DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 31E-17

DIST. 11 REGION

W.P. No. 401-64-02

CONT. No.

W. O. No.

STR. SITE No.

HWY. No. 118

LOCATION Port Carling

No of Pages -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

143

Ministry of Transportation and Communications

Ontario

Memorandum

To: Mr. J. C. McAllister
Head,Structural Section
Northern Region

From: Geotechnical Section
Northern Region

Attention:

Date: 1978 09 01

Our File Ref.

In Reply to

Subject: RETAINING WALL AT PORT CARLING
WP 401-64-02, HWY 118
DISTRICT # 11 - HUNTSVILLE

In his memorandum of 78 08 29 to you, Mr. Payer refers the question of frost heaving and internal seepage to the Geotechnical Section.

It is the opinion of the Geotechnical Section that due to the granular nature of the sub soils that differential frost heaving should not be a problem.

During the course of the investigation no substantial zones of wet material were encountered and this Section does not foresee a problem with internal seepage.

M. J. Ramsden
Project Soils Supervisor
for
S. G. Wilson
Soils Engineer

MJR/SGW/ap

cc: S. McCombie Att'n: A. Parnemagi
K. Selby Att'n: P. Payer

RECEIVED
SEP 06 1978
MINISTRY OF TRANSPORTATION & COMMUNICATIONS
SOIL MECHANICS

Mr. J.C. McAllister
Head, Structural Section
Northern Region, North Bay

Soil Mechanics Section
Engineering Materials Office
Room 315, Central Building

78 08 29

Re: Retaining Wall at Port Carling
W.P. 401-64-02, Hwy. 118
District 11, Huntsville

We have reviewed the design proposals contained in your memorandum of 78 07 18, together with the attached subsurface information obtained by the Regional Geotechnical Section.

The proposal calls for the use of grouted, cobble size stones on a slope between Sta. 333+00± and Sta. 337+50±. The vertical height of the slope varies from 2 feet to 6 feet and steepness from 1:1 to 4:1.

The borings have indicated that the relatively shallow (2 to 12 feet) overburden material consists of dense to very dense silty sand with traces of gravel. Frequent boulders and cobbles were also found within this zone. The granular deposit is underlain by bedrock.

Three problems are usually considered in the design of cut-slopes constructed within non-cohesive deposits: surface erosion, internal seepage and frost heave effects.

At this location, it is our opinion that placing grouted cobble size stones on the slope will provide adequate protection against surface erosion.

As reported, no groundwater was encountered during the field investigation (1978 06 19-21) and the sampled material appeared to be in the dry state. From this information we have concluded that internal seepage and frost heave would not present problems concerning the performance of such slopes. However, the presence of internal seepage and the possibility of frost action (heave) should be further ascertained by the Regional Geotechnical Section.

P. Payer
Senior Engineer

For: K.G. Selby
Supervising Engineer

PP/KGS/gs

cc: W.W. Peck
Files

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 1

WP 401-64-02

LOCATION 337 + 75 26' Left Centreline Hwy. 118

ORIGINATED BY H.T.

DIST 11 HWY 118

BORING DATE June 21, 1978

COMPILED BY

DATUM Geodetic

BOREHOLE TYPE Hand Auger to Refusal

CHECKED BY S.W.

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER ELEV	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W; WATER CONTENT %	BULK DENSITY γ	REMARKS GR. SA. SI. CL.
780.3	Ground Level									
0.0	Top Soil 3"									
778.8	Sandy Loam Stoney									
1.5	End of Borehole Refusal Probable Bouldery Material									
						770				
						760				

E REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 2

WP 401-64-02 LOCATION 337 + 71 29' Left Centreline Hwy. 118 ORIGINATED BY H.T.

DIST 11 HWY 118 BORING DATE June 21, 1978 COMPILED BY

DATUM Geodetic BOREHOLE TYPE Hand Auger CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W; WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL.
780.0	Ground Level									
0.0	Sandy Loam Topsoil 4"									
777.5 2.5	Fine Sandy Loam-Stoney End of Borehole Refusal - Probable Bedrock									
						770				
						760				

REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 3

WP 401-64-02	LOCATION 337 + 60 29' Left Centerline		ORIGINATED BY H.T.
DIST 11	HWY 118	BORING DATE June 21, 1978	COMPILED BY
DATUM Geodetic	BOREHOLE TYPE Hand Auger to Refusal		CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL.
780.0	Ground Level									
0.0	Sandy Loam Topsoil									
777.5	Fine Sandy Loam-Stoney									
2.5	End of Borehole Refusal - Probable Bedrock									
						770				
						760				

E REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE No 4

WP 401-64-02 LOCATION 337 + 55 29' Left Centreline Hwy. 118 ORIGINATED BY H.T.

DIST 11 HWY 118 BORING DATE June 21, 1978 COMPILED BY

DATUM Geodetic BOREHOLE TYPE Hand Auger to Refusal CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W; WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL.
779.8	Ground Level									
0.0	Sandy Loam Topsoil									
777.5	Fine Sandy Loam Stoney									
2.3	End of Borehole Refusal - Probable Bedrock									
						770				
						760				

E REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 5

WP 401-64-02

DIST 11 HWY 118

DATUM Geodetic

LOCATION 337 + 50 24.5' Left Centreline Hwy. 118

BORING DATE June 20, 1978

BOREHOLE TYPE Hollow Stem Auger to Refusal

ORIGINATED BY H.T.

COMPILED BY

CHECKED BY S.W.

SOIL PROFILE: ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT %	BULK DENSITY γ	REMARKS GR.SA.SI.CL.
779.7	Ground Level									
0.0	Sandy Loam Topsoil 10"									
			1053	SS	135					14,84,22,0
774.7 / 5.0	Gravel									
			82	SS	37					2,82,16,0
770.7 / 9.0	Fine Sandy Loam End of Borehole Refusal Probable Bedrock					770				
						760				

E REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 6

WP 401-64-02
LOCATION 337 + 50 28.5' Left Centerline Hwy. 118
ORIGINATED BY H.T.

DIST 11 HWY 118
BORING DATE June 19, 1978
COMPILED BY

DATUM Geodetic
BOREHOLE TYPE Hollow Stem Auger & BXL Rock Core
CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (20 40 60 80 100); SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; W_P W W_L; WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL
779.8	Ground Level									
0.0	Sandy Loam Topsoil 6"									
			79	SS	105					12,66,22,0
			80	SS	128/5"					5,67,28,0
			81	SS	100/6"	770				19,74,7,0
767.5	Sandy Loam - Bouldery									
12.3	Bedrock Recovery Good									
762.5										
17.3	End of Borehole									
						760				

REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 7

WP 401-64-02

DIST 11 HWY 118

DATUM Geodetic

LOCATION 337 + 50 32.5' Left Centreline Hwy. 118

BORING DATE June 20, 1978

BOREHOLE TYPE Hollow Stem Auger

ORIGINATED BY H.T.

COMPILED BY

CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W / W_P W W_L / WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL.
780.0	Ground Level									
0.0	Sandy Loam Topsoil 4"									
	Fine Sandy Loam Stoney		83	SS	102/3"					6,48,34,12
776.0 4										
			1054	SS	99					12,81,7,0
771.0 9	Gravel									
						770				10,67,23,0
768.5	Fine Sandy Loam Stoney		84	SS	43					
11.5	End of Borehole									
						760				

E REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 8

WP 401-64-02 LOCATION 337 + 40 29' Left Centerline Hwy. 118 ORIGINATED BY H.T.

DIST 11 HWY 118 BORING DATE June 21, 1978 COMPILED BY

DATUM Geodetic BOREHOLE TYPE Hand Auger CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; W_P W W_L; WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL.
778.1	Ground Level									
0.0	Sandy Loam Topsoil									
	Fine Sandy Loam Stoney									
775.6 2.5										
774.1 4.0	Very Fine Sandy Loam End of Borehole									
						770				
						760				

REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 9

WP 401-64-02

LOCATION 336 + 65 11' Left Centreline Hwy. 118

ORIGINATED BY ~~H.T.~~

DIST 11 HWY 118

BORING DATE June 20, 1978

COMPILED BY

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY S.W.

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w; WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH 764.4	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV				GR.SA.SI.CL.
0.0	Asph 2" Gravel 2"									
			85	Sample from Auger		760				6,56,37,0
759.4 5.0	Fine Sandy Loam Stoney End of Borehole Refusal - Probable Bedrock									
						750				

REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 10

WP 401-64-02

DIST 11 HWY 118

DATUM Geodetic

LOCATION 336 + 65 22' Left Centreline Hwy. 118

BORING DATE June 20, 1978

BOREHOLE TYPE Hollow Stem Auger

ORIGINATED BY H.T.

COMPILED BY

CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W; W_P W W_L; WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL.
769.4	Ground Level									
0.0										
			86	SS	125/5"					6,71,23,0
			87	SS	155/7"					
762.9 / 6.5	Sandy Loam Stoney / End of Borehole									
						760				

REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No 11

FOUNDATIONS OFFICE

WP 401-64-02

LOCATION 335 + 50 12' Left Centreline Hwy. 118

ORIGINATED BY H.T.

DIST 11 HWY 118

BORING DATE June 20, 1978

COMPILED BY

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY S.W.

SOIL PROFILE: ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w; WATER CONTENT %	BULK DENSITY γ	REMARKS GR.SA.SI.CL.
752.5	Ground Level									
0.0	Asph 2"									
751.25	Crushed Gravel 6" Fine Sand									
1.25	Mulch 2"					750				
			88	SS	6					4, 50,26,20
			89	SS	27					10,69,21,0
746.0	Sandy Loam Stoney									
6.5	End of Borehole									
						740				

REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS – ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE No 12

WP 401-64-02

LOCATION 335 + 50 22' Left Centreline Hwy. 118

ORIGINATED BY H.T.

DIST 11 HWY 118

BORING DATE June 21, 1978

COMPILED BY

DATUM Geodetic

BOREHOLE TYPE Hand Auger to Refusal

CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W; W_P W W_L; WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL.
752.5	Ground Level									
0.0	Sandy Loam Stoney									
750.5 2.0	End of Borehole Refusal - Probable Bouldery Material									

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 13

WP 401-64-02

DIST 11 HWY 118

DATUM Geodetic

LOCATION 334 + 65 27' Left Centreline Hwy. 118

BORING DATE June 21, 1978

BOREHOLE TYPE Hand Auger to Refusal

ORIGINATED BY H.T.

COMPILED BY

CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w / WATER CONTENT %	BULK DENSITY γ	REMARKS GR.SA.SI.CL.
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				
749.5	Ground Level									
0.0	Fine Sandy Loam Topsoil									
1011	Fine Sandy Loam									
747.0 / 2.5	Clay Loam Stiff									
745.0 / 4.5	End of Borehole Refusal - Probable Bouldery Material									
						740				

E REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 14

WP 401-64-02 LOCATION 334 + 50 11' Left Centreline Hwy. 118 ORIGINATED BY H.T.

DIST 11 HWY 118 BORING DATE June 21, 1978 COMPILED BY

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W; W_P W W_L; WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL.
749.5	Ground Level									
	Asph 3"									
	Crushed Gravel 7"									
	Sandy Loam Stoney		90 AUGER SAMPLE							8,65,17,10
744.0	Clay Loam Stiff		91	SS	25					3,36,19,32
5.5	End of Borehole									
						740				

E REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 15

WP 401-64-02 LOCATION 333 + 50 14' Left Centreline Hwy. 118 ORIGINATED BY H.T.

DIST 11 HWY 118 BORING DATE June 21, 1978 COMPILED BY

DATUM Geodetic BOREHOLE TYPE Hand Auger to Refusal CHECKED BY S.W.

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w / w_P w w_L / WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				GR.SA.SI.CL.
749.0	Ground Level									
0.0	Sandy Loam Stoney									
748.0 1.0										
746.5	Course Sand									
2.5	End of Borehole Refusal - Probable Bouldery Material									
						740				

E REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 16

WP 401-64-02
DIST 11 HWY 118
DATUM Geodetic

LOCATION 333 + 50 22' Left Centreline Hwy. 118
BORING DATE June 21, 1978
BOREHOLE TYPE Hand Auger to Refusal

ORIGINATED BY H.T.
COMPILED BY
CHECKED BY S.W.

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER ELEV	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT %	BULK DENSITY γ	REMARKS GR.SA.SI.CL.
749.0	Ground Level									
0.0	Sandy Loam Stoney									
747.0 2.0	End of Borehole Refusal - Probable Bouldery Material									
						740				

REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

Memorandum

To: Mr. J. McAllister,
Head, Structural Office,
Northern Region.

From: Geotechnical Section,
Northern Region.

Attention:

Date: 1978 07 11

Our File Ref.

In Reply to

Subject:

W.P. 401-64-02, HIGHWAY 118
PORT CARLING - RETAINING WALL
DISTRICT #11, HUNTSVILLE

We have completed our field investigation for the proposed retaining wall in Port Carling. Enclosed please find the logs of our boreholes, sample sheets, cross-sections and a plan showing the test hole locations.

Our field work was done in the textural classification system and the percentages of gravel, sand, silt, and clay are shown in accordance with this system. Elevations were determined from cross-section. During the investigation no free water was encountered and the soils were described as being only moist.

Although the information as presented is not in accordance with the present practise of the Soil Mechanics Section I trust they will find it adequate.

If the information contained meets with your approval please forward to Mr. K. Selby, Soil Mechanics Section. If you or Mr. K. Selby have any questions please feel free to contact us.

S.G. Wilson

S.G. WILSON,
SOILS ENGINEER.

SGW:tp
Encl.
c.c. S. McCombie

RECEIVED
JUL 19 1978
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

Ministry of
Transportation and
Communications

Ontario

Memorandum

To: Mr. C. Mirza
Manager, Soil Mechanics Section
West Building, Downsview

From: Engineering & Right-of-Way Office
Structural Section
Northern Region

Attention: K. Selby

Date: 78 07 18

Our File Ref.

In Reply to

Subject: W.P. 401-64-02
Retaining Wall at Port Carling
Highway #118, District #11

I discussed Planning and Design's proposal, at this location, with you some time ago. Regional Soils have carried out the field work, the results of which I am forwarding to you.

Planning and Design propose to use grouted, cobble size stone on a maximum 1:1 slope and contained between a type 'B' curb and a concrete sidewalk.

Plan of the location and cross-sections are also enclosed.

Please review this proposal and let me have your comments as soon as possible.

J. C. McAllister

J. C. McALLISTER
HEAD, STRUCTURAL SECTION

JCMcA/tt
Encl.

cc: A. Parnamagi

RECEIVED
JUL 19 1978
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

Ken Selby

Department of Transportation and Communications

MEMORANDUM

TO: Mr. J. McAllister, (2)
Regional Bridge Planning Supervisor,
Northern Region,
North Bay, Ontario.

FROM: Foundations Office,
Design Services Branch,
Central Bldg., Downsview.

ATTENTION:

DATE: November 22, 1971.

OUR FILE REF.

IN REPLY TO

NOV 25 1971

SUBJECT:

31E-17

FOUNDATION INVESTIGATION REPORT
For
Proposed New Structure and
Related Retaining Walls at the
Crossing of Hwy. #118 (Line 'E' and
Indian River
Village of Port Carling - Dist. of Muskoka
District No. 11 (Huntsville)
W.O. 71-11074 -- W.P. 401-64

Cont 73-125

Attached, we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above structure site.

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER.

AGS/ao
Attach.
cc: Messrs. D. W. Farren
B. R. Davis
A. Rutka
H. McArthur
R. S. Chapman
B. J. Giroux
R. Northwood
G. A. Wrong
B. A. Singh
Foundations Office
Documents

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FOUNDATION INVESTIGATION REPORT
For
Proposed New Structure and
Related Retaining Walls at the
Crossing of Hwy. #118 (Line 'E') and
Indian River
Village of Port Carling - Dist. of Muskoka
District No. 11 (Huntsville)
W.O. 71-11074 -- W.P. 401-64

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1. INTRODUCTION:

The Foundation Office was requested to carry out a subsurface investigation for a proposed structure and related retaining walls to replace the existing ones at the aforementioned crossing, in the Village of Port Carling, District of Muskoka. In addition, an investigation was carried along the alignment of the proposed detour. The request was contained in a memo from Mr. J. C. McAllister, Regional Bridge Planning Supervisor, Northern Region, dated July 15, 1971. An investigation was subsequently carried out by this Office at the proposed site. During this investigation the subsoil, bedrock and groundwater conditions in this area were determined.

This report contains the factual results obtained from the investigation, together with the recommendations pertaining to the foundations of the proposed permanent structure as well as temporary detour structure and retaining walls.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The area investigated is located in a commercially developed part of the Village of Port Carling, specifically at the point where Hwy. #118 crosses the Indian River. The south to north-west flowing Indian River channel varies from 60 to 130 feet in width and 8 to 18 feet in depth, with the natural side slopes ranging from 2½:1 to 3:1. At the time of the investigation the water level in the river was approximately elevation 739; i.e., the water was between 4 to 11 feet in depth.

2. DESCRIPTION OF THE SITE AND GEOLOGY: (cont'd) ...

A lift lock system merges with the Indian River downstream of the existing structure. This lock is approximately 33 feet wide and 20 feet deep; the lock is confined by concrete retaining walls.

The surrounding terrain rises gently in all directions from the Indian River - lift lock complex, reaching a maximum elevation of 782 south and west of the river.

Hwy. #118, in this area, is a two lane paved roadway. The existing Hwy. #118 structure crossing the Indian River, however, has a single lane only. Pertinent details of the structure will be described in Section No. 6.

Physiographically the site is situated within the Precambrian Shield. The terrain has a bedrock oriented topography with numerous rock outcrops noticed throughout the region. In many areas the bedrock is covered by relatively thin deposits of sand and gravel laid down as outwash deposits during the various interglacial stages.

