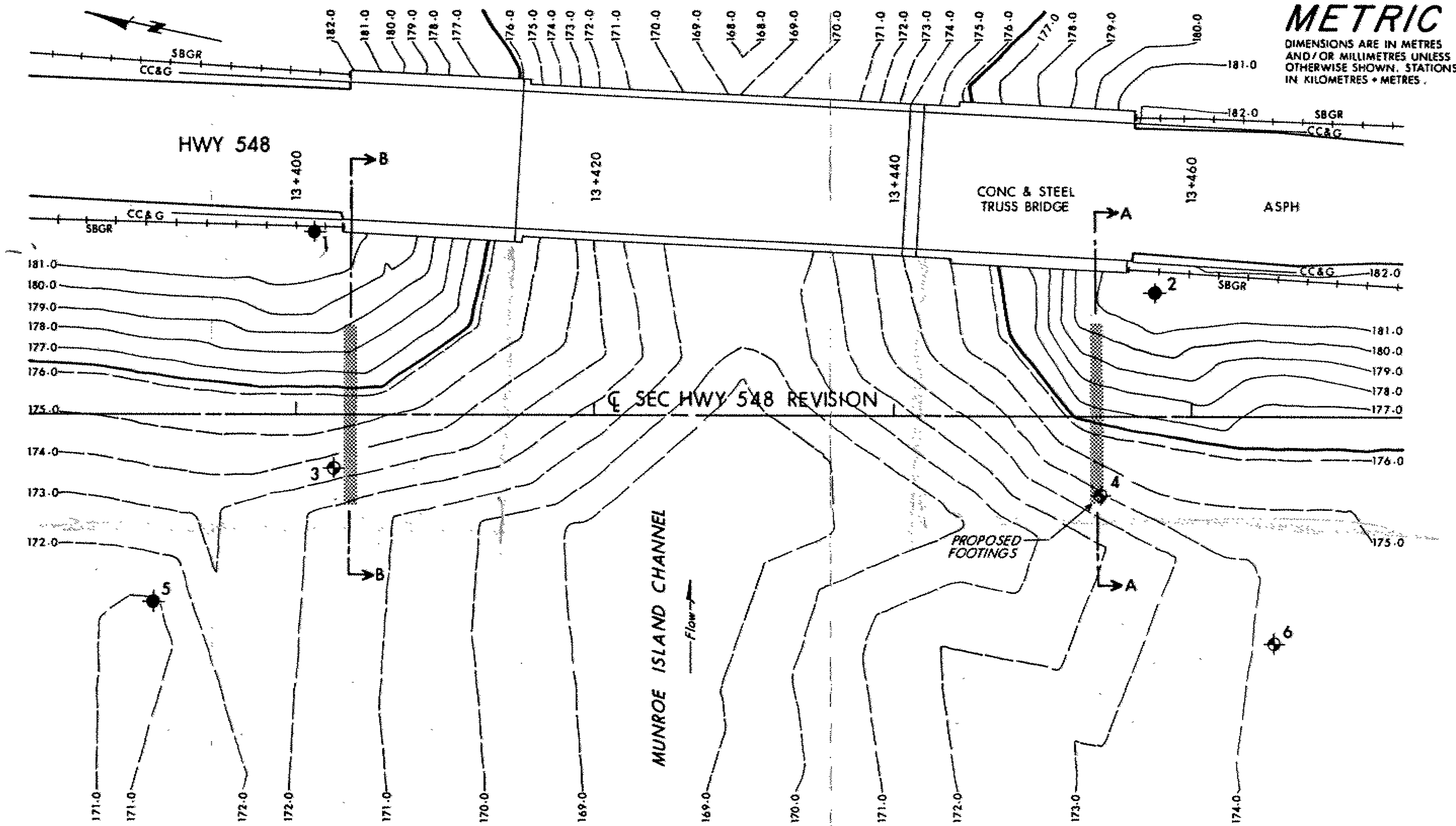
MUNROE ISLAND CHANNEL

BORE HOLE LOCATIONS & SOIL STRATA

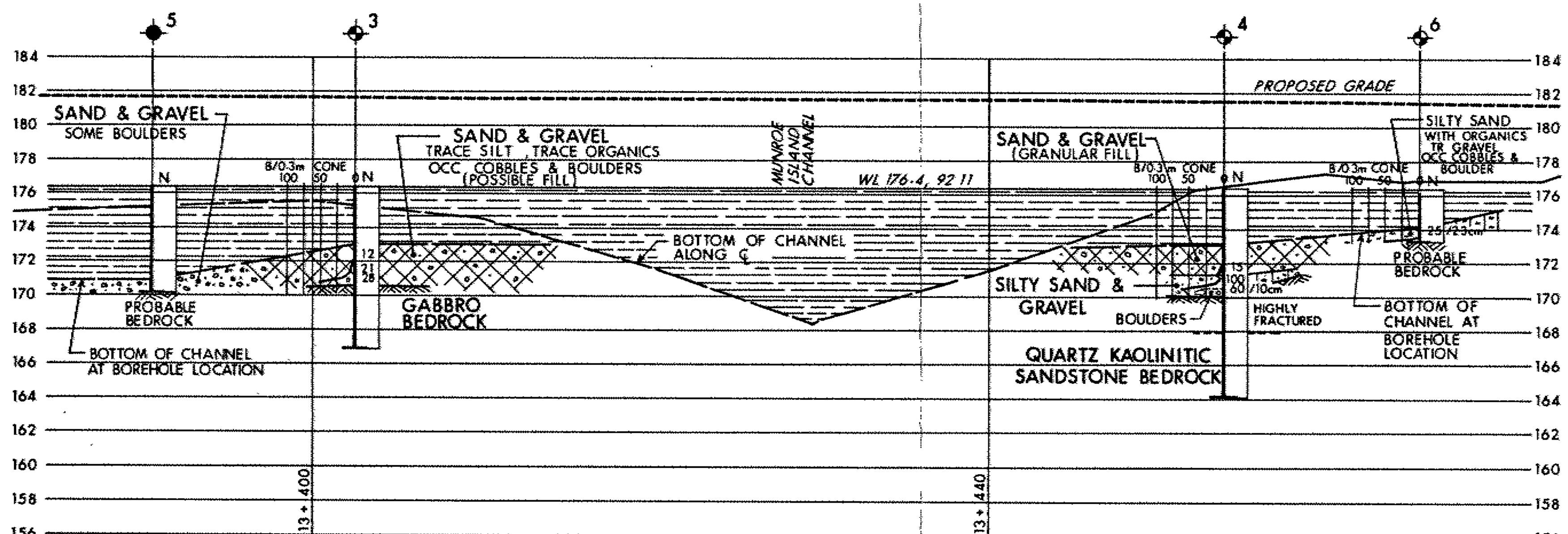
SHEET



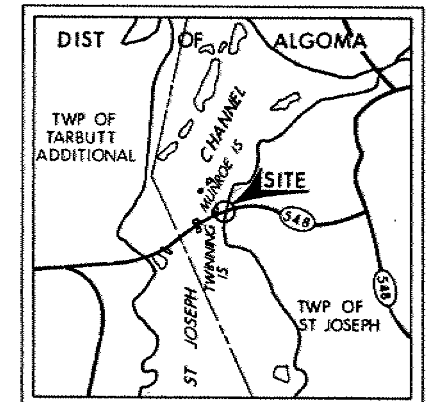
SECTIONS
SCALE 4m 0 4m



PLAN
SCALE 4m 0 4m



℄ PROFILE SEC HWY 548 REVISION
SCALE 4m 0 4m



KEY PLAN
SCALE 1km 0 1km

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 92 11

No	ELEVATION	STATION	OFFSET
1	182.2	13+401.2	12.3m Lt
2	182.3	13+457.4	8.3m Lt
3	176.4	13+402.6	3.6m Rt
4	176.4	13+453.9	5.3m Rt
5	176.4	13+390.5	12.5m Rt
6	176.4	13+465.5	15.1m 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.