3. FIELD AND LABORATORY WORK:

Twenty-six sampled boreholes, 16 of which were accompanied by a dynamic cone penetration test, were put down at this site. The borings and cones were advanced using a diamond drill rig adapted for soil sampling purposes. Four borings were put down through the river bed of the Indian River; these were carried out by mounting the drilling equipment on a drum raft.

Samples of the fill and overburden were obtained at required intervals, in a 2-inch O.D. split-spoon sampler, which was hammered into the soil in accordance with the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration tests. The overburden is often bouldery in nature; diamond drilling techniques were periodically required to advance through the bouldery zones.

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3. <u>FIELD AND LABORATORY WORK</u>: (cont'd) ...

Bedrock was proven in 24 of the boreholes by obtaining between 4 and 16 feet of either BX or AXT size rock core samples.

Groundwater level observations were carried out, during the period of the investigation, in the open boreholes.

The soil, bedrock and groundwater conditions, encountered at those borings put down at the proposed site are presented on the Record of Borelog sheets appended to this report. The location and elevation of each of the boreholes were provided by personnel from the Regional Engineering Surveys Section (North Bay). The elevations in this report are referenced to a Geodetic datum.

The locations of the borings put down are shown in plan on Drawing No. 71-11074A. Stratigraphical profiles, along the proposed alignment of Hwy. #118 (Line 'E') as well as along the temporary detour, inferred from the boring data, are also presented on the aforementioned drawing. Additional inferred stratigraphical sections are shown on Drawing No. 71-11074B.

All the samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected representative samples to determine the following engineering properties of the overburden.

Natural Moisture Content
Atterberg Limits
Grain-Size Distribution

The results of the testing are plotted on the Record of Borelong sheets and summarized on Figures No. 1, 2 and 3 - all contained in the Appendix of this report.

4. <u>SUBSOIL AND BEDROCK CONDITIONS</u>:

4.1) <u>General</u>:

The predominant stratum across the site is a loose to very dense silty sand and gravel to sandy gravel, which varies from 1.5 to 40.5 feet in thickness. A 3.5 to 5.5 feet thick deposit of stiff to hard clayey silt is occasionally sandwiched within the granular stratum.

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4. SUBSOIL AND BEDROCK CONDITIONS: (cont'd) ...

4.1) General: (cont'd) ...

The overburden deposits are periodically overlain by fill which ranges from 5 to 6.5 feet in depth. The composition of the fill varies randomly from a clayey silt to a silty sand and gravel.

The overburden is directly underlain by gneiss bedrock, the surface of which was encountered between elevations 712 and 751.5.

The stratigraphy encountered at the borings put down is plotted on the Record of Borelog sheets. Stratigraphical profiles and sections, inferred from this data, are plotted on Drawing No. 71-11074A and B. A resumé of the subsoil and bedrock, encountered from ground surface downward in this area, is presented in the following subsections.

4.2) Fill Material:

Fill has been placed in many areas for site grading purposes. Where encountered the thickness of the fill varies from 5 to 6.5 feet. The composition of the fill was found to be quite variable ranging from a clayey silt with some sand and gravel at some locations to a silty sand and gravel at others.

Standard penetration testing, carried out within the fill,gave 'N' values which varied from 2 to 12 blows/ft. These values would indicate that the fill has not been subjected to much of a compactive effort.

4.3) Parent Granular Stratum:

The fill, where it is present, and the thin (1 foot max.) topsoil cover elsewhere, is underlain by the predominant stratum across the site, which is composed of a silty sand and gravel to sandy gravel. The thickness of the stratum varies from 1.5 to 40.5 feet, being most extensive on the south bank of the Indian River, in the vicinity of the Lakeview Restaurant.

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4. <u>SUBSOIL AND BEDROCK CONDITIONS</u>: (cont'd) ...

4.3) <u>Parent Granular Stratum</u>: (cont'd) ...

At random locations wood fibre and related organic matter are present in the upper 3 to 7 feet of the stratum. Numerous boulders are present throughout, the boulders vary anywhere from 8 to 22 inches in size. Grain-size distribution testing was carried out on samples of the granular deposit, obtained with 2" inch O.D. sampling equipment. The results of this testing are plotted in envelope form on Figure #1, in the Appendix of this report.

Standard penetration testing was carried out within the stratum, the results are plotted on the Record of Borelog sheets. The testing gave 'N' values which ranged from 3 blows/ft. to 100 blows for 4 inches, being typically between 30 and 80 blows/ft. Based on these values it is estimated that the relative density of the stratum varies from loose to very dense, being generally in the dense to very dense range.

4.4) <u>Clayey Silt with Some Sand and Gravel</u>:

At B.H.'s #9 and 11 a deposit of firm to hard ('N' values 4 blows/ft. to greater than 100 blows/ft.) clayey silt with sand and gravel is sandwiched within the parent granular deposit, while at B.H. #5 the cohesive material directly overlies bedrock. The thickness of this deposit ranges from 3.5 to 5.5 feet. Numerou sand and silt seams and layers, up to 4 inches thick, are present throughout the deposit. Grain-size distribution curves were obtained for three samples of the cohesive material; these are plotted on Figure #2 in the Appendix.

Atterberg limit tests were carried out on samples from the stratum; these are plotted on the borelogs as well as on the Plasticity Chart, Figure #3. The results indicate that the cohesive material is inorganic with a plasticity in the low range. The natural moisture content is generally at or slightly below the plastic limit.

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4. SUBSOIL AND BEDROCK CONDITIONS: (cont'd) ...

4.5) Gneiss Bedrock:

The overburden is directly underlain by metamorphic bedrock of Precambrian Age. The bedrock was proven in 24 of the boreholes by obtaining between 4 and 16 feet of either BX or AXT size rock core samples.

The bedrock core samples were examined by Mr. K. Ingham, Geologist, Department of Transportation and Communications. Mr. Ingham presented the results of his bedrock examination, as well as an interpretation of the geologic conditions existing at this site, in a letter to this office, dated October 25, 1971. A copy of this letter is appended to this report. The bedrock description, presented in the paragraphs to follow, is an excerpt from this letter.

The dominant type of rock encountered in the drilling and exposed in the immediate vicinity of the site is a medium grained pink granite gneiss displaying a pronounced lineation dipping steeply at approximately 50 degrees. Bands of dark grey quartz biotite gneiss with less conspicuous lineation are a subordinate type but form layers parallel to the granite gneiss. Veins of granite pegmatite are common cutting across the gneissic structure. These were the only types intersected in the bedrock and most of the boulders overlying the bedrock appear to be the same.

"Fractures are common and apparently form a fairly close pattern. Evidence of four sets of fractures is present in the cores, one sub-parallel to the lineation, a horizontal set, a vertical set and less commonly a steeply inclined set. These joints are generally tight except where noted. In some instances, open joints are sandfilled near the surface."

The bedrock contour map, derived from rock elevations in 24 boreholes, gives a more or less straightforward knob-and-valley topography. The lowest elevations (710 to 715) follow a course underlying the restaurant which may be an earlier channel of the Indian River, however, there is insufficient evidence to establish this.

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4. <u>SUBSOIL AND BEDROCK CONDITIONS</u>: (cont'd) ...

4.5) <u>Gneiss Bedrock</u>: (cont'd) ...

The rock exposed north of the service station drops by a series of steps down to this channel. An area of rock of lower general relief exists between here and the present lock excavation and south of this the rock rises to an area of higher relief.

The only unusual rock conditions were noted in the vicinity of B.H. #6 and in the present Indian River channel. The latter case is relatively minor, where loose slabs of rock 1.0 to 1.5 feet in thickness appear to lie directly on the bedrock. Near B.H. #6 the rock appears to drop sharply towards the river, coupled with several open fractures observed in B.H. #6, there may be an unstable area of rock in this immediate vicinity. Excavation of the rock surface would be required to confirm this possibility.

A detailed description of the bedrock conditions, existing at each boring location, is presented on the respective borelog sheets. In view of the presence of numerous boulders in the overburden, just above bedrock, it is often difficult to delineate the precise boundary between the bedrock surface and the lower bouldery granular stratum.

5. <u>GROUNDWATER CONDITIONS</u>:

Groundwater level observations were carried out, during the period of the investigation, in the open boreholes. The observations are presented on the individual borelog sheets as well as on Drawing No. 71-11074A and B. The results indicate that the groundwater level varies between elevation 738.5 and 740.5, corresponding to depths of from 2 to 12.5 feet below existing ground surface. The water level in the Indian River, at the time of the investigation, was at about elevation 739.

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6. EXISTING STRUCTURE:

The existing structure, crossing the lock and the Indian River, is about 182 feet long and 18 feet wide having steel girders and steel truss with a timber deck. The northern portion, over the lock, swings in order to allow navigation along the Indian River. The southern portion over the Indian River is fixed. The study of the available bridge drawings appear to indicate that the north abutment, as well as the pivot mechanism foundation, are founded directly on or within the bedrock. Foundation details for the other pier and the south abutment are not available. It is quite possible that these are not founded on bedrock.

The profile grade of Hwy. #118, in the vicinity of the existing structure, varies between elevations 753 and 755. At this grade the heights of the approach embankments, above the original ground surface, are nominal.

The existing structure appears to be performing satisfactorily without any visible sign of distress.

7. DISCUSSION AND RECOMMENDATIONS:

7.1) General:

It is proposed to realign Hwy. #118 (refer to Line 'E' on Drawing 71-11074A), in the Village of Port Carling, District of Muskoka. The new bridge will accommodate two-lane traffic. This realignment will necessitate the replacement of the existing swing bridge across the lock and Indian River with a new one. The proposed 40 foot wide structure will have two spans. The most northern span, over the lock, will swing, while the most southerly one, over the Indian River, will be fixed. A massive swing mechanism will be located within the northern span.

Associated retaining walls will also be constructed. These walls, which will vary from 25 to 110 feet in length and 10 to 18 feet in clear height, are shown in plan on Drawing No. 71-11074A.

It is understood that the existing grade of Hwy. #118 will be maintained. This being the case the approach embankments will be of nominal height.

Hwy. #118 will have to be temporarily detoured during the construction period.

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7. DISCUSSION AND RECOMMENDATIONS: (cont'd) ...

7.1) General: (cont'd) ...

The location of this detour, in relation to Hwy. #118, is shown on Drawing No. 71-11074A. A temporary structure will have to be constructed at the crossing of the detour and Indian River. This structure will have to be approximately 140 feet long. A Bailey bridge with simply supported spans, incorporating a centre support, could be used at this site.

The predominant stratum across the site is composed of a 1.5 to 40.5 feet thick loose to very dense silty sand and gravel to sandy gravel, with numerous boulders throughout. The overburden deposits are periodically overlain by between a 5 and 6.5 foot depth of fill. The overburden is directly underlain by gneiss bedrock, the surface of which was encountered between elevations 712 and 751.5.

Comments and discussion of the factors relating to foundation design of the permanent as well as temporary structure, and associated retaining walls are presented in the sections to follow.

7.2) Proposed New Structure:

Foundation recommendations, pertaining to the individual structure elements, will be discussed separately.

7.2.1) North Abutment Foundation (Refer to B.H.'s No. 1 and 2):

Bedrock is located at a shallow depth below ground surface (2.5 to 6.5 feet) at this location. This abutment could, therefore, be founded on a spread footing located beneath the upper broken and fractured portion of the bedrock, using an allowable bearing value of 10 t.s.f. in design.

The bedrock surface increases in elevation in an easterly direction. It is estimated that the base of the footing would vary from elevation 737 (west end) to elevation 742 (east end).

The excavation for the north abutment will extend below the groundwater level recorded during the period of the investigation.

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7. DISCUSSION AND RECOMMENDATIONS: (cont'd) ...

7.2) Proposed New Structure: (cont'd) ...

7.2.1) North Abutment Foundation (Refer to B.H.'s No. 1 and 2): (cont'd)...

Since the overburden in this area is granular and bouldery in nature some seepage can be expected in the excavation. This groundwater seepage, as well as any uncontrolled surface runoff could be handled using standard techniques, such as pumping from sumps. The southern portion of this excavation will be located in close proximity to the existing lock. It may be necessary to prevent the water, within the lock channel, from seeping into the excavation. This could be accomplished by driving closed interlocking steel sheeting adjacent to the lock area.

If the closed type abutment is designed as a rigid wall, then a coefficient of earth pressure at rest (Ko) of 0.5 should be assumed for the granular material placed behind the abutment when designing the wall section. In order to relieve the buildup of excess hydrostatic pressure behind the retaining structure, suitable drainage measures should be provided.

The horizontal resistance of the abutment wall may be computed using a coefficient of friction of 0.75 between the rough footing and the gneiss bedrock.

7.2.2) South Abutment Foundation (Refer B.H.'s No. 13 and 14):

This abutment could be founded on a spread footing located in the bouldery silty sand and gravel deposit. A footing, founded at elevation 740, could be designed using an allowable bearing value of 3.0 t.s.f. The excavation for this abutment will be located at or slightly above the groundwater level. No major dewatering problems are, therefore, anticipated. Any minor seepage, due to surface runoff, could be controlled using sumps. The northern portion of the excavation will be in close proximity to the Indian River. Due to space restrictions in this area the face of the excavation may have to be sheeted.

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7. DISCUSSION AND RECOMMENDATIONS: (cont'd) ...

7.2) Proposed New Structure: (cont'd) ...

7.2.2) South Abutment Foundation (Refer B.H.'s No. 13 and 14): (cont'd) ...

Such sheeting should be carried out in accordance with the regulations set down in the Trench Excavators Act.

The bouldery granular subsoil, located beneath the abutment, will settle due to the applied footing pressure. This settlement, which will be elastic in nature, should not exceed 1/2 inch, provided the subsoil is not loosened by uncontrolled surface runoff or construction operations. In this regard it is recommended that, as soon as the founding elevation is reached, the base be protected by covering it with a 6-inch thick lean concrete working slab.

It is understood that this abutment footing may be located adjacent to the Indian River. This would mean that the subsoil, beneath the footing, would be subjected to the river scour action. Under such circumstances, the footing would have to be located beneath the active scour zone - i.e. it would be situated at a lower elevation than quoted previously. At this lower founding elevation the excavation base will be located below the groundwater, as well as the river water level. Since the overburden is relatively pervious some groundwater seepage can be expected into the excavation. A dewatering scheme will, therefore, be required. One possible scheme would be to carry out the excavation from within an enclosure formed of interlocking steel sheeting. Theoretically the sheeting should be driven to a depth below the base of the excavation equal to the unbalanced hydrostatic pressure head above this level, in order to prevent boiling of the subsoil. It may be difficult to drive the sheeting through the bouldery zones in the granular deposit to reach the required elevation. An alternative method would be to drive the sheeting to an elevation slightly below the base of the excavation and to excavate below water inside the enclosure.

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7. DISCUSSION AND RECOMMENDATIONS: (cont'd) ...

7.2) Proposed New Structure: (cont'd) ...

7.2.2) South Abutment Foundation (Refer B.H.'s No. 13 and 14): (cont'd) ...

Once the base of the excavation is reached a seal of tremie concrete should be poured under water. The thickness of this seal should be sufficient to counterbalance the excess hydrostatic water pressure head existing at the base of the excavation. Once the seal is in place the enclosure can be pumped out and construction can proceed in the dry.

The footing can be designed using the allowable bearing value quoted previously.

The closed type wall should be designed for the lateral earth pressure component discussed in the previous section.

The horizontal resistance of the abutment footing may be computed using a coefficient of friction of 0.6 between the rough footing and the granular stratum.

7.2.3) Pivot Mechanism Foundation (Refer to B.H.'s #5, 6, 7, 8 and 9):

This circular hydraulic mechanism will be approximately 40 feet in diameter. It will be located in the island between the lock and Indian River. In the eastern portion of the proposed location the bedrock surface is located at a relatively shallow depth below ground surface (6 to 9.5 feet - elevation 734.5 to 738). In the western portion, however, the bedrock surfaces dip to as low as elevation 724.5 (18.5 feet below ground surface). Due to a) the heterogeneous composition of the fill and parent overburden deposits and b) the fact that the heavily loaded pivot mechanism is extremely settlement sensitive, it is recommended that this element be founded directly on or within the moderately fractured bedrock, using an allowable bearing pressure of up to 10 t.s.f. in design. The base of the foundation could be founded as follows:

Elev. 724.5 (West End) to Elev. 737 (East End)

It is suggested that the excavation be inspected prior to pouring concrete, in order to ensure that the foundation will be founded directly on bedrock as proposed.

7. DISCUSSION AND RECOMMENDATIONS: (cont'd) ...

7.2) Proposed New Structure: (cont'd) ...

7.2.3) Pivot Mechanism Foundation (Refer to B.H.'s #5, 6, 7, 8 and 9): (cont'd) ...

The excavation, which will be primarily carried out in bouldery granular deposits will extend between 6 and 18.5 feet below the existing ground surface. Further, the base of the excavation will be located anywhere from 2 to 15 feet below the groundwater level. Groundwater seepage can be anticipated through the pervious granular overburden as well as from the river. Therefore, a positive dewatering scheme will be necessary during construction. One scheme would be to carry out the excavation from within an enclosed steel sheet pile wall cofferdam. In some localized areas, such as in the south-western portion of the foundation (refer to B.H. #8), it will be extremely difficult to advance the steel sheeting through the lower bouldery portion of the overburden. In these areas, it may be necessary to drive the sheeting as far as practically possible then remove the boulders or other obstacles under water from beneath the sheeting and continue driving to the next obstruction where this process will be repeated. Using this method it may be possible to reach the surface of the bedrock.

The bedrock surface is sloping in nature; further, the upper 1 to 2 feet is often fractured and thus quite pervious. This being the case it may be difficult to effect a satisfactory water tight seal at the contact of the steel sheeting with the bedrock. Such a problem could be overcome by pouring a seal of tremie concrete under water as discussed in detail in Subsection 7.2.2.).

7. DISCUSSION AND RECOMMENDATIONS: (cont'd) ...

7.2) Proposed New Structure: (cont'd)...

7.2.4) Centre Pier Foundation (Refer to B.H.'s #22, 23 and 25):

The centre pier will be located within the channel of the Indian River. It can be founded on the moderately fractured bedrock, located below the upper badly fractured zone (maximum thickness 1.5 feet). The base of the footing would thus be located as follows:

Elev. 721 (West End) to Elev. 728 (East End)

A footing, founded as recommended, could be designed using an allowable bearing value of up to 10.0 t.s.f.

A dewatering scheme will be required. One possibility would be to carry out the excavation from within an enclosure formed of interlocking steel sheet piling. The granular material, overlying bedrock, is bouldery. This being the case it may be necessary to advance the sheeting, using techniques described in detail in Subsection 7.2.3). For reasons discussed previously, it may be difficult to effect a proper water-tight seal at the contact of the sheeting and bedrock. It may be necessary to form a seal of tremie concrete under water at the base of the excavation prior to dewatering the enclosure.

7.3) Alternate Type of Foundation - Caissons Socketed Into Bedrock:

If spread footing foundations are used to support the south abutment, pivot mechanism and pier, major dewatering problems can be anticipated. Possible dewatering schemes for these elements were discussed in Subsections 7.2.2), 7.2.3), and 7.2.4), respectively.

One method of limiting these dewatering complications would be to found these three structure elements on caissons socketed at least 5 feet into the gneiss bedrock. Caissons could be installed through the bouldery overburden down into bedrock using churn drilling operations. The allowable bearing pressure of the caissons will be dependent on the diameter adopted.

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7. DISCUSSION AND RECOMMENDATIONS: (cont'd) ...

7.3) Alternate Type of Foundation - Caissons Socketed Into Bedrock: (cont'd) ...

For preliminary estimating purposes an allowable load of 500 and 900 tons per caisson can be used in designing a 3 and 4 foot diameter installation, respectively.

As noted on the individual borelog sheets, some jointing is present in the bedrock to a considerable depth. If some of these joints are open at any of the caisson locations then groundwater may come into the installation. If this occurs, it may be necessary to tremie concrete under water in the lower portion of the caisson.

At the south abutment location the pile cap may be situated at an elevation above the water level. As such the soil beneath the cap may be subjected to scour action of the river. The pile cap should, therefore, be designed as a free standing beam supported on the caisson.

7.4) Approach Fills:

The approach embankments will have a maximum height of 6 feet above the surrounding ground surface in certain locations. The foundation subsoil will be primarily the bouldery granular deposit. No stability problems are anticipated. Further, the settlement of the foundation subsoil will be negligible in magnitude.

7.5) Proposed Retaining Walls:

Tentatively four retaining walls may be constructed in connection with this project. The location and extent of the proposed walls are shown in plan of Drawing No. 71-11074A. This office has been advised that design details for these walls are not finalized at this particular time. Detailed recommendations, pertaining to foundation design, will be submitted once this information becomes available. In the interim some general comments, with regard to foundation design, will be discussed in the paragraphs to follow.

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7. DISCUSSION AND RECOMMENDATIONS: (cont'd) ...

7.5) Proposed Retaining Walls: (cont'd) ...

i) Retaining Walls A-1 and A-2:

These walls can be founded on spread footings founded in the bouldery granular deposit, using an allowable bearing value of up to 2.5 t.s.f. Founding elevation considerations will be similar to those discussed for the south abutment (refer to Subsection 7.2.2), i.e. the scour action of Indian River will have to be taken into consideration. A major dewatering scheme will be required. A scheme, similar to that proposed in Subsection 7.2.2) would have to be employed.

As an alternative the walls could be supported on caisssons socketed into bedrock as discussed previously.

ii) Retaining Walls B-1 and B-2:

These walls could be founded on spread footings located directly on or within the gneiss bedrock, using an allowable bearing value of 10.0 t.s.f. in design. These walls will be placed within the existing river channel as such a dewatering scheme will be required. A steel sheet pile cofferdam could be used for this purpose. It may be difficult to penetrate through the bouldery overburden; a method of accomplishing this was outlined in detail in Subsection 7.2.3).

As an alternative caissons socketed into bedrock could be employed.

If the walls are designed as rigid frames, then a coefficient of earth pressure at rest (Ko) of 0.5 should be assumed for the granular material placed behind the wall, when designing the wall sections. However, if some movement of the top of the wall is permitted, then a coefficient of active earth pressure (Ka) of 0.33 can be used. In order to relieve the buildup of excess hydrostatic pressure behind the walls, suitable drainage measures should be provided.

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7. DISCUSSION AND RECOMMENDATIONS: (cont'd) ...

7.6) Temporary Detour Structure:

A two-span (approx. 70' - 70') Bailey bridge structure could be used at the proposed detour crossing. The structure could be supported on rock-filled timber cribs; one located on either bank of the Indian River, with the other on the island between the lock and the river. The rock-filled cribs will be founded on loose to dense granular soils. An allowable bearing value of up to 1.0 t.s.f. can be used when designing the cribs.

The underlying **granular foundation** subsoil will settle due to the crib loading. This settlement could be of the order of 2 to 3 inches. It will be elastic in nature; i.e. take place during or immediately following the construction period.

8. MISCELLANEOUS:

This project was carried out between the period of August 11 and September 29, 1971, under the direct supervision of Mr. V. Korlu, Project Foundation Engineer.

The drilling equipment used was owned and operated by Master Soil Investigation Ltd., Toronto, Ontario.

This report was written by Mr. B. T. Darch, Senior Foundation Engineer with the assistance of Mr. V. Korlu. The entire project was carried out under the general supervision of Mr. M. Devata, Supervising Foundation Engineer, who reviewed this report.

B.T. Darch

B. T. Darch, P. Eng.

REGISTERED PROFESSIONAL ENGINEER
B. T. DARCH
PROVINCE OF ONTARIO

M Devata

M. Devata, P. Eng.

BTD/ao
November 17, 1971.

APPENDIX I

FORM OD-MT ■26 (REV. 1969) OFFICE REPORT O■ SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 342 +44 o/s 26' Rt. Ø Line 'E' ORIGINATED BY VK
W.P. 401-64 BORING DATE Aug. 11, 1971 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing,BXL Rock Core CHECKED BY [illegible]

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (o UNCONFINED, + FIELD VANE, ⊗ QUICK TRIAXIAL, × LAB. VANE)	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT					
744.5	Ground Level									
0.0	Clayey silt with sand & gravel (Fill) Random layers of sandy si.up to 6" thick)					740				740.3 WL in open BH Aug.11/71
738.3	Stiff to Hard		1	SS	89			o ├		
6.2	Broken - Loose		2	BXL						
	Bedrock		3	BXL	85%					
728.5	(Moderately Fractured)		4	BXL	83%	730				
16.0	End of Borehole					720				

DETAILED BEDROCK DESCRIPTION

Elev. 738.3 to 736.9: fine grained quartz biotite gneiss (badly broken, loose in this section)

Elev. 736.9 to 733.5: fine grained quartz biotite gneiss (moderately fractured, fractures sub-parallel to lineation, horizontal and steeply inclined)

Elev. 733.5 to 728.5: medium to coarse grain'ed granite gneiss (prominent lineation dipping at approx. 50°, moderately fractured as above)

FORM OD-MT 6 (REV. 1969) OFFICE REPORT SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 342 + 48 o/s 20' Lt. Ø Line 'E' ORIGINATED BY VK
W.P. 401-64 BORING DATE Aug. 12, 1971 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing, BXL and AXT Rock Core CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W — WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
745.0	Ground Level									
0.0	Gravel. Brown. Very Dense		1	SS	62/6"					
742.7	Broken		2	BXL	63%					▼ 741.5
2.3			3	BXL	100%					
						740				
	Fractured		4	AXT	90%					
	Bedrock		5	AXT	100%					
			6	AXT	100%	730				
726.7	Sound									
18,3	End of Borehole									
						720				

DETAILED BEDROCK DESCRIPTION:

Elev. 742.7 to 735.7: medium grained granite gneiss (inclined lineation and badly fractured possibly loose in upper 0.8 ft. inclined and vertical fractures, vertical fractures probably open down to Elev. 735.7)

Elev. 735.7 to 726.7: medium grainted granite gneiss (sound)

FORM OD-MT [illegible]26 (REV. 1969)   OFFICE REPORT [illegible] SOIL EXPLORATION

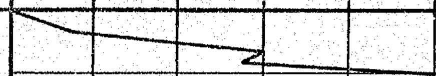
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 71-11074   LOCATION Sta. 342 + 47 o/s 53' Rt. Ø Line 'E'   ORIGINATED BY VK
W.P. 401-64   BORING DATE Aug. 16, 1971   COMPILED BY VK
DATUM Geodetic   BOREHOLE TYPE Washboring-BX Casing, BXL & AXT Rock Core; Cone   CHECKED BY [signature]

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (20 40 60 80 100); SHEAR STRENGTH P.S.F. (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE)	LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
743.7	Ground Level									
0.0	Silty sand with gravel Compact to Very Dense					740				▼741. WL in open BH Aug.16/71
	Boulder Zone (boulders up to 22" in size)		1	BXL	83%					
733.7			2	AXT	56%					
10.0	Weathered		3	AXT	100%					
	Bedrock Sound		4	AXT	100%	730				
727.2										
16.5	End of Borehole					720				



DETAILED BEDROCK DESCRIPTION:

Elev. 733.7 to 727.7: fine grained quartz biotite gneiss (inclined lineation, upper 2 ft. weathered, conspicuous inclined fractures)

FORM OD-MT ■26 (REV. 1969) OFFICE REPORT O■ SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

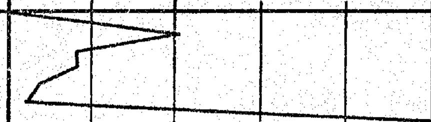
FOUNDATION SECTION

JOB 71-11074 LOCATION Sta.341 + 63 o/s 28' Rt. Ø Line 'E; ORIGINATED BY VK

W.P. 401-64 BORING DATE Aug. 18, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing, BXL & AXT Rock Core; Cone CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 40 60 100 — SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w — WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT					
758.4	Ground Level									
0.0	Clayey silt with some sand & gravel (Fill)									
753.4	Brown Stiff		1	SS	6					
751.7	Silty sand with gravel Compact		2	SS	25			o		25 45 26 4
			3	BXL	100%					
6.7	Badly fractured					750				Hole dry
	Bedrock		4	AXT	100%					
746.4	Sound									
12.0	End of Borehole									
						740				



DETAILED BEDROCK DESCRIPTION

Elev. 751.7 to 746.4: medium grained granite gneiss with minor sections of granite pegmatite and quartz biotite gneiss (badly fractured in the upper 0.8 ft. - sound below this depth)

FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB 71-11074
W.P. 401-64
DATUM Geodetic

LOCATION Sta. 343 + 15 o/s 21' Lt. ℄ Line 'E'
BORING DATE Aug. 19, 1971
BOREHOLE TYPE Washboring, BX Casing, AXT Rock Core; Cone

ORIGINATED BY VK
COMPILED BY VK
CHECKED BY [signature]

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS GR.SA.SI.CL.
744.8	Ground Level						
0.0	Silty sand, trace of clay (Fill)		1	SS	5	740	
738.8	Loose		2	SS	7		22 28 26 24
6.0	Clayey silt with some sand & gravel.						
735.0	Stiff to Hard		3	SS	100/5"		
9.5	weathered		4	AXT	100%	730	
	Bedrock						
726.8	(moderately fractured)		5	AXT	100%		
18.0	End of Borehole					720	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20, 40, 60, 80, 100

SHEAR STRENGTH P.S.F.: o UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB. VANE

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w

WATER CONTENT %: 10, 20, 30

BULK DENSITY γ P.C.F.

DETAILED BEDROCK DESCRIPTION

Elev. 735.3 to 734.6: granite pegmatite (weathered)

Elev. 734.6 to 726.8: granite gneiss (inclined lineation, weathered in the upper 0.8 ft., moderately fractured below this depth).

FORM OD-MT 26 (REV. 1969) OFFICE REPORT SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 33 o/s 21' Lt. Ø Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE August 23, 1971 COMPILED BY

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core; Cone CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS GR. SA. SI. CL.
743.9	Ground Level						
0.0	Silty sand with some gravel.						
737.9	Compact to Dense		1	SS	34	740	29 42 22 7; 749.
6.0	Bedrock		2	BXL	100%		WL in open BH Aug.23/71
	Fractured		3	BXL	100%		
	Sand, some wood and organic matter.		4	BXL	30%		
	Fractured					730	
	Bedrock		5	AXT	83%		
720.7	Sound		6	AXT	100%		
23.2	End of Borehole					720	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

DETAILED BEDROCK DESCRIPTION

Elev. 737.9 to 732.1: granite gneiss (lineation inclined, approx. 50°, evidence of open fractures in this section - high angle or vertical fractures at depth of 7.3', 10.6', 11.3' and 11.8', parallel to the lineation at depth of 8.5' and 9.3').

Elev. 732.1 to 730.6: medium and fine sand plus some wood, (inclined fracture surface at elev. 730.6)

Elev. 730.6 to 729.9: granite gneiss (possible open inclined fractures at depths of 13.6' and 14.0')

Elev. 729.9 to 728.9: granite gneiss (fractured)

Elev. 728.9 to 727.9: quartz biotite gneidd (fractured)

Elev. 727.9 to 720.7: granite gneiss (sound inclined fracture at depth 19')

FORM OD-MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 24 o/s 26' Lt. Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE August 25, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring, BX Casing, BXL & AXT Rock Core; Cone CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
744.4	Ground Level									
0.0	Silty sand, some clay (Fill) Loose									
739.5			1	SS	8	740				
	Silty sand & gravel. Very Dense		2	SS	150/6"			o		35 57 (8)
737.9 / 6.5	Concrete		2A	BXL	100%					
734.8 / 9.6	Bedrock		3	AXT	100%					
						730				
726.4	(Moderately Fractured)		4	AXT	100%					
18.0	End of Borehole					720				

DETAILED BEDROCK DESCRIPTION

Elev. 734.8 to 726.4: granite gneiss (0.2 ft. granite pegmatite vein at a depth of 10.4 moderately fractured).

FORM OD-MT 26 (REV. 1969) OFFICE REPORT SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 37 o/s 13' Rt. Ø Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE August 27, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core: Cone CHECKED BY

ELEV. DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT. PLOT	NUMBER (SAMPLES)	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
743.1	Ground Level						
0.0	Clayey silt with some sand & gravel (Fill)						▼739.5 WL in open BH Aug.27/71
738.1	Firm		1	SS	2	740	
5.0	Silty sand and gravel. Compact		2	SS	25		
	Bouldery Zone		2A	BXL	84%		
	boulders up to 12" in size		2B	AXT	42%	730	
			2C	AXT	58%		
724.5	Very Dense						
18.6			3	AXT	72%		
			4	AXT	100%	720	
	Bedrock		5	AXT	88%		
712.0	(Moderately fractured)		6	AXT	100%		
31.1	End of Borehole					710	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

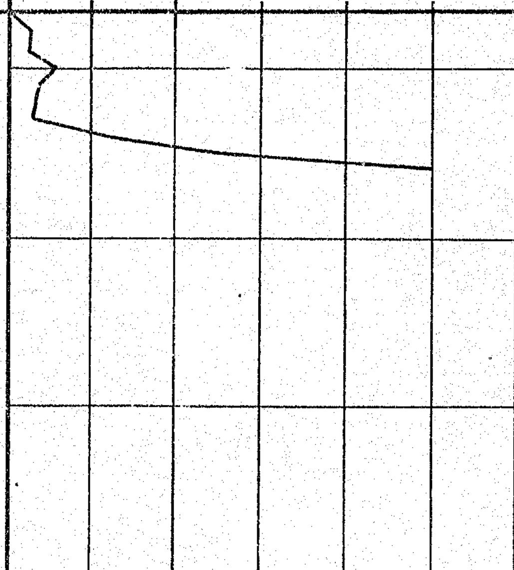
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L PLASTIC LIMIT —— w_P WATER CONTENT —— w

WATER CONTENT %

BULK DENSITY γ P.C.F.

GR. SA. SI. CL.



DETAILED BEDROCK DESCRIPTION

Elev. 724.5 to 712.0: granite gneiss (generally medium grained, coarse grained between depths of 25' to 28', moderately fractured - prominent near-vertical fractures between depths of 21' to 22.5')

FORM OD-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 9

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 20 o/s 18' Rt. Ø Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE August 30, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core; Cone CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS GR. SA. SI. CL.
743.0	Ground Level					
0.0	Silty sand and gravel some clay (Fill)					
738.0	Compact	1	SS	12	740	▼ 739. WL in open BH Aug.30/71
5.0	Sand and gravel.					
735.0	Compact	2	SS	29		
8.0	Clayey silt with some sand and gravel.	3	SS	4		16 31 43 10
729.5	Stiff	4	SS	7	730	
13.5	Sand and gravel.					
726.5	Very Dense	5	SS	78		
16.5	Bedrock	6	BXL	100%		
		7	AXT	88%		
		8	AXT	75%	720	
		9	AXT	80%		
	Fractured					
		10	AXT	80%	710	
707.0	Sound	11	AXT	100%		
36.0	End of Borehole				700	

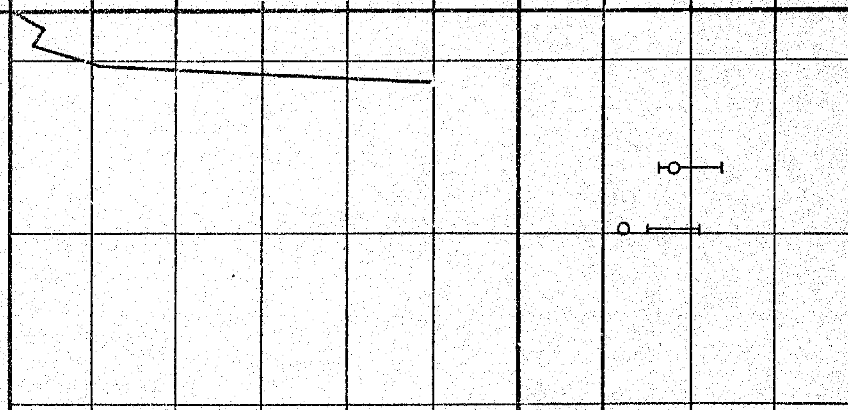
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.