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 GC 2.01 of OPS Gen Cond

REV.	DATE	BY	DESCRIPTION

Geocres No 41J-55

HWY No SEC 548			DIST 18
SUBM'D DK	CHECKED	DATE 1993 01 27	SITE 385-177
DRAWN DT	CHECKED	APPROVED	DWG 298801-A

REF No E-833-548-3, 92 10

Z → Tae

RECEIVED
DEC 07 1994
PAVEMENT DESIGN, EVALUATION & MANAGEMENT SECTION

MEMORANDUM

TO: Tae Kim
Sr. Foundation Engineer
Pavements and Foundations Section
Downsview

FROM: Structural Section
Northwestern Region
807/473-2064
807/473-2167 (fax)

DATE: December 2, 1994

RE: Review of Foundation Designs

This is in reply to your November 8, 1994, memorandum concerning Munroe Island Bridge.

For the most part, that memo dealt with technical foundation related issues which can be easily resolved. You saw fit in that memo, however, expressed additional concerns you have which relate to our working relationship. It is in reply to these concerns that I am writing this memo.

Specifically, you identified three projects where you indicate that we did not submit plans to your office for review, but where you had to resolve construction problems. Some clarification is in order.

1) **Contract 93-227 - Lamont Creek Culvert**

Our office designed this culvert in a matter of weeks after the project tenders had already been called. This was necessary in order to not delay the project. We were under extreme pressure to complete the work in time and could not have submitted plans to your office for review. Criticism of this office for not doing so is unwarranted.

2) **Contract 93-210 - White fish River Bridge**

Preliminary plans were submitted to your office in 1990. The fact that final plans were not submitted was an oversight and may have been related to the fact that this project was deferred by a number of years. Notwithstanding, to our knowledge, there were no foundation related problems during construction.

3) **Contract 94-228 - Munroe Island Bridge**

Your suggestion of no drawing review on this project is surprising. Our files are full of correspondence between your office, M.M. Dillon, and ourselves that occurred throughout the design process. We even met on-site with the Project Foundation Engineer to develop the foundation design concept. Upon design completion, plans were submitted to your office and the project is not yet out to tender. It's difficult to imagine a process whereby more consultation could have occurred. I do suggest that the confusion may have stemmed from the staff turn-overs and reorganization that occurred in your office.

I indicated to you on the phone that any recent occurrences where we did not provide plans to your office for review were not intentional, but rather, simple oversights. I have further asked my engineers to ensure that foundation plans are submitted for review. In turn, however, it would be appreciated if you could exercise tolerance and recognize that management of projects is frequently complicated beyond our control. It may simply not be possible for us to always comply with normal procedures.

R.J. Krisciunas
Head, Structural Section

RJK/af

cc - T. Kazierowski - Manager, Pavements and Foundations
- G. Cautillo - Sr. Manager, EMO
- G.E. Norman - Manager, Engineering
- F.M. Adams - Manager, Construction
- G. Al-Bazi - RSS Committee

memorandum

Ontario

To:	R,J. Krisciunas Head, Structural Section Thunder Bay	Date: 95 05 08
Att:	D. Dykstra	
From:	Foundation Design Section Rm. 315, Central Building Downsview, Ontario	
Re:	Munroe Island Bridge Hwy. 548, WP 29-88-01, Site 38S-177 District 18, Sault Ste. Marie	

In response to your request for a review of the retained soil system submitted by the Reinforced Earth Company this office has the following comments.

1. The foundations are suitable to carry the applied bearing pressures required for the retained soil system. In addition, no global stability problems are anticipated for this system placed on rockfill.

2. The bottom of the reinforced earth retaining wall should be placed above the highest water level.

3. Clear stone (maximum size less than 19 mm) with a thickness of 1 m should be placed on top of the 300 mm thick crushed rock (Max. 50 mm particle size) base. As per plan a Geotextile would than be placed on the Clear Stone below the Granular 'A'material.

4. A geotextile should be placed vertically between the reinforced earth walls and the Granular 'A' backfill material.

5. Since some differential settlement is expected to occur within the rockfill embankment, the reinforced earth wall should be designed to tolerate any long term settlement.

We have no further comments. If you have any questions, please contact this office.

M. Michalek
Jr. Foundation Engineer
For:
T. Kim
Sr. Foundation Engineer

7540-1479 (Rev 10/89)

memorandum

Ontario

To: T. Mills
Construction Supervisor
Northwestern Region

Date: 95 06 20

Att: Glen Carlson
Project Supervisor

Frm: Pavements and Foundations Section
Rm 315, Central Building, Downsview

Re: WP 29-88-02, Contract No. 94-228
Monroe Island Bridge Approach Fills
Sta. 13+160 to 13+220
Hwy. 548, Dist. No. 62, Sault. Ste. Marie

This memorandum is in response to a request for recommendations at the above site where failure of a rock fill slope has occurred. Observation at the site and subsequent monitoring by surveys indicate this to be of a rotational nature. This type of failure is common in constructed embankments and fills due to their homogeneous composition. Movement seems to be taking place at the toe of the slope as the slope tends to restore equilibrium by deceasing the driving moment and flattening the slope.

Sounding of the surface at the toe of the embankment indicate that the fill height is greater than 6 m, requiring a 2 m berm. It is our understanding that the fill height was underestimated, as the area of failure approaches the limits of the rock fill with any slope treatment being tapered off. Therefore it is the recommendation of this office to extend the 2 m berm and incorporate it in the zone of failure.

Construction of this berm could be accomplished by dumping armour stone just north of the failure are, creating a working platform from which the remedial work can be placed moving southward parallel to the embankment. A sketch of this slope treatment has already been faxed to Mr. Carlson.

We hope this meets with your requirements. If there are further questions please contact this office.

M. Michalek
Jr. Foundation Engineer
For:
T. Kim
Sr. Foundation Engineer

cc W. Prystanski, Head Geotechnical Section

memorandum

File

Ontario

To: Glen Carson
Construction Supervisor
Construction Office
160 McDougall Street
Sault Ste. Marie

June 2, 1995

From: Pavements and Foundations
Room 315, Central Building

Re: Site Meeting
Contract 94-228: Munroe Island Bridge
Contract 94-205: Two Three River Bridge

Further to the site meeting on May 18 and 19, 1995, this memo summarizes the results of our meeting.

Contract 94-22: Munroe Island Bridge

1. Caisson Inspection - Pavements and Foundations Section will provide a training of caisson installation to construction staff and caisson inspection services.

2. Rockfill Slope Failure- Glen Carson will provide the following information to foundation group as soon as possible to evaluate the rockfill slope stability.

   a) Cross Sections
   b) Sounding

Contract 94-205: Two Three River Bridge

- Required capacity of 4000 kN for piles could not be obtained to terminate pile driving during the construction.

- Actual capacity was calculated to be approximately from 2,000 kN to 3,000 kN using e = 0.32

- Since separate cushion was not used during the pile driving operation, e value can be increased to 0.4.

- New capacity is calculated to be about from 3.000 to 3.500 kN.

- Since the pile hammer appears to be small for driving HP 310 x 132 and the site is located on a secondary highway, the above capacity is acceptable.

If you have any questions concerning this memo, please contact this office.