DETAILED BEDROCK DESCRIPTION

Elev. 726.5 to 714.5: granite gneiss (inclined lineation approx. 50°, minor horizontal and inclined fractures)

Elev. 714.5 to 713.0: granite pegmatite (fractured)

Elev. 713.0 to 707.0: granite gneiss (sound)

FORM OD-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 10

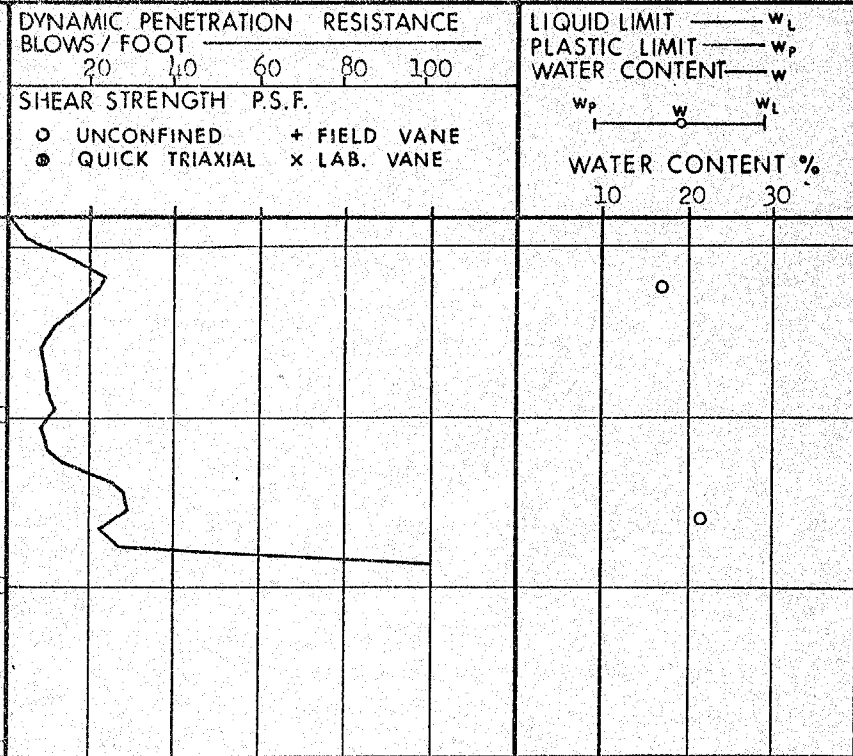
FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 24 o/s 50' Rt. Ø Line'E; ORIGINATED BY VK

W.P. 401-64 BORING DATE Sept. 8, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL AND AXT Rock Core; Cone CHECKED BY [signature]

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	REMARKS GR.SA.SI.CL.
741.7	Ground Level				
0.0	Silty sand with some gravel (Fill)				▼739.5 WL in open BH Sept.3/71
736.7	Compact	1	SS	10	
5.0	Silty sand and gravel (occ.fragments of decayed wood & related organic matter) (boulders up to 22" in size below el.736.5)	2	SS	63	
		3	SS	100/6"	
		4	SS	100/2"	
		5	SS	13	47 47 (6)
720.7	Compact to Very Dense	6	BXL	43%	
21.0	Bedrock				
714.7	(Moderately fractured)	7	AXT	100%	
27.0	End of Borehole				



DETAILED BEDROCK DESCRIPTION

Elev. 720.7 to 714.7: quartz biotite gneiss (minor near vertical fractures)

FORM OD-M 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 11

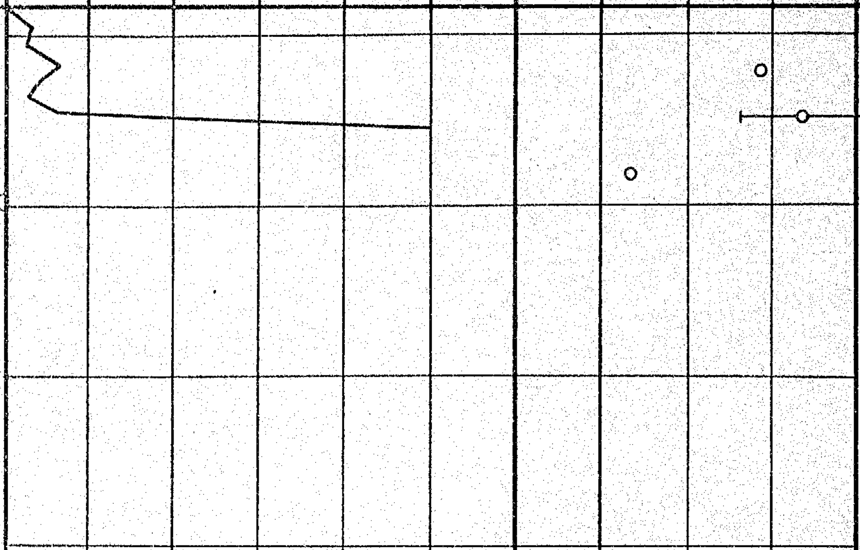
FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 29 o/s 36' Rt. ℄ Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE Sept. 9, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring, BX Casing, BXT & AXT Rock Core; Cone CHECKED BY

SOIL PROFILE: ELEV. / DEPTH	DESCRIPTION	SAMPLES: NUMBER	TYPE	BLOWS/FOOT	REMARKS (GR. SA. SI. CL.)
741.5	Ground Level				
0.0 / 737.5	Silty sand, trace of clay. Loose				▼ 739.5
		1	SS	3	
4.0 / 732.5	Clayey silt with some sand(occ.seams of sand up 4" thick) Hard	2	SS	107/10"	3 19 58 20
9.0 / 718.0	Silty sand & gravel (boulders up to 14" in size below El.731.5)	3	SS	175/11"	2 44 (14)
		3A	BXL	33%	
		3B	BXL	47%	
		3C	BXL	76%	
	Dense to Very Dense	4	SS	31	
23.5 / 710.0	Badly fractured Bedrock (moderately fractured)	5	AXT	71%	
		6	AXT	100%	
		7	AXT	100%	
31.5	End of Borehole				



DETAILED BEDROCK DESCRIPTION

Elev. 718.0 to 717.0: granite gneiss (evidence of horizontal and inclined open fractures in this section)

Elev. 717.0 to 710.0: quartz biotite gneiss (frequently closed near-vertical fractures).

FORM OD-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 12

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 344 + 51 o/s 40' Lt. Ø Line 'E' ORIGINATED BY VK
W.P. 401-64 BORING DATE August 30, 1971 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Winkie Drill - AXT Rock Core CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE)	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT					
741.3	Ground Level									
0.0	Silty sand and gravel (boulders up to 14" in size below el.737.) Compact to Very Dense		1	AXT	28%	740				739.5 WL in open BH Sug.30/71
722.3						730				
19.0	Bedrock Fractured		2	AXT	67%	720				
714.3			3	AXT	100%					
27.0	End of Borehole					710				

DETAILED BEDROCK DESCRIPTION

Elev. 722.3 to 716.3: granite gneiss (lineation variable in this section - fractured)

Elev. 716.3 to 714.3: quartz biotite gneiss (sound)

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 13

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 344 + 39 o/s 28' Lt. Ø Line 'E' ORIGINATED BY VK
W.P. 401-64 BORING DATE Sept. 22, 1971 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core; CHECKED BY

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	WATER CONTENT %	REMARKS GR.SA.SI.CL.
749.2	Ground Level						
0.0	Silty sand & gravel	1A	BXL	37%			
		1	SS	11			
	(numerous boulders up	2	SS	25	740		▼ 739.5
	to 22" in size through	3	SS	80		o	83 15 (2)
	out)	4	SS	56			
		4A	BXL	14%			
		5	SS	60/1"	730	o	41 47 (12)
	Compact to Very Dense						
723.7 / 25.5	Badly fractured	6	BXL	70%			
718.7	Bedrock (moderately fractured)	7	AXT	100%	720		
30.5	End of Borehole				710		

DETAILED BEDROCK DESCRIPTION

Elev. 723.7 to 722.2: granite gneiss (several open fractures, possibly loose in this section)

Elev. 722.2 to 718.7: granite gneiss (moderately fractured)

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 14

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 344 + 28 o/s 10' Rt. Ø Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE Sept. 15, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core; Cone CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
752.0	Ground Level				
0.0					750
	Sily sand and gravel	1	SS	24	
		1A	BXL	100%	
		2	SS	11	
	(occ. boulders up to				
		3	SS	63	
	21" in size throughou	3A	BXL	88%	740
		4	SS	50	
		5	SS	40	
		5A	BXL	100%	
		6	SS	67	
		6A	BXL	100%	730
		7	SS	23	
		8	SS	33	
		9	SS	23	
		10	SS	100/6"	720
	Compact to Very Dense	10A	BXL	25%	
		11	SS	93	
716.5					
35.5	Bedrock	12	AXT	100%	
					710
707.2	Sound	13	AXT	100%	
44.8	End of Borehole				

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F.

O UNCONFINED + FIELD VANE

● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT — w_L

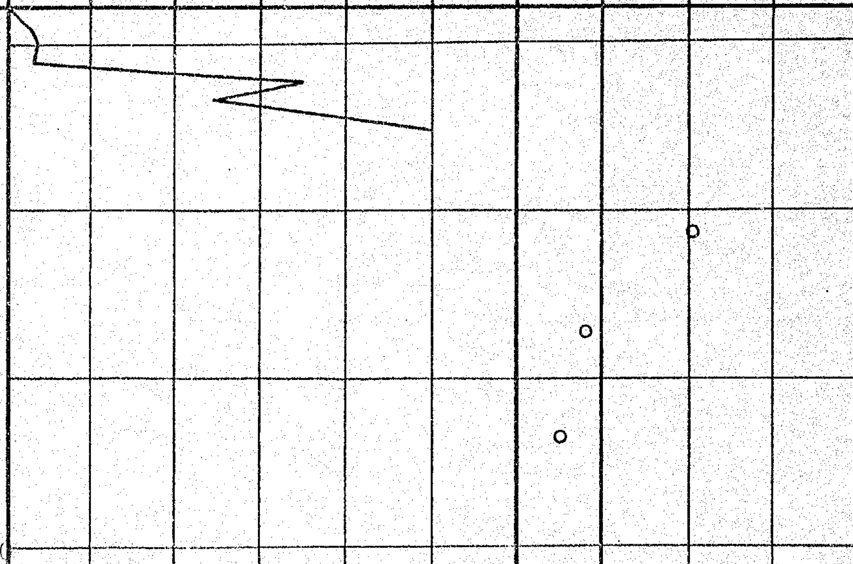
PLASTIC LIMIT — w_P

WATER CONTENT — w

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

REMARKS GR.SA.SI.CL.



REMARKS: 9 85 (6)

DETAILED BEDROCK DESCRIPTION

Elev. 716.5 to 713.7: granite gneiss (sound

Elev. 713.7 to 712.2: quartz biotite gneiss (sound)

Elev. 712.2 to 707.2: granite gneiss (sound minor fractures sub-parallel to the lineation)

FORM OD-M[illegible]126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 15

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 97 o/s 64' Rt. Ø Line 'E' ORIGINATED BY VK
W.P. 401-64 BORING DATE Sept. 8, 1971 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL Rock Core; Cone CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ⊗ QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
742.6	Ground Level									
0.0	Silty sand & gravel					740				738.5
	(trace of wood fragments & related organic matter in upper 7')		1	SS	2					WL in open BH Sept.3/71
			2	SS	14					
			2A	BXL	66%					
			3	SS	20	730		o		33 55 (12)
	occ. random boulders up to 22" in size throughout)		4	SS	17					
			5	SS	20					
			6	SS	11	720		o		
	Loose to Dense		7	SS	30					
			7A	BXL	44%					
714.6			8	SS	100/½"					
28.0	End of Borehole					710				

FORM OD-MT-[illegible]26 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 16

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 89 o/s 92' Rt. Ø Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE Sept. 8, 1971 COMPILED BY VK

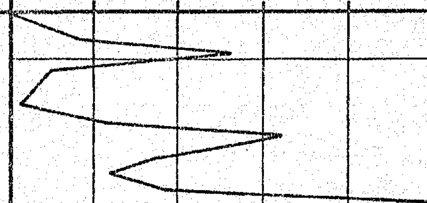
DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core; Cone CHECKED BY [illegible]

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	BLOWS / FOOT
742.5	Ground Level			
0.0	Silty sand & gravel			
		1	SS	100/6"
	occ. boulders up to	1A	BXL	48%
	24" in size througout	2	SS	43
		2A	BXL	88%
		3	SS	46
	Compact to Very Dense	3A	BXL	88%
		3B	BXL	66%
719.		4	SS	100/5"
23.5	badly fractured	4A	BXL	82%
	Bedrock	4B	BXL	100%
		5	AXT	100%
	moderately fractured			
709.0	sound	6	AXT	100%
33.5	End of Borehole			

ELEV. SCALE: 740, 730, 720, 710, 700

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT
20 40 60 80 100

SHEAR STRENGTH P.S.F.
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE



LIQUID LIMIT — W_L
PLASTIC LIMIT — W_P
WATER CONTENT — W

W_P — W — W_L

WATER CONTENT %
10 20 30

BULK DENSITY γ P.C.F.

REMARKS
GR.SA.SI.CL.

▼ 739.5
WL in open BH Sept.8/71

DETAILED BEDROCK DESCRIPTION

Elev. 719.0 to 711.0: granite gneiss (fractured and possibly loose in the upper 1.0 ft., moderately fractured in the rest of the section)

Elev. 711.0 to 709.5: quartz biotite gneiss (moderately weathered)

Elev. 709.5 to 709.0: granite gneiss (sound)

FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 17

FOUNDATION SECTION

JOB 71-11074
W.P. 401-64
DATUM Geodetic

LOCATION Sta. 343 + 85 o/s 120' Rt. Ø Line 'E'
BORING DATE Sept. 13, 1971
BOREHOLE TYPE Washboring-BX Casing-BXL Rock Core; Cone

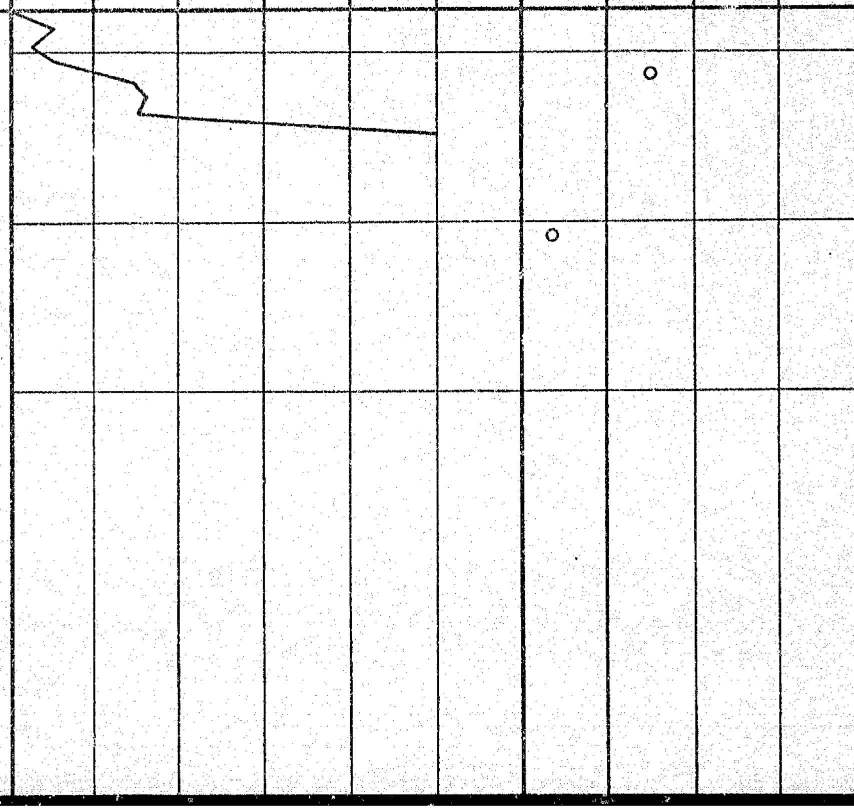
ORIGINATED BY VK
COMPILED BY VK
CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR.SA.SI.CL
742.5	Ground Level					
0.0	Silty sand & gravel occ. boulders up to 12" in size throughout				740	739.5 WL in open BH Sept.13/71
		1	SS	2		
		1A	BXL	38%		
		2	SS	100/6"		
		3	SS	56		
		4	SS	117	730	81 15 (4)
		4A	BXL	50%		
	Loose to Very Dense	5	SS	93		
724.5 / 18.0	End of Borehole				720	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE

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

BULK DENSITY γ P.C.F.



--MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 18

FOUNDATION SECTION

JOB 71-11074

W.P. 401-64

DATUM Geodetic

LOCATION Sta. 344 + 57 o/s 50' Rt. Ø Line 'E'

BORING DATE Sept. 1, 1971

BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core; Cone

ORIGINATED BY VK

COMPILED BY VK

CHECKED BY [signature]

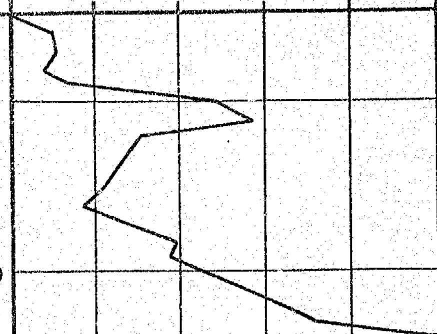
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w

WATER CONTENT % 10 20 30

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
745.0	Ground Level							
0.0	Sandy gravel to silty sand and gravel boulders up to 22" in size below el. 733. Loose to Dense	1	SS	30	740			▽ 739.5
		2	SS	9				
		3	SS	37				
		3A	BXL	100%		○		32 57 (11)
		4	SS	73	730			
		4A	BXL	100%				
722.7		5	SS	42				
22.3	Granite Gneiss Bedrock Sound	6	BXL	100%	720			
716.0		7	AXT	100%				
29.0	End of Borehole				710			



FORM OD-MT-126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 19

FOUNDATION SECTION

JOB 71-11074
W.P. 401-64
DATUM Geodetic

LOCATION Sta. 345 + 44 o/s 12' Rt. Ø Line 'E'
BORING DATE August 30, 1971
BOREHOLE TYPE Washboring-BX Casing-BXL & Axt Rock Core

ORIGINATED BY VK
COMPILED BY VK
CHECKED BY [signature]

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE)	WATER CONTENT % (LIQUID LIMIT — W_L, PLASTIC LIMIT — W_P, WATER CONTENT — W)	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
754.8	Ground Level									
0.0	Sand with some gravel (Fill)									W.L. not established
749.8	Compact		1	SS	21	750				
5.0	Silty sand & gravel boulders up to 12" in size throughout.									
743.3	Dense		2	BXL	67%					
11.5	Bedrock		3	AXT	84%	740				
734.8	Moderately fractured		4	AXT	100%					
20.0	End of Borehole					730				

DETAILED BEDROCK DESCRIPTION

Elev. 743.3 to 739.4: granite gneiss (thin veins of granite pegmatite - moderately fractured)

Elev. 739.4 to 734.8: granite gneiss (moderately fractured evidence of open fractures at depths of 15.5' and 16.5'.

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 20

FOUNDATION SECTION

JOB 71-11074
W.P. 401-64
DATUM Geodetic

LOCATION Sta. 342 + 47 o/s 38' Lt. Ø Line 'E'
BORING DATE Sept. 2, 1971
BOREHOLE TYPE Winkie Drill - AX Casing

ORIGINATED BY VK
COMPILED BY VK
CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
745.0	Ground Level									
0.0	Silty sand & gravel. Compact									▼
740.3						740				
4.7	Granite Gneiss Bedrock		1	AXT	100%					
	Moderately fractured		2	AXT	100%					
734.3										
10.7	End of Borehole									
						730				

FORM OD-M■126 (REV. 1969) OFFICE REPORT ■ SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 21

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 51 o/s 34' Lt. Ø Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE Sept. 15, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core; Cone CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR. SA. SI. CL.
743.7	Ground Level					
0.0	Silty sand & gravel to sandy gravel occ. boulders up to 9" thick throughout.	1	SS	37	740	▼739.5
		1A	BXL	17%		41 37 21 1
		1B	BXL	17%		
		2	SS	21		
730.7	Compact to Dense	2A	BXL	33%		
13.0	badly fractured	2B	BXL	50%	730	
	Bedrock	3	BXL	100%		
723.7	moderately fractured	4	AXT	100%		
20.0	End of Borehole				720	

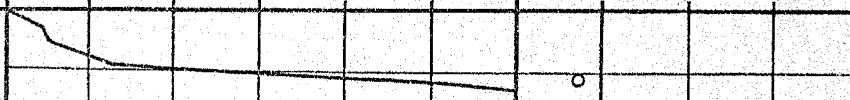
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w

WATER CONTENT %

BULK DENSITY γ P.C.F.



DETAILED BEDROCK DESCRIPTION

Elev. 730.7 to 723.7: granite gneiss (possibly fractured and loose in the upper 1.5 ft., moderately fractured throughout).

FORM OD-MT 126 (REV. 1969) OF CE REPORT SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 22

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 66 o/s 20' Lt. Ø Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE Sept. 28, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL Rock Core CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ⊕ QUICK TRIAXIAL, × LAB. VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W — WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
739.5	Water Level									
0.0	Water									
732.5 / 7.0	Sand & gravel (boulders up to 8" size) Dense		1	BXL	80%					
729.6 / 9.9	badly fractured					730				
	Bedrock		2	BXL	100%					
722.5	Sound		3	BXL	100%					
17.0	End of Borehole					720				

DETAILED BEDROCK DESCRIPTION

Elev. 729.6 to 728.3: granite gneiss (probably loose slabs lying on the bedrock.)

Elev. 728.3 to 722.5: granite gneiss (minor fractures)

FORM OD-MT ●26 (REV. 1969) OFFICE REPORT ● SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 24

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 66 o/s 51' Rt. Ø Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE Sept. 20, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing -BXL & AXT Rock Core CHECKED BY

SOIL PROFILE ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE)	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
739.5	Water Level									
	Water									
729.5						730				
10.0	Silty sand & gravel to sandy gravel boulders up to 12" in size throughout.			BXL	66%					
			1	SS	100/6"					
			1A	BXL	23%	720				
717.0	Very Dense		2	SS	107			o		25 67 (8)
22.5 / 713.0	Quartz biotite gneiss Bedrock) fine grained) Sound		3	AXT	100%					
26.5	End of Borehole					710				

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 25

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 65 o/s 20' Rt. ℄ Line 'E' ORIGINATED BY VK
W.P. 401-64 BORING DATE Sept. 29, 1971 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core CHECKED BY [signature]

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	WATER CONTENT %	REMARKS GR.SA.SI.CL.
739.5	Water Level						
0.0	Water						
730.5 / 9.0	Silty sand & gravel to sandy gravel boulders up to 19" in size throughout. Loose to Very Dense				730		
		1A	BXL	95%			
		1	SS	7		o	54 43 (3)
		2	SS	174/9"			
721.7		3	SS	100/3"			
	Loose Bedrock Slabs	4	BXL	100%			
18.5	Bedrock moderately fractured	5	AXT	100%	720		
715.5 / 24.0	End of Borehole				710		

DETAILED BEDROCK DESCRIPTION

Elev. 721.7 to 721.0: granite gneiss (probably loose slabs lying on bedrock)

Elev. 721.0 to 715.5: granite gneiss (lineation approx. 50°, prominent near-vertical fractures at depths of 19. to 19.5' and 22.5 to 23.5').

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 23

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 343 + 57 o/s 20' Rt. Ø Line 'E; ORIGINATED BY VK

W.P. 401-64 BORING DATE Sept. 23, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & Axt Rock Core CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ⊗ QUICK TRIAXIAL, × LAB. VANE)	LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT % (10, 20, 30)	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
739.5	Water Level									
0.0 736.0	Water									
3.5	Silty sand & gravel to sandy gravel boulders up to 10" in size throughout.		1	BXL	26%					
			1A	SS	82	730		o		
			1B	BXL	16%					
			2	SS	100/3"					
			2A	BXL	84%					
			2B	BXL	50%					
721.0	Very Dense		3	SS	00					
18.5	Bedrock		4	BXL	100%	720				
	~~Sand Seam~~									
711.6	Sound		5	AXT	100%					
27.9	End of Borehole					710				

DETAILED BEDROCK DESCRIPTION

Elev. 721.0 to 715.5: bonded granite gneiss (lineation approx. 50° - sound)

Elev. 715.5 to 715.3: medium and fine sand seam.

Elev. 715.3 to 711.6: granite gneiss (with minor quartz biotite gneiss at a depth of 27 ft. open fracture sub-parallel to the lineation at 24.3 ft.)

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 26

FOUNDATION SECTION

JOB 71-11074 LOCATION Sta. 344 + 47 o/s 16' Rt. Ø Line 'E' ORIGINATED BY VK

W.P. 401-64 BORING DATE Sept. 29, 1971 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing-BXL & AXT Rock Core; Cone CHECKED BY [signature]

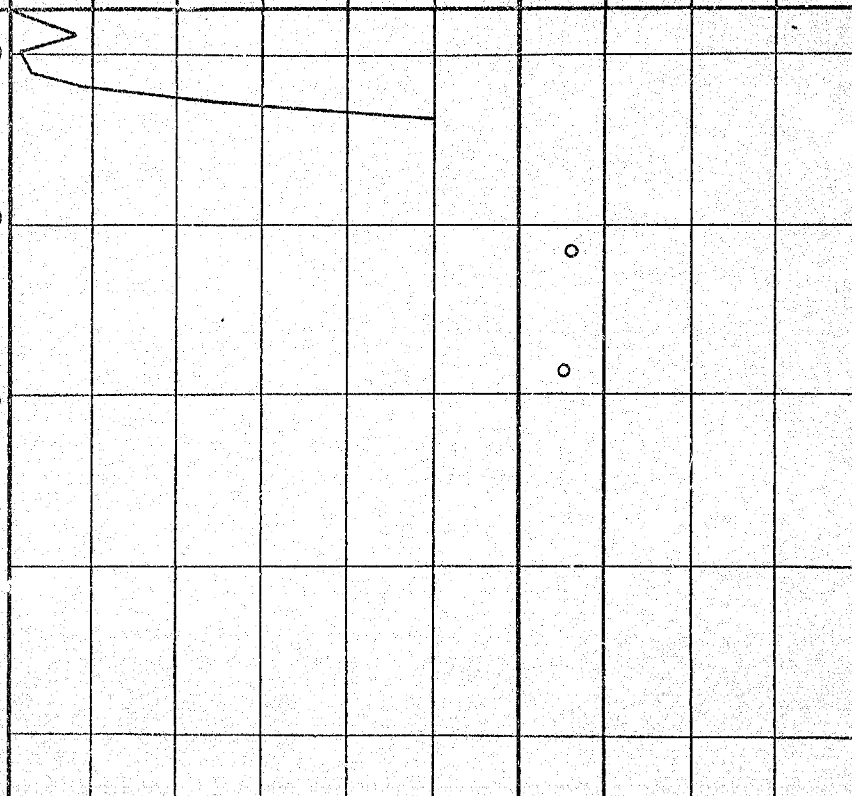
SOIL PROFILE: ELEV. DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS / FOOT	REMARKS: GR. SA. SI. CL.
752.5	Ground Level					
0.0	Silty sand and gravel to sandy gravel		1	SS	100/4"	
	(boulders up to 14" size below el. 740.)		2	SS	26	
			2A	BXL	22%	
			3	SS	38	65 31 (4)
			4	SS	45	
	Compact to Very Dense		5	SS	81	
			6	AXT	77%	
			7	AXT	33%	
712.0			8	AXT	40%	
710.5	Bedrock		9	AXT	100%	
42.0	End of Borehole					

ELEV. SCALE: 750, 740, 730, 720, 710

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT
20 40 60 80 100
SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

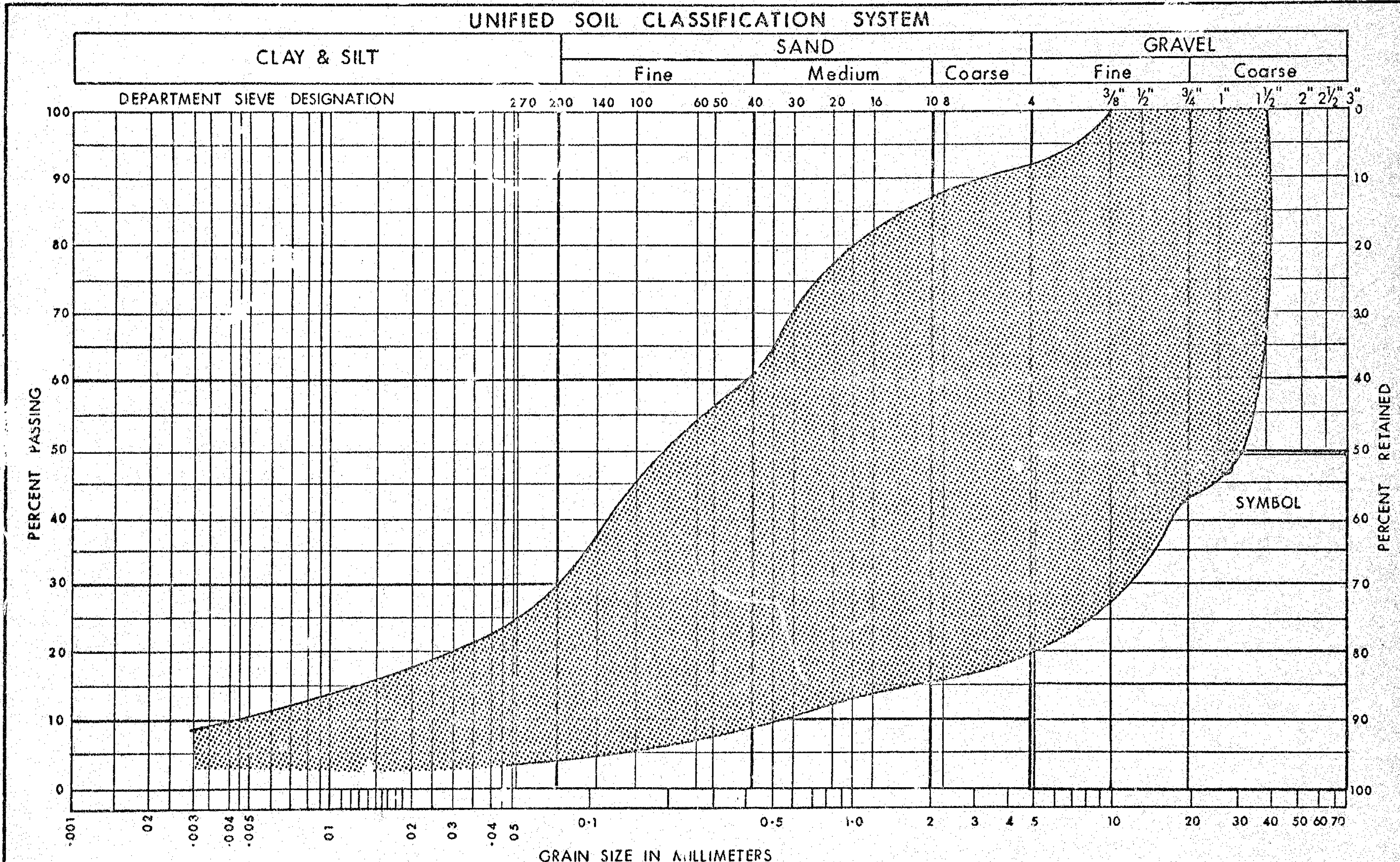
LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT ——
w_P w
WATER CONTENT %
10 20 30

BULK DENSITY γ P.C.F.



DETAILED BEDROCK DESCRIPTION

FORM OD-MT-1?
REV. AUG. 1971



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

ONTARIO

DESIGN SERVICES BRANCH

GRAIN SIZE DISTRIBUTION

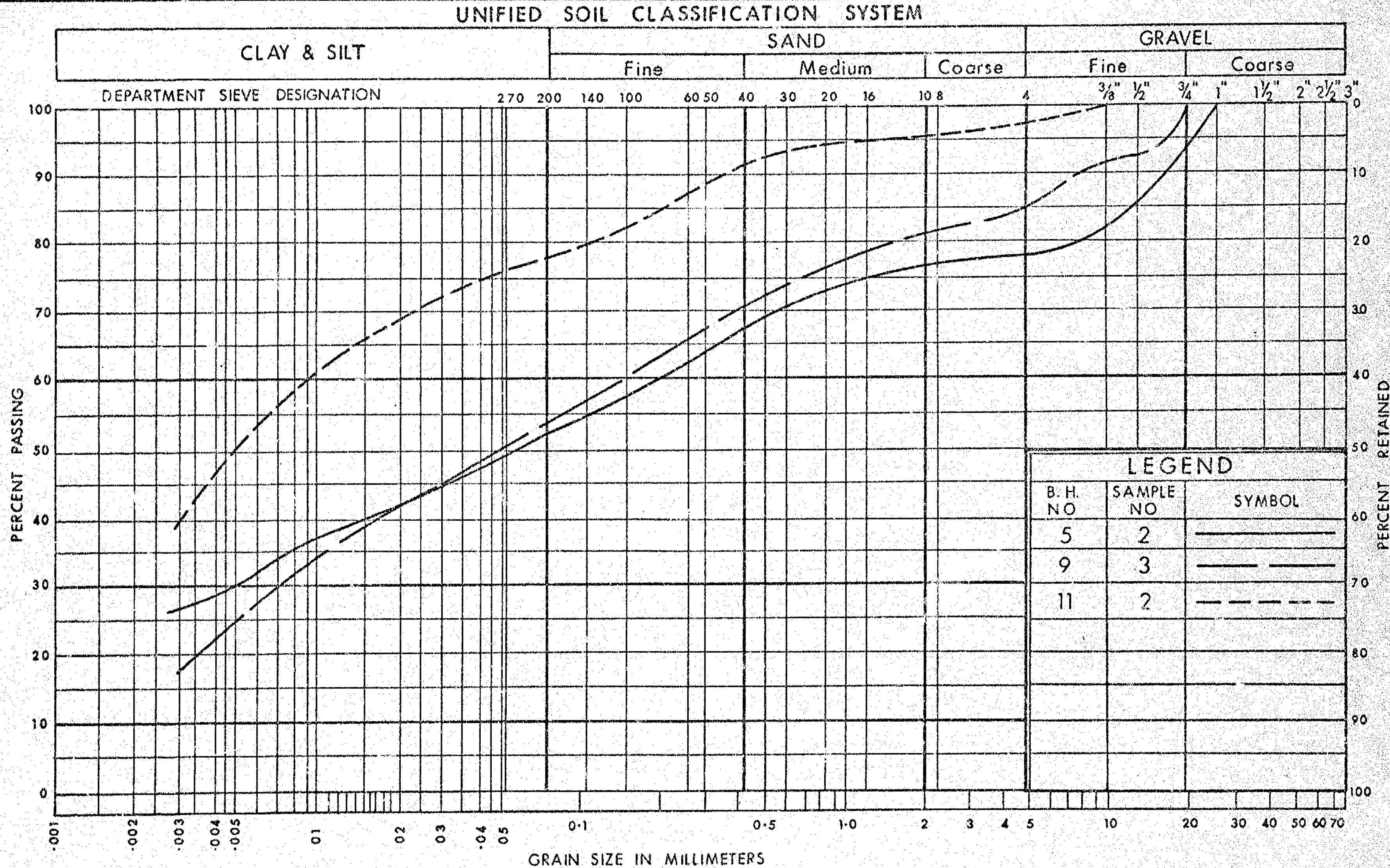
SILTY SAND & GRAVEL TO SANDY GRAVEL

W.P. No. 401-64

JOB No. 71-11074

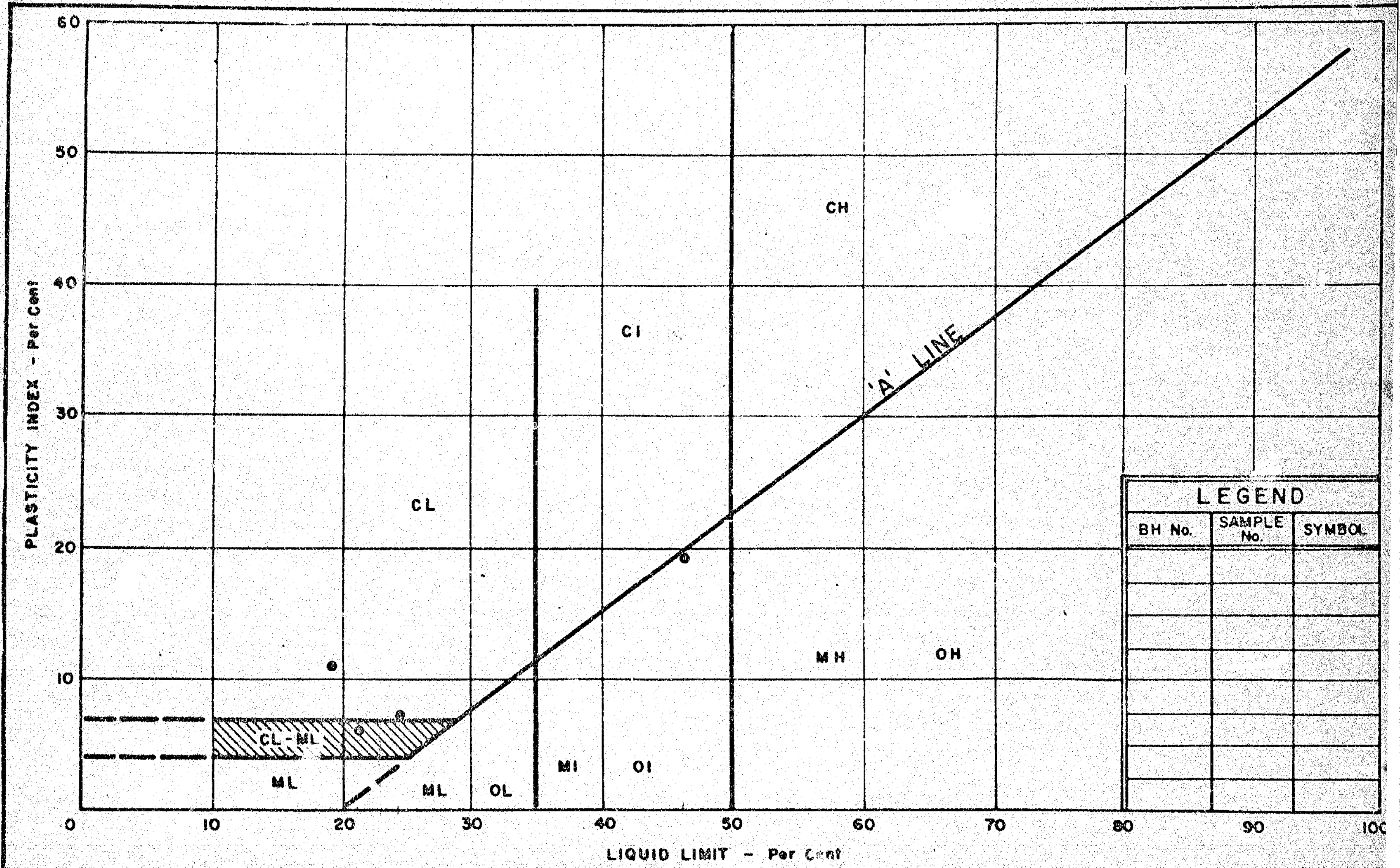
FIG. 1

FORM OD-MT-17
REV. AUG. 1971



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS — DESIGN SERVICES BRANCH — ONTARIO	GRAIN SIZE DISTRIBUTION CLAYEY SILT SOME SAND & GRAVEL	W.P. No. 401-64 JOB No. 71-11074 FIG. 2

FORM OB-MT-252
REVISED JUL 1966



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

PLASTICITY CHART

CLAYEY SILT

SOME SAND & GRAVEL

W.P. No.	401 - 64
JOB No.	71 - 11074
FIG. 3	

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_P	PLASTIC LIMIT
I_P	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma'}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
	(c', ϕ':) IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
	(c_u, ϕ_u:) IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	$= 3{\cdot}1416$
e	BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ε	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	'N' BLOWS / FT.	c LB. / SQ. FT.	DENSENESS	'N' BLOWS / FT.
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

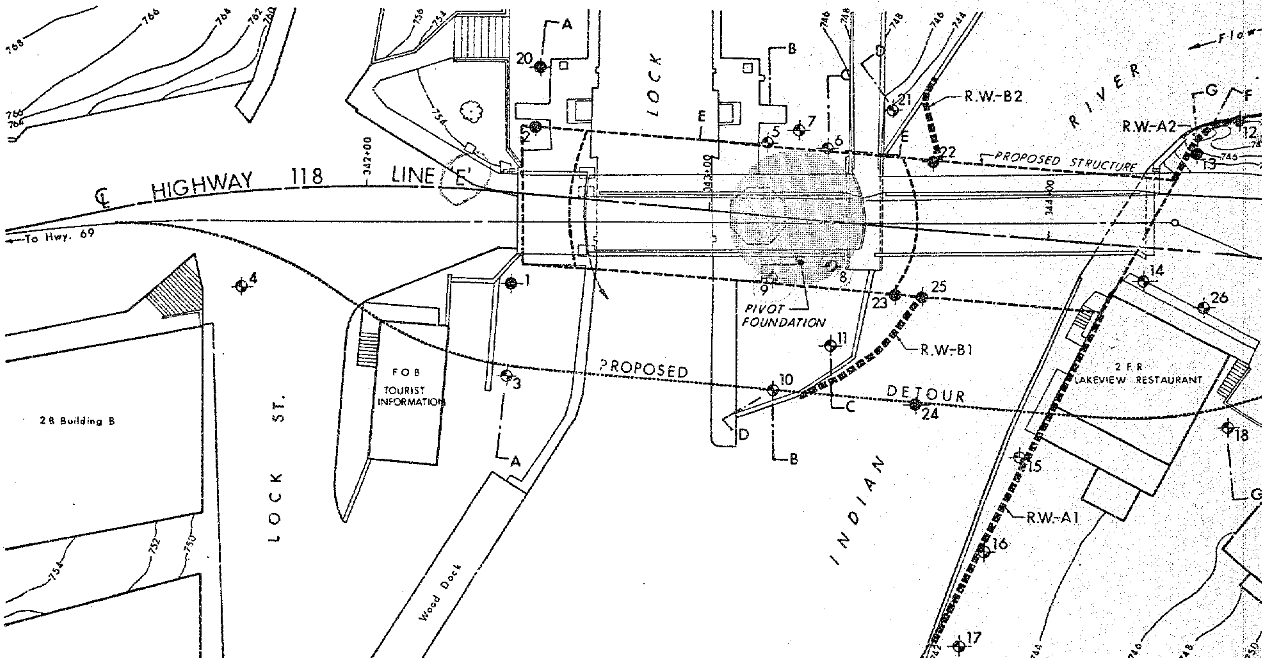
S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		

P.H. SAMPLE ADVANCED HYDRAULICALLY

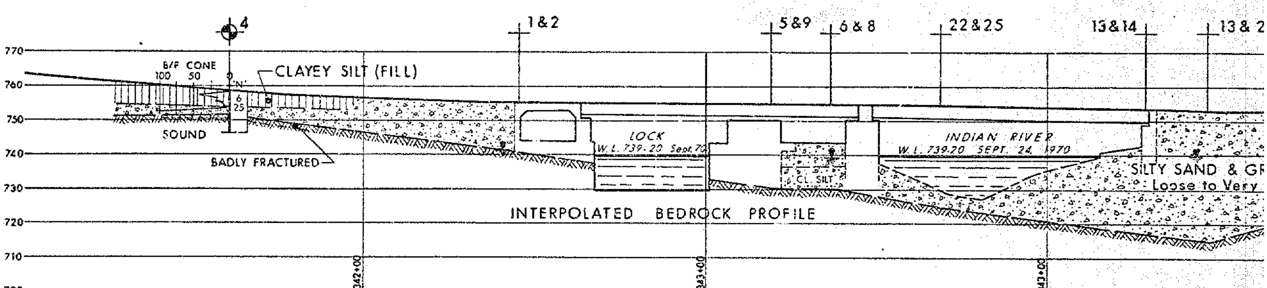
P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

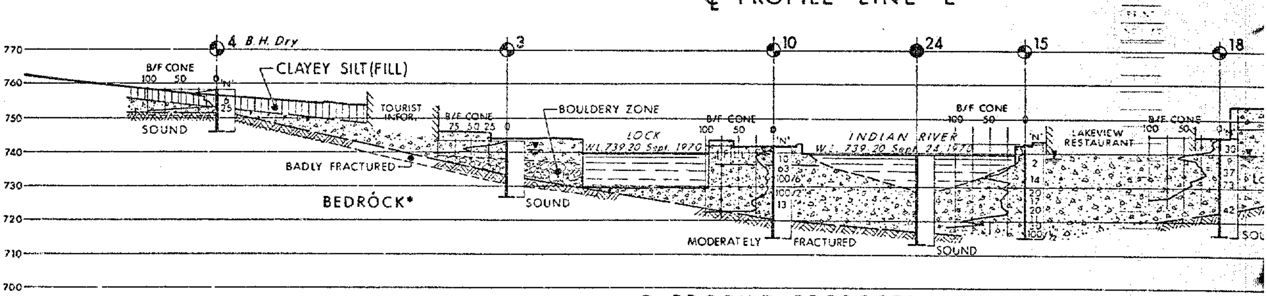
Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY



PLAN
SCALE
20 10 0 20 40FT.



℄ PROFILE LINE 'E'

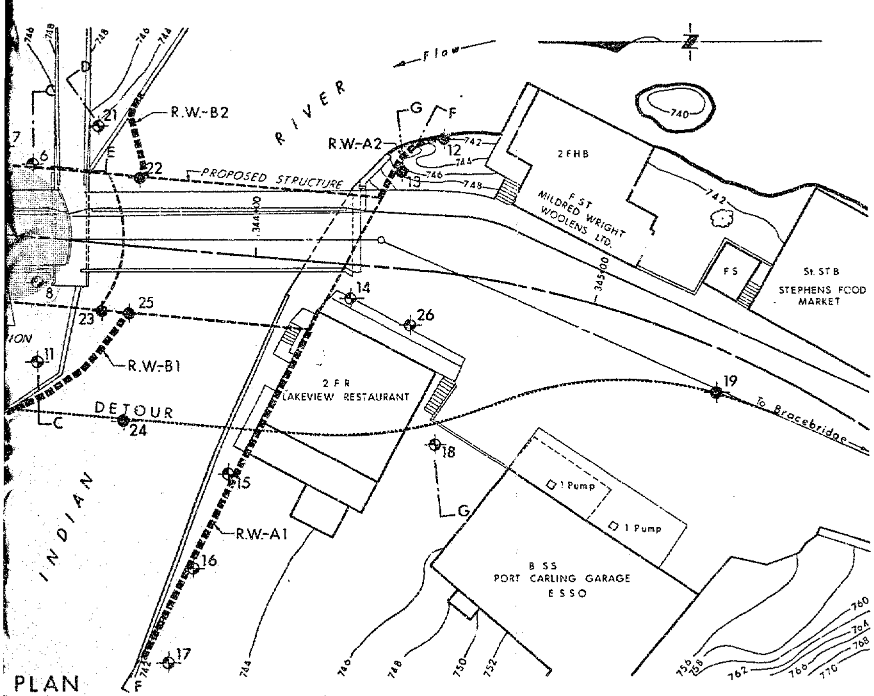


℄ PROFILE PROPOSED DETOUR
SCALE
20 10 0 20 40FT.

611950E
4997050N

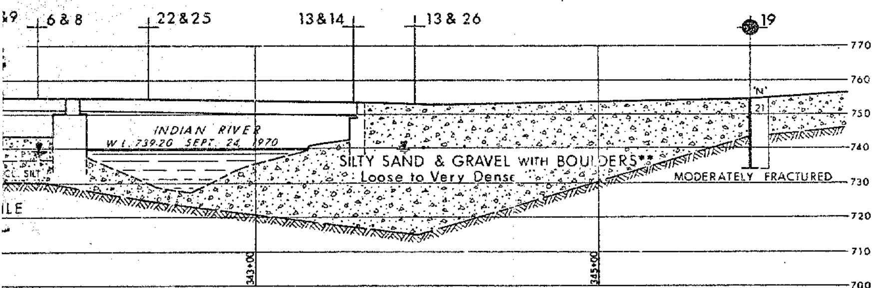
17

31E4E

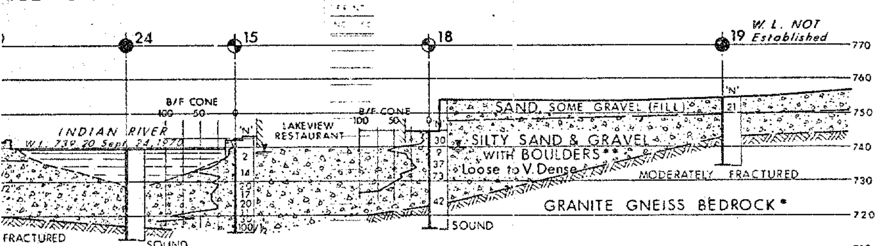


PLAN

SCALE 0 20 40FT.



PROFILE LINE 'E'

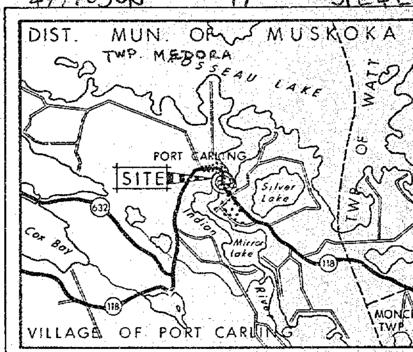


** FOR DETAILS SUCH AS LOCATION & SIZE OF BOULDERS SEE BOREHOLE LOG SHEETS

PROPOSED DETOUR

SCALE 0 20 40FT.