Taechul Kim

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

TCK/mmj

c.c. - Tom Mills - Construction, Sault Ste. Marie
Ray Krisciunas - Structural Section
Wayne Prystanski - Geotechnical Section

FAX 705-246-0198

File

Ministry of Transportation
Ontario

FAXGRAM

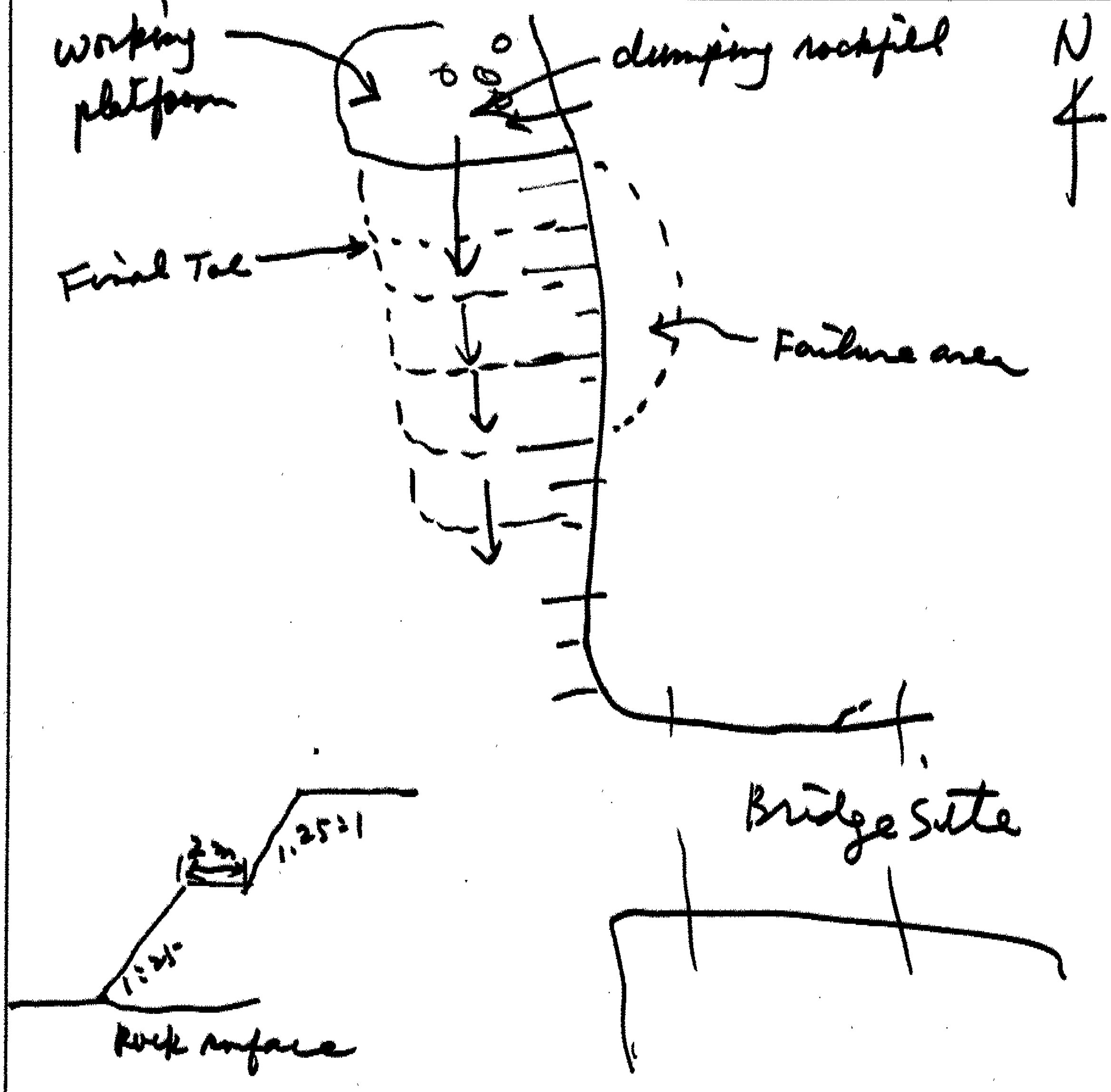
PLEASE TYPE

DATE June 9, 1995

PAGE 1 OF 1

TO: Martin Michalek Foundation Engineer	FROM: Tae. C. Kim Sr. Foundation Engineer Pavements & Foundations Section Tel: (416) 235-3731, Fax:(416)235-5240

SUBJECT: SLOPE TREATMENT



MEMORANDUM

Ontario

To: R.J. Krisciunas, P. Eng.
Head, Structural Section
Northwestern Region

Date: 1994 11 08

From: Pavements & Foundations Section
Room 315, Central Bldg.
Downsview

Tel: 235-3731
Fax: 235-5240

Re: Review of Contract Drawings
Munroe Island Bridge, Hwy 548
W.P. 29-88-01 (Cont. 94-228), Site 38S-177
District 18, Thunder Bay (West)

Further to the telephone conversation between you and the writer on October 24, 1994, we have reviewed the Contract Drawings and Special Provisions which you provided to us.

It is understood that the original design adopted battered caissons for the bridge foundation in order to resist the high moment and lateral loads due to the earth pressure from the bridge abutment walls. It is also understood that the forward slope was proposed to form at 2H:1V gradient with a 2 m wide rockfill mid-height berm.

Based on our initial review, this office suggested the following revisions on drawings (see our memo dated August 11, 1993).

1) Since the battered caissons would be very difficult to install and be quite expensive to construct especially through old as well as new rockfill, proposed battered caissons should be revised to vertical caissons.

2) Accordingly, reinforced earth wall concept could be utilized to minimize the lateral load on the caissons. Caissons would mainly be required to take vertical loads and this would reduce socketing considerably.

3) Alternatively, lightweight fill could also be considered to reduce the lateral load significantly.

We have reviewed the drawings and contract documents you provided on November 1, 1994. Based on our review, the following comments should be made.

1) As discussed in our previous memo (item 1, above) the construction of battered caissons would be extremely difficult and quite expensive. We therefore, strongly recommended that the battered caisson be revised to vertical caissons.

2) During the construction, caisson inspection should be carried out by an experienced person.

We recommend that the construction office retain such an experienced person with consultation from this section. We also enclose a Non-Standard Special Provision for caisson piles for your reference.

3) Proposed Reinforced Earth Retaining Walls is a viable option at this location, since the bottom of reinforced walls is higher than the seasonal river water level. However, it is recommended to raise the bottom elevation of RERW above high water level, if possible, in order to eliminate some flooding related failures. If the raising is not possible, other protection measure can be considered:

(2) a) use clear stone (maximum size less than 19 mm) as a backfill for bottom 1m and place Geotextile between clear stone and Granular "A" material

or

b) place Geotextile vertically between Reinforced Earth Walls and Granular "A" backfill material.

4) (4) Since some differential settlement is expected to occur within the rockfill embankment, reinforced earth wall should be designed to tolerate such a long term settlement.

In addition to the above comments, we should express our concerns about the review process of Preliminary and Final Drawings and Contract documents. In your memorandum to Mr. George Al-Bazi, Chairman of Retained Soil System (RSS) Committee on October 5, 1994, you clearly emphasized that you agreed and followed our recommendations with your full confidence. However, this section did not have an opportunity to review the actual design drawings for Reinforced Earth Retaining Wall until the project was finalized and is on tendering process.

Recently, it was found that many projects from your Region have been finalized without our preliminary and/or final reviews of foundation drawings. During 1994 construction season, we have been requested to attend site meeting to resolve the construction problems, such as pile driving, slope instability, dewatering problems etc. for the following projects, even though this section was not requested to review any contract drawings.

1) Contract 93-227, Lamont Creek - Dewatering Problem.

2) Contract 93-210, Hwy 588 and Whitefish River - No Final Review.

3) Contract 94-228, Munroe Island - Reinforced Retaining Walls - No Drawing Review.

We strongly recommend that in order to reduce construction problems and their related claims, Preliminary and Final Drawings should be submitted for review by this section as indicated in our mandate. Your cooperation for these concerns would be appreciated.

We have no further comments. If you have any questions, please contact this office.