* FOR DETAILED BEDROCK DESCRIPTION BOREHOLE LOG SHEETS



KEY PLAN

SCALE IN MILES 1 .5 0 1 2 Miles

LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation. Aug. & Sept. 1971

NO.	ELEVATION	STATION	OFFSET
1	744.5	342+44	26' RT.
2	745.0	342+48	20' LT.
3	743.7	342+47	53' RT.
4	758.4	341+63	28' RT.
5	744.8	343+15	21' LT.
6	743.9	343+33	21' LT.
7	744.4	343+24	26' LT.
8	743.1	343+37	13' RT.
9	743.0	343+20	18' RT.
10	741.7	343+24	50' RT.
11	741.5	343+39	36' RT.
12	741.3	344+51	40' LT.
13	749.2	344+39	28' LT.
14	752.0	344+28	10' RT.
15	742.6	343+97	64' RT.
16	742.5	343+89	92' RT.
17	742.5	343+85	120' RT.
18	744.9	344+57	50' RT.
19	754.8	345+44	12' RT.
20	745.0	342+47	38' LT.
21	743.7	343+51	34' LT.
22	739.5	343+66	20' LT.
23	739.5	343+57	20' RT.
24	739.5	343+66	51' RT.
25	739.5	343+65	20' RT.
26	752.5	344+47	16' RT.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH — FOUNDATION SECTION

INDIAN RIVER
(PORT CARLING)

HIGHWAY NO. 118 LINE 'E' DIST. NO. 11
Dist. Mun. of MUSKOKA Village of PORT CARLING
Formerly Twp. of MEDORA LOT 31 CON. IV

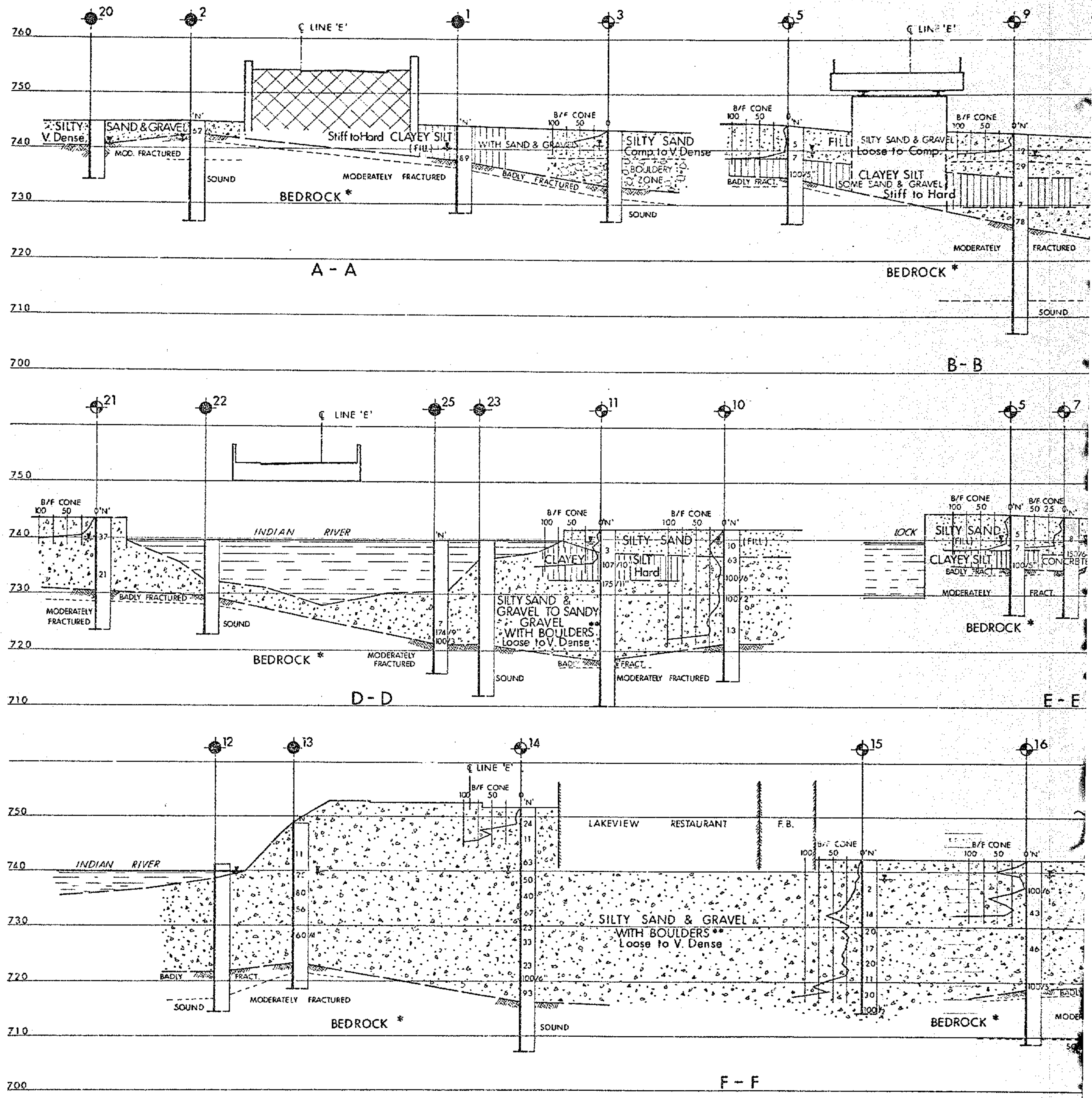
BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. V. K.	CHECKED	W.P. NO. 401-64
DRAWN	CHECKED	JOB NO. 71-11074
DATE Oct. 27, 1971	SITE NO.	
APPROVED PRINCIPAL FOUNDATION ENGINEER	CONT. NO.	

DRAWING NO. 71-11074A

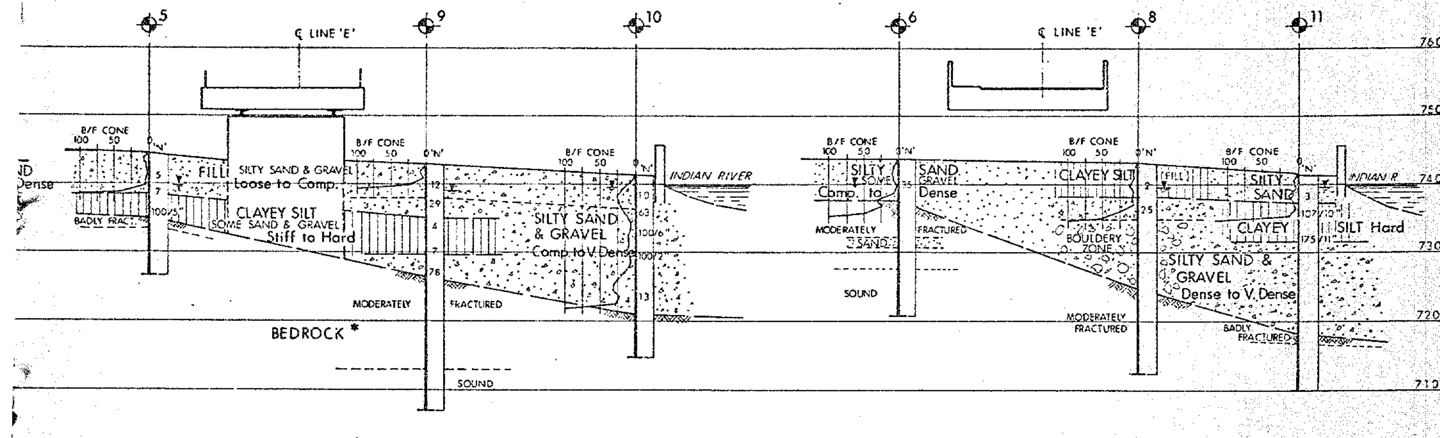
BRIDGE DRAWING NO.

REF. NO. E-5018-1



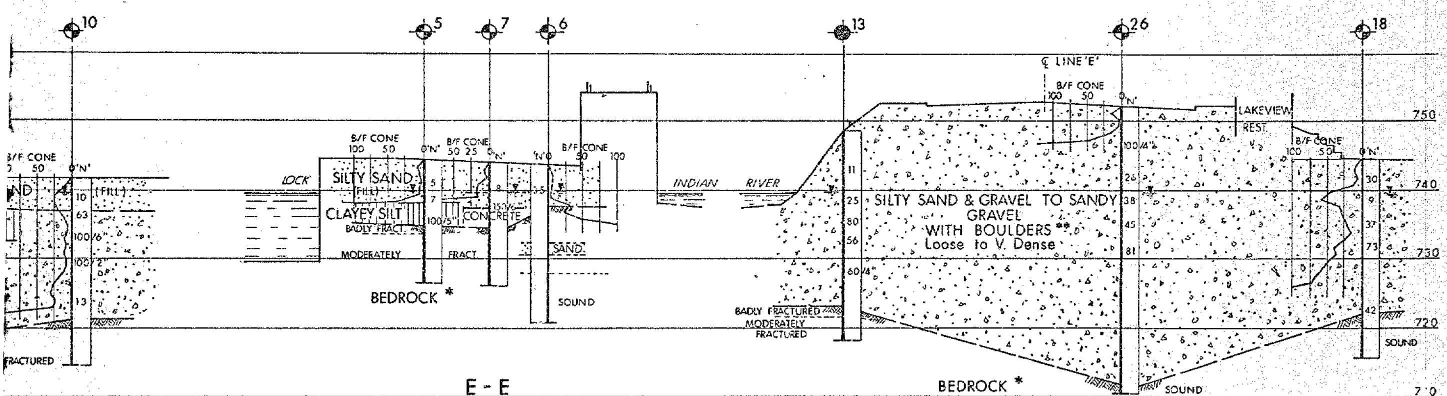
SECTIONS

10 5 0 SCALE 10 20 FT.



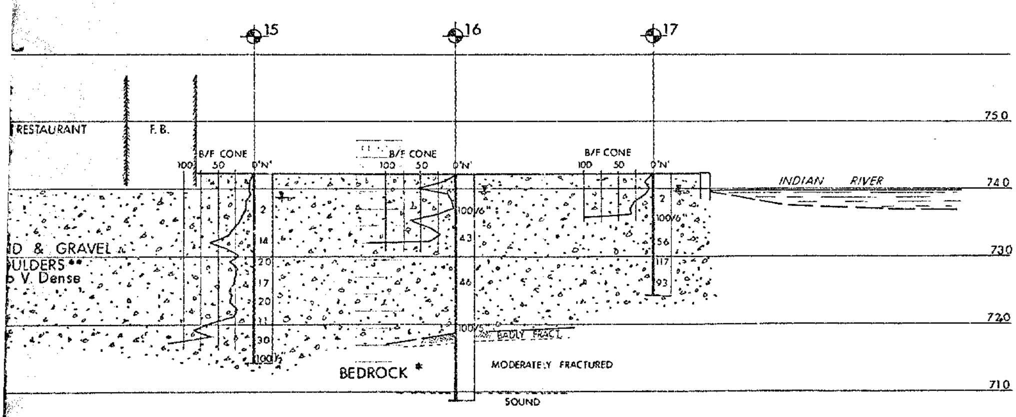
B - B

C - C



E - E

G - G



F - F

— NOTE —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS		
DATE	BY	DESCRIPTION

** FOR DETAILS SUCH AS LOCATION & SIZE OF BOULDERS SEE BOREHOLE LOG SHEETS

* FOR DETAILED BEDROCK DESCRIPTION SEE BOREHOLE LOG SHEETS

SECTIONS

5 0 SCALE 10 20 FT.

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH - FOUNDATION OFFICE

INDIAN RIVER
(PORT CARLING)

HIGHWAY NO. 118 LINE 'E' DIST. NO. 11
Dist. Mun. of MUSKOKA Village of PORT CARLING
Formely Twp. of MEDORA LOT 31 CON IV

SECTIONS & SOIL STRATA

SUBMD. V.K.	CHECKED	W.P. NO. 401-64	DRAWING NO. 71 - 11074 B
DRAWN	CHECKED	JOB NO. 71-11074	
DATE 27 OCT. 1971		SITE NO.	BRIDGE DRAWING NO.
APPROVED	PRINCIPAL FOUNDATION ENGINEER	CONT. NO.	

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Downsview.

From: Bridge Planning,
North Bay.

Attention:

Date: 15 July 1971

Our File Ref.

In reply to

Subject:

Re: W. P. 401-64 Site 42-1
Indian River @ Port Carling
Hwy. # 118 District #11

70-11-074

Attached are two prints of site plan E-5018-1 showing the general layout of the proposed structure and retaining walls at the above crossing.

It is proposed to replace the existing swing and fixed span with a similar facility which will be two lanes and designed to modern highway loadings. Because of the wider roadway section, the location and size of the swing pivot will be different from the existing. I have sent for drawings of the existing structure which I believe is founded on bedrock. A copy of these drawings will be forwarded to you as soon as they are available.

It will be necessary to drill on D. P. W. property and permission of the lock master should be obtained prior to starting work. I have been in touch with the Property Section, North Bay, and have forwarded them a plan indicating the approximate locations where you will be drilling. They have agreed to approach the two properties on the east side of the crossing and obtain permission to enter. I will inform you as soon as this has been obtained.

REGISTERED PROFESSIONAL ENGINEER
B. T. DARCH
PROVINCE OF ONTARIO

JCMcA/bn
attach.

J. C. McAllister,
Regional Bridge
Planning Supervisor.

COMPLETION DATE
SEP 29.1971

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

To: Mr. M. Devata,
Supervising Foundation Engineer.

From: K. Ingham

Attention:

Date: October 25, 1971

Our File Ref.

In reply to

Subject: Foundation Investigation - Indian River Bridge at Port Carling

The dominant type of rock encountered in the drilling and exposed in the immediate vicinity of the site is a medium grained pink granite gneiss displaying a pronounced lineation dipping steeply at approximately 50°. Bands of dark grey quartz biotite gneiss with less conspicuous lineation are a subordinate type but form layers parallel to the granite gneiss. Veins of granite pegmatite are common cutting across the gneissic structure. These were the only types intersected in the bedrock and most of the boulders overlying the bedrock appear to be the same.

Fractures are common and apparently form a fairly close pattern. Evidence of four sets of fractures is present in the cores, one sub-parallel to the lineation, a horizontal set, a vertical set and less commonly a steeply inclined set. These joints are generally tight except where noted. In some instances, open joints are sand-filled near the surface. The bedrock contour map derived from rock elevations in 26 boreholes gives a more or less straightforward knob-and-valley topography. The lowest elevations follow a course underlying the restaurant which may be an earlier channel of the Indian River, however, there is insufficient evidence to establish this. The rock exposed north of the service station drops by a series of steps down to this channel. An area of rock of lower general relief exists between here and the present lock excavation and south of this the rock rises to an area of higher relief.

The only unusual rock conditions were noted in the vicinity of hole No. 6 and in the present Indian River channel. The latter case is relatively minor, where loose slabs of rock 1.0 to 1.5 ft. in thickness appear to lie directly on the bedrock. Near hole No. 6 the rock appears to drop sharply towards the river, coupled with several open fractures observed in hole 6, there may be an unstable area of rock in the immediate vicinity. Excavation of the rock surface would be required to confirm this possibility.

A brief description of each rock core together with the appropriate bedrock elevation is given below:

(Cont'd....2)

B. H. No. 1 Bedrock at 738.3

6.2 to 7.6: fine grained quartz biotite gneiss, badly broken and possibly loose in this section.

7.6 to 11.0: fine grained quartz biotite gneiss, moderately fractured, fractures sub-parallel to the lineation, horizontal and steeply inclined.

11.0 to 16.0: medium to coarse grained granite gneiss, prominent lineation dipping at approx. 50°, moderately fractured as above.

B. H. No. 2 Bedrock at 742.7

2.3 to 9.3: medium grained granite gneiss, inclined lineation, badly fractured possibly loose in the upper 0.8 ft., inclined and vertical fractures, vertical fractures probably open down to 9.3 ft.

9.3 to 18.3: medium grained granite gneiss.

B. H. No. 3 Bedrock at 733.7

10.0 to 16.0: fine grained quartz biotite gneiss, inclined lineation, upper 2.0 ft. weathered, conspicuous inclined fractures.

B. H. No. 4 Bedrock at 751.7

6.7 to 12.0: medium grained granite gneiss with minor sections of granite pegmatite and quartz biotite gneiss, badly fractured in the upper 0.8 ft.

B. H. No. 5 Bedrock at 735.3

9.5 to 10.2: granite pegmatite, weathered.

10.2 to 18.0: granite gneiss, inclined lineation, weathered in the upper 0.8 ft., moderately fractured.

B. H. No. 6 Bedrock at 737.9

5.8 to 6.0: Boulder

6.0 to 11.8: granite gneiss, lineation inclined approx. 50°, evidence of open fractures in this section - high angle or vertical fractures at 7.3, 10.6, 11.3 and 11.8, parallel to the lineation at 8.5 and 9.3.

11.8 to 13.3: medium and fine sand plus some wood, inclined fracture surface at 13.3.

(Cont'd...3)

B. H. No. 6 (Continued) Bedrock at 737.9

13.3 to 14.0: granite gneiss, possible open inclined fractures at 13.6 and 14.0.

14.0 to 15.0: granite gneiss.

15.0 to 16.0: quartz biotite gneiss.

16.0 to 23.3: granite gneiss, inclined fracture at 19.0.

B. H. No. 7 Bedrock at 734.8

6.5 to 9.6: concrete.

9.6 to 18.0: granite gneiss, 0.2 ft. granite pegmatite vein at 10.4, moderately fractured.

B. H. No. 8 Bedrock at 724.5

10.4 to 18.5: boulders.

18.5 to 31.0: granite gneiss - generally medium grained, coarse grained 25.0 to 28.0, moderately fractured - prominent near-vertical fracture 21.0 to 22.5.

B. H. No. 9 Bedrock at 726.5

16.5 to 28.5: granite gneiss, inclined lineation approx. 50°, minor horizontal and inclined fractures.

28.5 to 30.0: granite pegmatite.

30.0 to 36.0: granite gneiss.

B. H. No. 10 Bedrock at 720.7

5.0 to 21.0: granite gneiss, boulders.

21.0 to 27.0: guartz biotite gneiss, minor near vertical fractures.

B. H. No. 11 Bedrock at 718.0

11.5 to 23.5: boulders plus loose rock.

23.5 to 24.5: granite gneiss, evidence of horizontal and inclined open fractures in this section.

24.5 to 31.5: quartz biotite gneiss, frequenty closed near-vertical fractures.

(Cont'd....4)

B. H. No. 12 Bedrock at 722.3

9.0 to 19.0: boulders plus loose rock.

19.0 to 25.0: granite gneiss, lineation variable in this section.

25.0 to 27.0: quartz biotite gneiss.

B. H. No. 13 Bedrock at 723.7

25.5 to 27.0: granite gneiss, several open fractures, possibly loose in this section.

27.0 to 30.5: granite gneiss, moderately fractured.

B. H. No. 14 Bedrock at 716.5

35.5 to 38.3: granite gneiss.

38.3 to 38.8: quartz biotite gneiss.

38.8 to 44.8: granite gneiss, minor fractures sub-parallel to the lineation.

B. H. No. 15

No bedrock recognized down to 28.0.

B. H. No. 16 Bedrock at 719.0

23.5 to 31.5: granite gneiss, fractured and possibly loose in the upper 1.0 ft., moderately fractured in the rest of the section.