Taechul Kim
Tae C. Kim, P. Eng.
Sr. Foundation Engineer

TCK/jb

cc: T. Kazmierowski - Manager, Pavements & Foundations
G. Cautillo - Sr. Manager, EMO
G. Norman - Manager, Engineering & Right-of-Way Office
F. Adams - Manager, Construction Office
G. Al-Bazi - RSS Committee

MEMORANDUM

RECEIVED
OCT 12 1994
FOUNDATION DESIGN SECTION

TO: G. Al-Bazi 5857
Chair, RSS Committee 5858
Structural Office
Downsview

FROM: Structural Section
Northwestern Region
807/473-2064
807/473-2167 (fax)

DATE: October 5, 1994

RE: Munroe Island Bridge, Site 38S-177 WP 29-88-01

This concerns your faxgram to Dick Dykstra on behalf of the RSS Committee regarding the Munroe Island Bridge. In it, you indicate that the committee recommends against the use of a reinforced soil wall at this site. There are a couple of issues that you should made aware which may change your opinion.

First of all, the Foundation Report for this structure very clearly indicated that a reinforced soil wall should be considered as a means of resisting lateral earth pressure behind the abutments. While, at first, we were not in favour of this system, we ultimately complied with the recommendation after various discussions with the Foundation Office and cost analyses by our consultant.

Secondly, from what I can deduce, the committee's reluctance stems from concerns about the effects that the water will have on the wall. While partial submergence of the wall is a possibility, it would not be a frequent occurrence since this is only associated with extreme events in the channel. Under these circumstances, the flow velocity would be minimal, precluding degradation of backfill material. Suggestions about forces upon the wall due to river dynamics are incorrect. The channel hydraulics and details of the wall are such that it simply cannot encounter any forces other than what it is designed for.

In closing, this reinforced soil wall was incorporated into the foundation design at the recommendation of the Foundation Office and based upon their first hand knowledge of the site. We relied upon their expertise, and we are fully confident that their recommendations are sound, and that the system will function as they intended.

Original Signed By:
R. J. Krisciunas

R.J. Krisciunas
Head, Structural Section

cc Tae Kim

RJK/af

w:\docs\krisciun\bazi.mun

memorandum

Ontario

To:	R.J. Krisciunas Head, Structural Section Thunder Bay	Date:	93 08 11
Attn:	D. Dystra		
From:	Foundation Design Section Room 315, Central Building Downsview, Ontario		
Subject:	Munroe Island Bridge Highway 548 W.P. 29-88-01, Site 38S-177 District 18, Sault Ste. Marie		

Further to our meeting with R. Radolli and P. Tam of M.M. Dillon Limited on 93 07 30, we provide herewith additional design parameters for preliminary estimating purpose:

- The caisson bottom elevations given in the foundation report are interpolated from available borehole information and have taken into account the recommended 300 mm min. socket. For preliminary design purpose, the founding elevations for intermediate caissons can be interpolated from the end caissons. Caisson bottom elevations have to be verified in the field during construction.

- Skin friction values, socket lengths required for fixity and allowable lateral bearing capacities for the two types of bedrock are provided as follows:

	Gabbro	Kaolinitic Quartz Sandstone
Skin Friction (Compression)	3000kPa	750kPa
Skin Friction (Uplift)	2000kPa	500kPa
Socket length required for fixity	1.0 m	1.5 m
Allowable lateral bearing capacity	2500kPa	1000kPa

Our initial review based on the structural loadings provided by Dillon via a fax transmittal dated 93 08 05 has revealed that extensive length of dowels up to 10 m is required from the bottom of the caisson sockets to resist the moment and lateral load from the structure. In our opinion, this design is too expensive and hence not feasible. We understand that a majority of the lateral load and moment is due to the soil load from the abutment walls. In view of this, we suggest alternative methods as follows:

1. Light weight fill can be employed behind the abutments in lieu of regular earth fill. This will reduce the lateral load significantly and thereby reduce the socket requirements.

2. Reinforced Earth concept can be utilized to minimize the lateral load on the caissons. Caissons will mainly be required to take vertical loads and this will reduce socketing considerably.

Our past experience suggests that batter caissons may be quite expensive to construct especially when they have to advance through old as well as new rock fill.

Should there be any questions, please contact our office.

David Kwok, P. Eng.
Project Foundation Engineer
for
Balu Iyer, P. Eng.
Senior Foundation Engineer

c.c. M.M. Dillon Limited
(Attn. R. Radolli)

DILLON
Consulting Engineers · Planners · Environmental Scientists

RECEIVED
AUG 5 1993
FOUNDATION DESIGN SECTION

FAX TRANSMITTAL

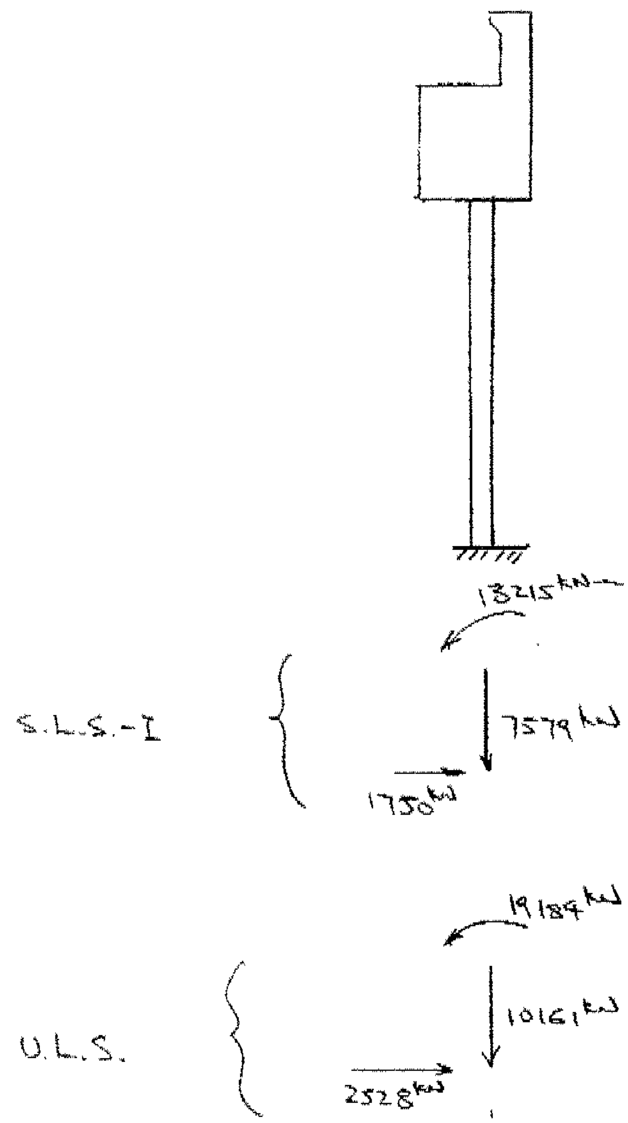
TO: David Kwok, P.Eng	**FAX NO:** 235-5240
COMPANY: M.T.O. Foundation Design Section	**DATE:** Aug 5, 1993
FROM: Peter Tam, P.Eng.	**TOTAL PAGES** (including this sheet): 6
DILLON PROJECT NO: 92-3678-01-00	If you do not receive all pages of this fax, please call:
SUBJECT: Foundation Design Information — Munroe Island Bridge	

As we discussed in our meeting on last Friday, two alternatives are being considered for the Munroe Island Bridge. The attached show the <u>total</u> reactions at South Abutment and North Abutment respectively for two alternatives. Tentatively, we anticipate 5 – 1267 mm ϕ caissons for Alternative #1 and 5 – 1067 mm ϕ caissons — 3 comp. at 1:8 batter in the front and 2 at 1:10 batter at the back for Alternative #2.

tension For Alternative #1, we need (1) the required embedment length of the caisson to develop the fixity to resist the applied moment, (2) allowable shaft friction for vertical compression load and (3) allowable bearing pressure for horizontal load.

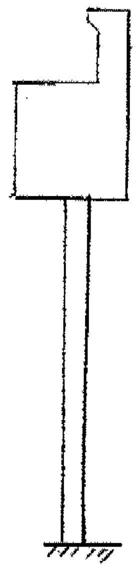
M.M. DILLON LIMITED: 100 Sheppard Avenue East, 3rd Floor, Toronto, Ontario M2N 6N5
MAIL: Box 1850, Station A, North York, Ontario M2N 6K7
TEL: (416) 229-4646
FAX: (416) 229-4692

Option 1 : Straight Caisson W/ Fixed End at Bottom

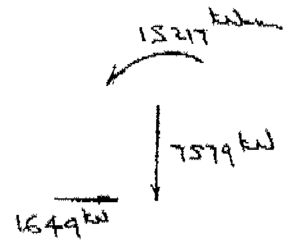


Total Reactions at North Abutment

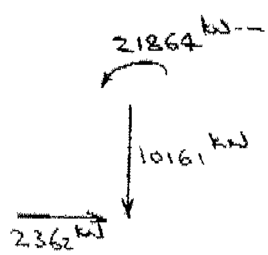
Option 1 : Straight Caissons W/ Fixed End at Bottom



S.L.S.-I



U.L.S.



Total Reactions at South Abutment

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FAX TRANSMITTAL

TO:	**FAX NO:**
COMPANY:	**DATE:**
FROM:	**TOTAL PAGES** (including this sheet):
DILLON PROJECT NO:	If you do not receive all pages of this fax, please call:
SUBJECT:	

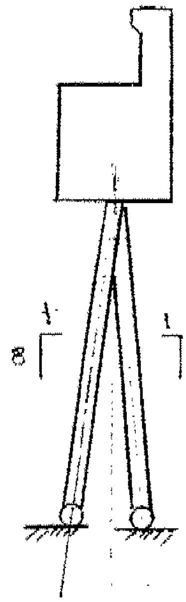
Likewise, for Alternative #2, we need (1) allowable shaft friction for vertical compression load, (2) allowable shaft friction for vertical uplift load, (3) allowable bearing pressure for horizontal load and (4) allowable bonding stress for rock anchor if rock anchor is used to resist uplift force.

Should you have any questions, please call.

Peter Tam

M.M. DILLON LIMITED: 100 Sheppard Avenue East, 3rd Floor, Toronto, Ontario M2N 6N5
MAIL: Box 1850, Station A, North York, Ontario M2N 6K7
TEL: (416) 229-4646
FAX: (416) 229-4692

Option 2 : Batter Caissons w/ Hinged Support at Bottom



S.L.S.-I

1324 kN → 325 kN →
11929 kN ↓ 4350 kN (uplift) ↑

U.L.S. with maximum compression reaction

1857 kN → 509 kN →
16816 kN ↓ 6656 kN (uplift) ↑

U.L.S. with maximum uplift reaction

1655 kN → 645 kN →
15101 kN ↓ 7989 kN (uplift) ↑

Total Reactions at South Abutment

By P. Tam Date Aug 5, 1993 Project Name Munroe Island Bridge

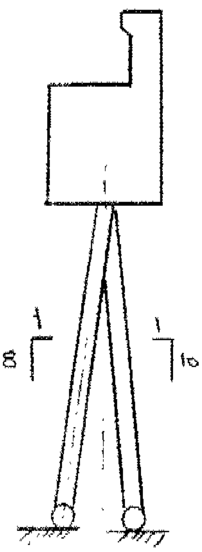
Checked ______ Date ______

Page ______ Of ______ Project No 92-3678-01-00

DILLON

121-90

Option 2 - Batter Caissons w/ Hinged Support at Bottom



U.L.S.-1
1374 kN
376 kN
12919 kN
5343 kN (uplift)

U.L.S. with maximum compression reaction
1441 kN
587 kN
15380 kN
8219 kN

U.L.S. with maximum uplift reaction
1696 kN
690 kN
16127 kN
9014 kN

Total Reactions at North Abutment

P. Tam
Aug 5, 1993
Munroe Island Bridge
92-3678-01-00

(RETAIN THIS COPY FOR FOLLOW-UP)

7540-1021

SEND TO Head, Structural Section
Northwestern Region
Attn: D. Dystra

FROM David Kwok **DEPT.** Foundation Design Section **DATE** 93 06 22

SUBJECT W.P. 29-88-01, Munroe Island Bridge

Re your discussion with Balu Iyer on 93 06 21. Our major concern is on the construction of abutment slopes. We consider it very difficult in practice to construct the slopes under water to the specified geometry. Our report has indicated an alternative of closed abutments supported by reinforced earth walls. ~~Ha~~ Has this alternative been seriously looked at? There are also concerns on the feasibility of constructing batter caissons through existing rock fill.

REPLY

As suggested in the foundation report and again in our memo dated 93 06 18, a meeting with your office and consultant is warranted to clarify the above items and any other items the consultant may have.

REPLY FROM

REPLY DATE

memorandum

Ontario

To:	R.J. Krisciunas Head, Structural Section Thunder Bay	Date:	93 06 18
Attn:	D. Dystra		
From:	Foundation Design Section Room 315, Central Building Downsview, Ontario		
Subject:	Preliminary Drawing Review Munroe Island Bridge Highway 548 W.P. 29-88-01, Site 38S-177 District 18, Sault Ste. Marie		

We refer to your memorandum dated 93 06 03 and the preliminary General Arrangement drawing P1 attached therein.

In general, the drawing is found to be in conformance with the previous foundation recommendations. However, there are concerns in the following specific areas :

- construction of abutment slopes.

- construction of batter caissons.

As discussed with you, we suggest that a meeting be held to review the above items with representatives from your section, the design consultant and our office. Would you please arrange for it and let us know.

David Kwok, P. Eng.
Project Foundation Engineer
for
Balu Iyer, P. Eng.
Senior Foundation Engineer

MEMORANDUM

Ontario

To: R.J. Krisciunas
Head, Structural Section
Thunder Bay

Date: December 30, 1992

Attn: D. Dykstra

From: Foundation Design Section
Room 315, Central Building
Downsview, Ontario

Subject: Munroe Island Bridge
Highway 548
W.P. 29-88-01, Site 38S-177
District 18, Sault Ste. Marie

The field investigation for the above-noted project has been completed. This memorandum provides a summary of the subsurface conditions encountered at the site and engineering recommendations intended for design to proceed. The final foundation report will follow.

The site is located on Highway 548 at the southmost crossing from St. Joseph Island to the mainland, in the Township of St. Joseph, District of Algoma. It is about 3.4 km south of the intersection of Highway 17 and Highway 548. The existing structure is in poor condition with a history of distress. It is proposed to replace it with a new single span bridge.

The field work was conducted between 92 11 03 and 92 11 09 and consisted of six (6) sampled boreholes taken down to 3.1 to 16.3 m depth. One copy each of the borehole location plan and preliminary record of borehole sheets is attached. BH 1 and BH 2 were drilled by a track mounted continuous flight auger machine. BH 3 to BH 6 were advanced from a raft over water using a diamond drill.

In BH 1 to BH 4, which are on or close to the existing embankment, the subsurface material encountered typically comprised of fill over bedrock. A thin layer (1.3 m) of native material consisting of silty sand, some gravel, with boulders was found in BH 4 overlying bedrock. In BH 5 and BH 6 which are further from the shore, the bedrock is overlain by a thin layer of river bed material. The material is a 0.5 m layer of sand and gravel, some boulders in BH 5 and a 0.8 m layer of organic silt in BH 6. The groundwater level measured in the land holes ties in well with the water surface in the river. During the time of the investigation, the water level was at EL. 176.4 m. Laboratory tests are being carried out on the soil samples and the results will be included in the final report.

Two types of bedrock were encountered at this site. At the north abutment location, the bedrock is a Gabbro of igneous origin. The rock is considered strong with core recoveries range from 80 to 100 percent. At the south abutment location, the bedrock is a Quartz Sandstone of sedimentary origin. Rock cores from BH 2 have core recoveries of 65 to 96 percent, and the rock is generally strong. At BH 4 the rock is kaolinitic and shattered with poor core recoveries near the top 1.3$\pm$ m, ranging from 30 to 58 percent. The rock at this location is considered to be medium strong.

The project comprises construction of a replacement bridge to the west of the existing one. The clear distance between the two bridges is about 3 m at the south end and 7 m at the north. The elevation of the new bridge is similar to the existing one, at El. 182 $\pm$ m. The existing bridge will serve as detour during construction, and will be removed after construction of the new bridge.

The following are the engineering recommendations pertaining to the design and construction of the structure.

Foundation

According to the investigation results, the subsurface stratigraphy at abutment locations typically comprises fill over bedrock. Since the existing bridge will remain in service during construction, excavation of existing fill has to be kept a minimum to avoid undermining the existing embankment.

The structure can be supported by caissons socketed into bedrock. Liners should be used during excavation to stabilize the hole and surrounding ground. Boulder obstructions are expected when the excavation advances through the existing rock armouring layer.

Caissons should be socketed at least 300 mm into bedrock. The recommended minimum diameter for the caissons is 910 mm to allow for downhole inspection and hand-cleaning. Before concrete is poured, the loose material at the bottom of the caisson hole should be hand-cleaned or air-lifted. Tremie concrete is recommended for under water concreting. The following factored end bearing values are recommended for the purpose of O.H.B.D.C. Depending upon the final design caisson sizes and socket lengths, we are pleased to recommend upon contributions from shaft friction.

	Factored Capacity at U.L.S.
North Abutment (Gabbro)	7500 kPa
South Abutment (Kaolinitic Quartz Sandstone)	3000 kPa

S.L.S. type II does not govern for unyielding founding material.

The caissons should be founded into sound bedrock. For preliminary design purpose, the estimated caisson bottom elevations are as follows:

	Estimated Caisson Bottom Elevation (M)
North Abutment (West Side)	170.0
North Abutment (East Side)	171.5
South Abutment (West Side)	167.5
South Abutment (East Side)	171.0

Alternatively, driven piles may also be considered. For steel H-piles dr bedrock, the following design capacities in accordance with the O.H.B.D recommended.

	HP310 X 79	HP310
Factored Axial Capacity at U.L.S.	1150 kN	160
Axial Capacity at S.L.S. Type II	890 kN	115

Again, it is anticipated that the piles would encounter boulder obstructio the existing rock armouring layer. The obstructions at pile locations wi to be removed by pre-augering or other means and replaced with granular materials.

The new fill material at pile locations should have a maximum grain size limited to 75 mm. Piles should be equipped with driving shoes to minimize tip damage. Lateral loads may be taken by the horizontal component of batter piles.

For preliminary design purpose, the estimated pile tip elevations are as follows:

	Estimated Pile Tip Elevation (M)
North Abutment (West Side)	170.5
North Abutment (East Side)	172.5
South Abutment (West Side)	170.0
South Abutment (East Side)	171.5

Due to the anticipated obstructions, pile driving should be closely monitored to ensure that the piles are founded on bedrock. If this alternative is chosen, please contact our office for further recommendations on pile driving.

Alternatively, consideration may also be given to shallow foundation. However, since the new alignment is close to the existing, there are limitations regarding excavation on existing embankment in order not to undermine it. Accordingly, the footings will have to be supported on a granular pad perched over existing fill. The existing fill is poorly graded with a rock armouring layer along the shore. It is imperative that all voids be chinked with well graded granular material or mass concrete. There are concerns on the integrity of the existing fill and river erosion, in view of the history of distress of the existing bridge. If this alternative is adopted, please contact our office for details.

Earth Pressure

Backfill to abutments should consist of granular material in accordance with MTO Standard Special Provision No. 121 (83 10).

Computation of earth pressures should be in accordance with Section 6.6.1.2.1. of the O.H.B.D.C. The active condition will govern earth pressure design for the yielding case while the at-rest condition will govern for the unyielding case. The follow properties for backfill are recommended.

Material	ϕ	γ	Ka	Ko
Granular "A"	35°	22.8 kN/m^3	0.27	0.43
Granular "B"	30°	21.2 kN/m^3	0.33	0.50
Rockfill	35°	18.0 kN/m^3	0.27	0.43

Slope Stability

The Munroe Island Channel has a V shaped river bed with fairly steep sub-aqueous slopes. For preliminary design purpose, the following slope geometries may be used. Analyses are being carried out and these recommendations will be confirmed in the final report. Rockfill slopes may be formed at 1.25H:1V up to 6 m and with a 2 m wide berm every 6 m up to a maximum height of 14 m so that no uninterrupted slope is greater that 6 m. Alternatively, rockfill slopes may be formed to 1.5H:1V gradients up to 10 m and with a 2 m wide berm above 10 m up to a maximum height of 14 m. If earth fill is used, the slopes should be formed at 2H:1V gradient with a 2 m wide berm incorporated at 9 m for embankment heights above 9 m to a maximum height of 14 m.

Only relatively free draining granular material should be used underwater. Substantial filling will be required for the abutment forward slopes due to the relatively steep existing river bank gradient. If a closed abutment is used, consideration should be given to reinforced earth abutment construction. This support system enables the vertical loads be taken by the caissons/piles and the lateral loads by the reinforced earth wall. If this type of construction is adopted, please contact our office for further details.

Based on the investigation results, some organic material is found at the river bed. It is recommended that the organic material be removed prior to filling.

Provisions should be made in the contract document to take into account possible obstructions during pile driving or caisson construction.

We believe that the above is sufficient for the present purpose, should you have any questions or require further information, please call us.

D. Kwok, P. Eng.
Project Foundation Engineer

For

B. Iyer, P. Eng.
Senior Foundation Engineer

BI/DK/hh

memorandum

Ontario

To: R.J. Krisciunas
Head, Structural Section
Northwestern Region

Date: 1992 07 03

Attn: D. Dykstra

From: Foundation Design Section
Room 315, Central Bldg.
Downsview

Re: Munroe Island Bridge
Hwy. 548
W.P. 29-88-00
District 18, Sault Ste. Marie

We have now completed our review of the data provided to us on the above project.

1. From consideration of the steep sub-aqueous slope to the east of the existing bridge alignment, it is considered that a single span bridge, temporary detour or a permanent replacement, together with rockfill approaches at the south end and north end along the east will not be possible.

   If a detour or permanent bridge is to be constructed along the eastern alignment, consideration should be given to a three span bridge, with the piers supported on caissons socketed into the sloping bedrock. This would be somewhat difficult to construct and would be expensive compared to other options discussed below.

2. From consideration of the relatively gentle subaqueous slope to the west of the existing bridge alignment, it is considered that the western alignment would be suitable for the construction of a single span detour bridge or a single span permanent replacement bridge, together with approach fills at both ends.

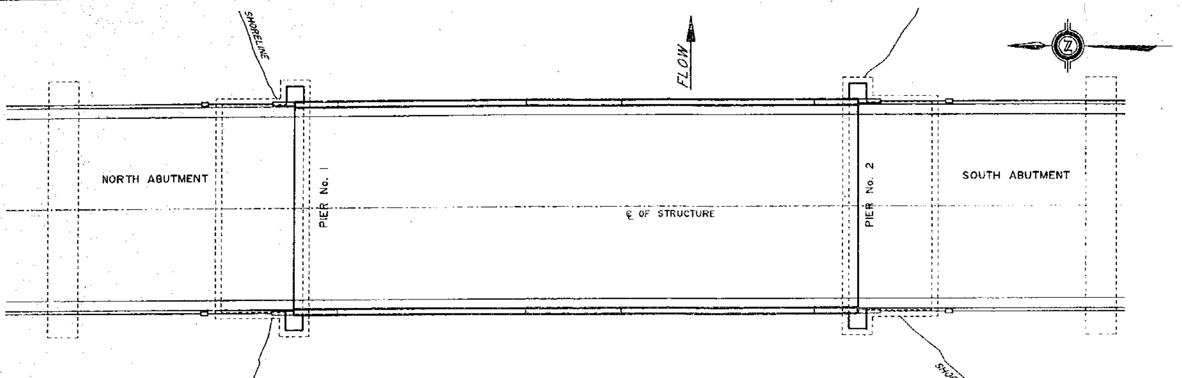
   From consideration of existing trees at the southern limits of the western alignment, it may be preferable to leave the permanent alignment where the present bridge is located and construct a detour bridge along a slightly modified alignment along the west, which will have minimal environmental impact.

The above comments are provided from a geotechnical standpoint. We understand that the final choice of detour and permanent alignment would be based also on other considerations. Once the alignments are finalized, please forward detailed E-plans to our office and we would then review the need for a foundation investigation at this site.

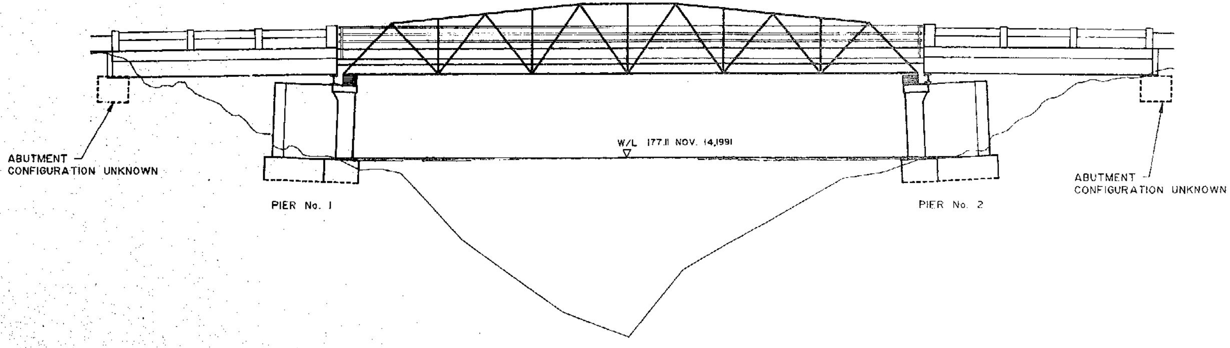
We trust that the information given above is sufficient for your present needs.

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/jb



BRIDGE PLAN



BRIDGE ELEVATION

MINISTRY OF TRANSPORTATION
NORTHWESTERN REGION

UNDERWATER INSPECTION
SITE No. 38S-177
ST. JOSEPH'S ISLAND -
MONROE ISLAND BRIDGE
HIGHWAY 548, 9km EAST OF RICHARDS LANDING

GENERAL ARRANGEMENT

NOTES:

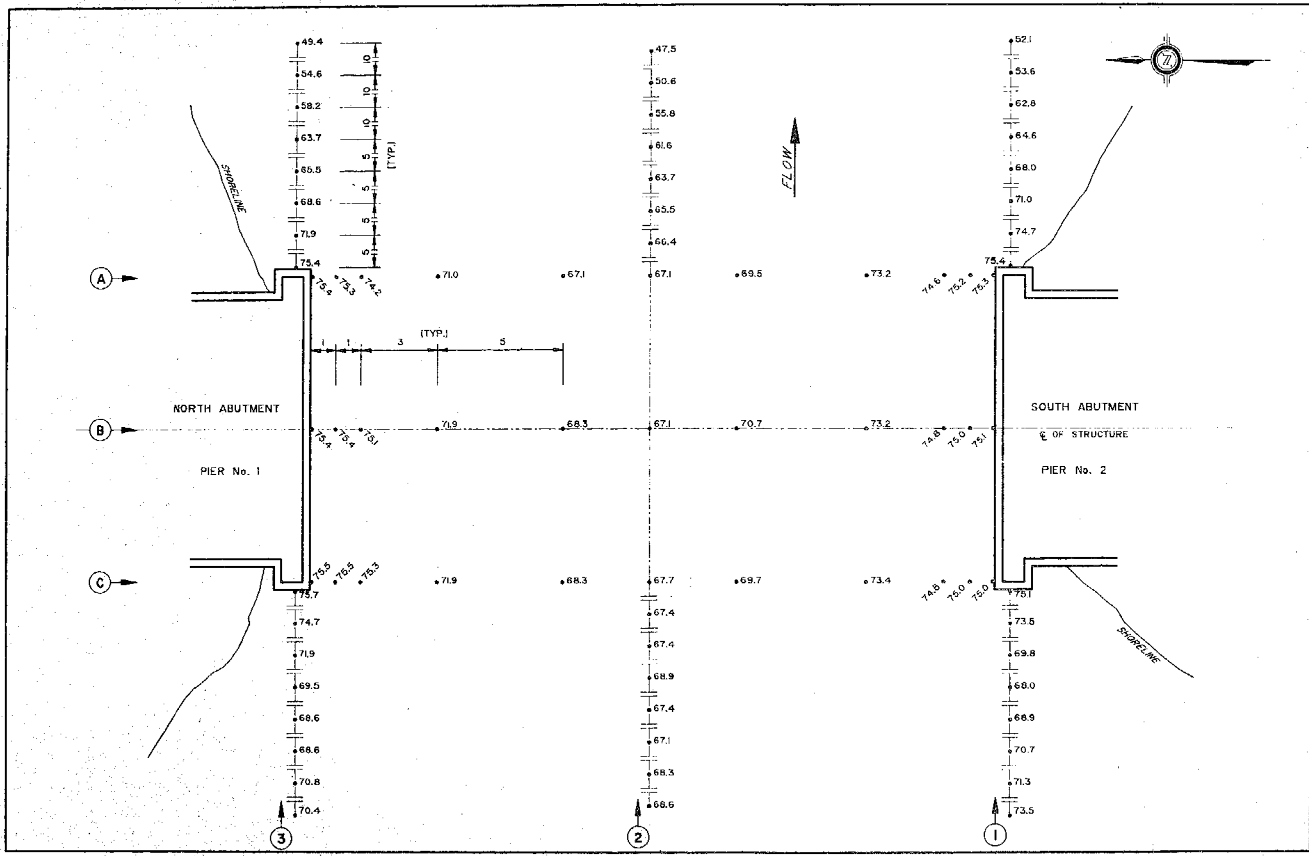
1. REPRODUCED FROM NOV. 1991 PHOTOGRAPHS AND DWG. No. D3087-1
2. WATER LEVEL ELEVATION 175.96m (NOVEMBER 14, 1991)
3. REFERENCE ELEVATION - TOP OF CONCRETE DECK, CENTER LINE OF HIGHWAY - EL. 182.93m

Figure 1

SCALE: N.T.S.

DATE : DEC. 1991 | DRAWN: R. TURGEON

Proctor & Redfern Limited
Consulting Engineers & Planners
London EO: 91964



MINISTRY OF TRANSPORTATION
NORTHWESTERN REGION

UNDERWATER INSPECTION
SITE No. 38S-177
ST. JOSEPH'S ISLAND -
MONROE ISLAND BRIDGE
HIGHWAY 548, 9km EAST OF RICHARDS LANDING

WATER DEPTH SOUNDINGS

NOTES:
1. ADD 100m TO ALL ELEVATIONS SHOWN
2. WATER LEVEL ELEVATION 175.96m (NOVEMBER 14, 1991)

Figure 2

SCALE: 1:125

DATE : DEC. 1991 | DRAWN: T. HALL

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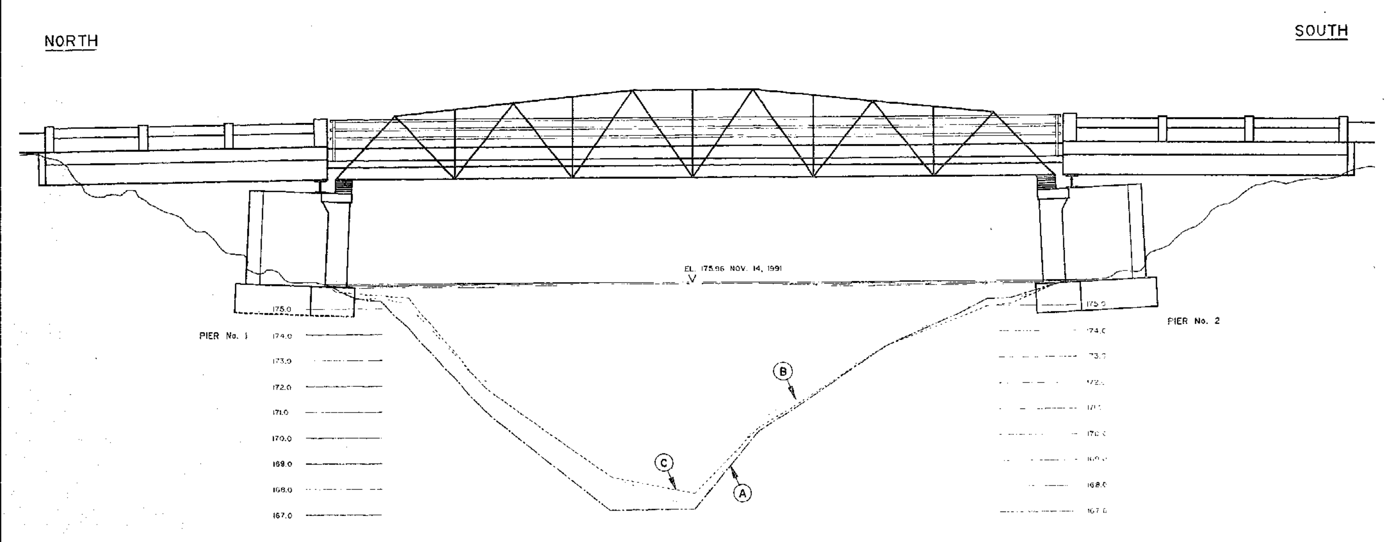
London EO: 91964



MINISTRY OF TRANSPORTATION
NORTHWESTERN REGION

UNDERWATER INSPECTION
SITE No. 38S-177
ST. JOSEPH'S ISLAND -
MONROE ISLAND BRIDGE
HIGHWAY 548, 9km EAST OF RICHARDS LANDING

RIVER BED CROSS SECTIONS

NOTES:

1. FOR GRID LINE LOCATION SEE FIG. 2
2. WATER LEVEL ELEVATION 175.96m (NOVEMBER 14, 1991)

LEGEND:

GRID LINE (A) DOWNSTREAM SIDE OF BRIDGE
GRID LINE (B) CENTRELINE OF BRIDGE
GRID LINE (C) UPSTREAM SIDE OF BRIDGE

Figure 3

SCALE: 1:100

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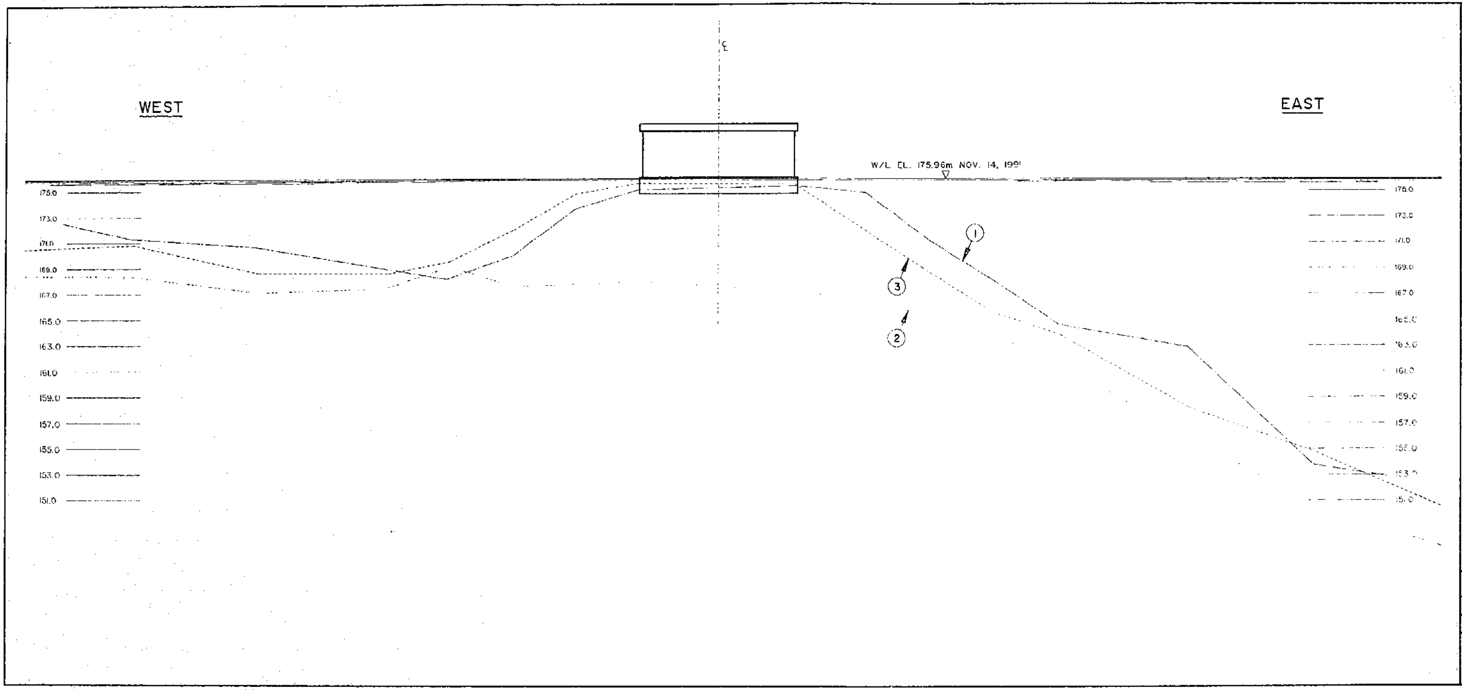

MINISTRY OF TRANSPORTATION
NORTHWESTERN REGION

UNDERWATER INSPECTION
SITE No. 38S-177
ST. JOSEPH'S ISLAND -
MONROE ISLAND BRIDGE
HIGHWAY 548, 9km EAST OF RICHARDS LANDING

RIVER BED PROFILES

NOTES:

1. FOR GRID LINE LOCATION SEE FIG. 2
2. WATER LEVEL ELEVATION 175.96m (NOVEMBER 14, 1991)

LEGEND:

GRID LINE (1) SOUTH ABUTMENT
GRID LINE (2) CENTRELINE OF BRIDGE
GRID LINE (3) NORTH ABUTMENT

Figure 4

SCALE: 1:200 0 2 4 6 8 10 12

DATE : DEC. 1991 | DRAWN: T. HALL

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