31.5 to 33.0: quartz biotite gneiss, moderately weathered.

33.0 to 33.5: granite gneiss.

B. H. No. 17

No bedrock encountered down to 18.0.

B. H. No. 18 Bedrock at 722.7

22.3 to 29.0: granite gneiss.

B. H. No. 19 Bedrock at 743.3

9.3 to 11.5: boulders.

11.5 to 15.4: granite gneiss with thin veins of granite pegmatite.

(Cont'd....5)

B. H. No. 19 (Continued) Bedrock at 743.3

15.4 to 20.0: granite gneiss, moderately fractured, evidence of open fractures at 15.5 and 16.5.

B. H. No. 20 Bedrock at 740.3

4.7 to 10.7: granite gneiss, moderately fractured.

B. H. No. 21 Bedrock at 730.7

13.0 to 20.0: granite gneiss, possibly fractured and loose in the upper 1.5 ft., moderately fractured throughout.

B. H. No. 22 Bedrock at 728.3

9.8 to 11.2: granite gneiss, probably loose slabs lying on the bedrock.

11.2 to 17.0: granite gneiss, minor fractures.

B. H. No. 23 Bedrock at 721.0

18.5 to 24.0: banded granite gneiss, lineation approx. 50°.

24.3 to 24.3: medium and fine sand.

24.3 to 27.8: granite gneiss with minor quartz biotite gneiss at 27.0 f open fracture sub-parallel to the lineation at 24.3.

B. H. No. 24 Bedrock at 717.0

22.5 to 26.5: quartz biotite gneiss, fine grained, no conspicuous lineation.

B. H. No. 25 Bedrock at 721.0

17.8 to 18.5: granite gneiss, probably loose slabs lying on bedrock.

18.5 to 24.0: granite gneiss, lineation approx. 50°, prominent near vertical fractures 19.0 to 19.5 and 22.5 to 23.5.

B. H. No. 26 Bedrock at 712.0

40.5 to 42.0: granite gneiss, lineation approx. 50°.

KI;nr

K. Ingham,
Geologist.

71-11074

foundation contractors
and engineers

WESTERN CAISSONS (1969) LIMITED

A DIVISION OF AGRA INDUSTRIES LIMITED

150 CREDITSTONE ROAD
MAPLE (TORONTO) ONTARIO

November 16th,1971.

E-71-215

Department of Transportation
and Communication,
Design Services Branch,
Foundation Office,
Room 107,Central Building,
Keele Street & Highway 401,
Downsview 464,Ontario.

ATTENTION:Mr. M.Devata, P.Eng.

RE: Proposed Caissons Bridge,
Village of Port Carling,Ontario

Dear Sir;

Further to our discussion of November 10th,1971, we have reviewed the foundation site investigations logs and it is our feeling that serious consideration should be given to the use of socketed caissons for supporting the swing bridge,pier and abutment on the above site.We would suggest that in view of the weathered and badly fractured rock for a depth of from 1 1/2 to 2 feet below the surface that consideration be given to socketing the caissons a depth of 5 feet below the surface of the bedrock.This would also eliminate any future problems with acour together with providing anchorage for the swing base.

We would estimate that if the caissons are installed to this depth in the bedrock, then the following capacities could be considered for the following caisson diameters.

36" - 1,050 KIPS per caisson

48" - 1,880 KIPS per caisson

54" - 1,380 KIPS per caisson

see page two- - - -

OFFICES IN: CALGARY EDMONTON SASKATOON WINNIPEG TORONTO MONTREAL DETROIT
TELEPHONE: (403)-272-5531 (403)-434-8796 (306)-274-6121 (204)-586-8295 (416)-889-7383 (514)-669-3581 (313) 961-8380
TELEX: 038-22562 037-2428 034-2360 07-58735 02-29212 012-0959

We would recommend that the half inch wall casing which we would use for churn drilling in the caissons be left in place and be structurally considered in designing the caissons.

Our estimate for the following caisson sizes per lineal foot considering that the Department would provide cement for the concrete and reinforcing steel, fabricated ready to be placed, into the caisson holes as follows:-

36" diameter	\$ 180.00 per lineal foot of drilling
42" diameter	\$ 190.00 per lineal foot of drilling
48" diameter	\$ 210.00 per lineal foot of drilling
54" diameter	\$ 225.00 per lineal foot of drilling
Mobilization estimate	\$8000.00

The above estimates are based on the assumption that access, including barges would be provided for the drilling equipment where required. It is also understood that if the caisson holes are not dry that provision would be made to allow the Sub-Contractor to tremie the concrete in place.

Due to wage increases which would take place if the project goes ahead next year and the above prices have been based on current wage rates, there could possibly be an increase of up to ten percent on the above prices if this work is performed next year.

Trusting the above information is satisfactory and should additional information be required, please feel free to contact us.

Yours very truly,

WESTERN CAISSONS (1969) LIMITED

P. Kozicki

P.KOZICKI,P.ENG.

PK:sr

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. J. McAllister,
Reg. Structural Planning Engineer,
NORTHERN REGION, North Bay.

FROM: Structural Office,
West Building, DOWNSVIEW.

ATTENTION:

DATE: December 11th, 1972

OUR FILE REF.

IN REPLY TO

SUBJECT:

Indian River Bridge,
W.P.#401-64-04, Site #42-1,
Hwy. #118, District #11.

71-11-074

Attached herewith are prints of the Preliminary Bridge Plan Drawing D-42-1-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is $750,000 which includes tender, materials, engineering and sundry construction.

Any comments or revisions you may have should be submitted within three weeks.

C. S. Grebski,
Structural Design Engi. .er.

CSG:dp
Attach.

cc. W. D. Birch,
A. E. McKim,
B. R. Davis,
A. Stermac,
Foundation Office,
J. Anderson,
R. Murphy.

RECEIVED
FOUNDATIONS OFFICE
DEC 12 1972
Ministry of Transportation & Communications

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO:
A. Stermac,
Principal Foundation Engineer.
Room 107, West Building.

FROM:
Structural Office,
West Building,
Downsview.

ATTENTION:

DATE:
May 3rd, 1973.

OUR FILE REF.

IN REPLY TO

SUBJECT:

SACH

Indian River Bridge,
W.P.#401-64-04, Site #42-1,
Hwy. #118, District #11.

71-11-074

Attached herewith we are submitting the final bridge drawings which show the foundation design for this structure.

Kindly give us your comments at your earliest convenience.

C.S. Grebski,
Structural Design Engineer.

CSG:dp
Attach.

cc. Foundation Office.

A positive dewatering scheme will be necessary for the construction of foundations.

M. Devata
May 14/73.

Dwg. 71-11074A Finalized
sent to Structural Office
31 May 73

Mr. Davis
Mr. Ruttka.

71-11074

Mr. L. R. Eadie,
Executive Director,
Operations Division.

J. W. MacDougall,
Claims Engineer.

March 6, 1974.

Re: Claim on Contract 73-125
O. J. Gaffney Limited
Huntsville District

Attached please find for your information, copy of Notification of Intent to Claim dated February 28, 1974 from O. J. Gaffney Limited regarding the above contract.

ORIGINAL SIGNED
BY
J. W. MacDOUGALL

J. W. MacDougall,
Claims Engineer.

JWM:dk
Attach.
c.c. - J. B. Wilkes
C. R. Wilmot ✓
A. McConnell
A. C. Lennox
J. M. Crannie
R. S. Chapman

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. M. Devata,
Supervising Foundation Engineer,
Soils Mechanics Section,
ATTENTION: Geotechnical Office, West Bldg.

FROM: Geologist,
Geotechnical Office,
West Building.

DATE: March 13, 1974.

OUR FILE REF.

IN REPLY TO

SUBJECT:

New Structure - Hwy. #118
Indian River, Port Carling, Ontario
W.P. 401-64 W.O. 71-11074

There appears to be a misunderstanding by the contractor on the information given by certain rock core logs in the immediate vicinity of the pivot foundation area.

The logs of holes 5 - 9 inclusive, indicate that the bedrock has fractures both vertically and inclined, and also some fractures parallel to the lineation of the rock structure. The covering report by Mr. K. Ingham states that these fractures appear to be a common structural occurrence of the rock in this area and that the four sets of fractures are generally tight.

The contractor has interpreted this logging and summary information for the rock in the pivot foundation area as being fractured rock; that is, broken rock in fragments that could be moved or taken out with a mobile mechanical shovel or loader.

The visit to the construction site on February 28, 1974 and subsequent inspection of the area excavated for the pivot foundation shows a homogeneous, massive, gneissic rock of structural characteristics as described by Mr. K. Ingham in his report. The fractures in the rock as mentioned in the foundation report are a set of rock structural joints or fractures. Nowhere does Mr. Ingham mention loose, broken sections of rock in his report.

I believe that had the contractor read the information supplied and interpreted it correctly, his findings would have been solid rock with structural fractures or joints as an inherent characteristic.

B.K. Glassford

B.K. Glassford,
Geologist.

BKG/sd

Files

Mr. R.S. Chapman,
District Engineer,
District #11, Huntsville.

Mr. W.D. Ham,
Construction Engineer.

Soil Mechanics Section,
Geotechnical Office,
West Building, Downsview.

March 21st, 1974.

RE: Construction of Foundations of the New Structure at the Crossing of Hwy. #118 (Line 'E') and Indian River, Village of Port Carling, Regional Municipality of Muskoka, District #11 (Huntsville), Contract #73-125.
W.O. 71-11074 W.O. 401-64.

As requested by your Office, personnel from this Office visited the abovementioned construction site to evaluate the rock conditions encountered at the pivot foundation area, on February 28th, 1974.

The Contractor claims that the rock encountered in the vicinity of B.H.'s #5-#9 inclusive, appears to be much harder than as indicated in the Foundation stratigraphical drawing.

It is further understood that the Contractor wishes to employ blasting techniques to carry out the remaining portion of the necessary rock excavation for the Bascule Pier foundation. However, the contract documents indicate that blasting techniques should not be used because of the existing canal facilities being in close proximity to the excavation.

Our observations at the site revealed that the rock condition is similar to that as described in the Geologist's report contained in our Foundation Report (W.O. 71-11074). This condition was further explained in a memo from Mr. B.K. Glassford, Geologist, Geotechnical Office dated March 13th, 1974, which is appended to this memo.

It should be noted that the Contractor was notified that the complete foundation investigation report for this structure was available for his review. This notification was contained in Addendum No. 2 of Contract No. 73-125, dated September 4th, 1973.

...../2

Mr. R.S. Chapman - RE: W.O. 71-11074.

In our opinion, had the contractor interpreted all the supplied information properly, he would have anticipated solid rock and employed appropriate excavation techniques to reach the founding level, other than the blasting method. However, if the Contractor wishes to use the blasting techniques, it will be his responsibility to obtain necessary approvals from the agencies involved and to ensure the safety of the surrounding facilities during blasting operations.

The aforementioned comments were discussed with construction personnel at the site. Should you require additional information or further clarification of any aspect of this project, please feel free to contact our Office.

M. Devata

M. Devata,
Supervising Engineer.

MD/mj
Attach*
c.c. C.S. Grebski
J.W. MacDougall
G. Martens

Files
Documents

O. J. GAFFNEY LIMITED

P.O. BOX 700 - DIAL 271-8800 (Area Code 519) - STRATFORD, ONTARIO

March 29, 1974.

Mr. J. E. Callaghan,
Director Construction Branch,
Ministry of Transportation and Communications,
Downsview, Ontario.

Dear Sir:

Re: Contract No. 73-125
Indian River Bridge, Port Carling
Huntsville District

In our telephone conversation of March 27, I agreed to summarize for you the events surrounding the use of explosives for removal of sound rock in the bascule pier excavation for the above noted structure.

To assist in describing the chain of events there is attached hereto a chronology of the various discussions and other communications concerning this subject.

Ernway Limited is our subcontractor for the rock excavation for structures on this contract. In removing the rock for the North abutment, Ernway encountered a small quantity of sound rock which could not be removed with the heavy air impact equipment that was used to remove the fractured rock in the North abutment area. This sound rock was removed by using a rock splitter in holes drilled into the rock. The cost of the removal of this sound rock was approximately $300.00 per cubic yard.

Having encountered the sound rock in the excavation for the North abutment, there was an immediate concern that this same material might be encountered in the excavation for the bascule pier. It was this concern that prompted the inquiry at the end of January with respect to the possibility of using explosives if, in fact, sound rock was encountered.

On February 18, Ernway did encounter sound rock in the bascule pier which could not be removed using the heavy air impact equipment. Incidentally, Ernway was using a Kent Airam 2000 which we are told developes the greatest amount of energy available from this type of equipment. To remove this sound rock underwater by drill and rock splitter was estimated to cost some $900.00 per cubic yard.

In view of the fact that the contract documents show sound rock at an elevation below the underside of the tremie concrete and badly fractured or moderately fractured rock in the areas of rock excavation, it was evident that we had encountered a condition not anticipated in bidding for this contract. To circumvent the exceedingly high cost of a claim for additional compensation that would result from drilling and splitting with a rock splitter, the much less costly method of the controlled use of explosives was investigated.

As the special provisions of the contract prevented the use of explosives to remove rock from the bascule pier excavation, we inquired if, in view of the faulty information on the soil strata drawing, blasting might be considered. This inquiry led to the convening of a meeting at the site on February 27, minutes of which as recorded by Wyllie and Ufnal are attached. Pursuant to that meeting we received the attached letter from Mr. W. D. Ham, dated March 1, in which he reiterated that the contract did not allow the blasting of rock, but went on to say that he had advised Mr. R. Weir of the Ministry of Natural Resources that the M.T.C. would not allow blasting until they had a letter from the M.N.R. and the contractor's letter acknowledging responsibility for any resultant damage. As this requirement had been mentioned verbally at an earlier date, our letter acknowledging responsibility for resultant damage went to the Huntsville office of the M.T.C. on March 1.

Test blasting was carried out March 6 after Wayne Borer of Ernway Limited had heard the letter from the M.N.R. giving their approval to a controlled blasting procedure read to him over the telephone two days previous. This letter which was dated March 6 apparently did not reach your Huntsville District Office until March 11. Prior to the test blasting for which the recommendations of C.I.L. were followed and the siesmic results were monitored by V.M.E., an inspection of the lock and associated mechanisms was made by the lockmaster, his assistant and Wayne Borer of Ernway Ltd. The results of the V.M.E. Monitoring are attached hereto and record siesmic readings that indicate that the possibility of any damage is extremely remote. Only the test blasting has been carried out. No subsequent blasting has been undertaken.

.... 3

We find that only a small quantity of the solid rock that is to come out has in fact been removed as a result of the test blasts. Subsequent to March 6 drilling through the overburden to reach the remaining rock excavation areas was undertaken but, because the holes could not be held, the overburden was removed and as a result the excavation is flooded. We are presently setting up to drill the area for rock removal in the bascule pier and intend to try the rock splitter underwater. We are also proceeding to unwater the valve chamber so that an evaluation might be made of any possible damage in that rather sensitive area.

In summary, we believe we proceeded in complete good faith when we encountered conditions we felt were beyond the terms of our undertaking. Our prime purpose was to minimize the ultimate cost to the M.T.C. by circumventing, if possible, the claim route. A claim based on removal of this sound rock by drilling and splitting underwater would be substantial. We also were most anxious to avoid the extended time that would be consumed by the drill and split procedure and the resultant disarray that kind of delay would cause for our progress schedule.

We feel that a meeting of all parties involved in this rock removal controversy should be convened at an early date so that all of the facts and factors might be recorded. Such a record would, in our opinion, provide the best guide to future actions and decisions in connection with this whole matter.

Yours very truly,

D. J. GAFFNEY LIMITED

D J Gaffney

D. J. Gaffney

DJG/lh
Encl.

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

RECEIVED APR 2? 1974 CONSTRUCTION BRANCH

To: Mr. J.E. Callaghan,
Director, Construction Branch,
Downsview.

From: W.D. Ham,
Dist. Construction Engr.,
Dist. #11, Huntsville.

Attention: Mr. G. Martens,
Asst. Const. Engr.

Date: April 16th, 1974.

Our File Ref.

In reply to

Subject:

Re: Contract #73-125, Hwy. #118, Indian River Bridge, Port Carling.

In reply to O.J. Gaffney's letter to J.E. Callaghan on March 29th, 1974, and especially your inquiry on the rock, I submit the following.

The Consultant tells me that the actual rock profile in the bascule pier area agrees very closely to that shown on the Contract Drawings.

Bridge Drawing No. 2 and No. 3 (page 30 and 31) indicate the bore hole locations and soil strata. The bascule pier area is in the section surrounded by bore holes 5, 6, 7, 8 & 9. Also refer to the Foundation Investigation Report for borehole records.

Page #41, Bascule pier elevations, show that the top of the tremie slab is at Elev. 732.5; bottom of tremie slab, East side, at Elev. 727.5. Actual sound rock elevation in the field at Elev. 734.0 (by scaling on Page #41 this elevation checks).

Borehole No. 5 indicates bedrock from Elev. 726.8 to Elev. 735.0, varying from moderately fractured to weathered. This range includes the underside of the tremie concrete elevation and thus does not agree with the Contractor's letter, Page 2, second paragraph. I do not believe that the soil strata information is faulty.

We are not aware that any inspection was made by the lockmaster, his assistant and the sub-contractor. The first three blasts were test blasts and the fourth blast was with maximum powder similar to the third blast (last paragraph, Page #2).

Continued ..../2

The small quantity of rock removed by the test blasts (first Par., Page #3) was approximately 3.5 Cu.Yds. The valve chamber has been unwatered and there is no damage.

I do not believe that the Contractor proceeded in good faith because the sub-contractor drilled rock on Mar. 4 and 5, advising that he would not blast until written permission from our Ministry was given. He put off four blasts on March 6 without permission. The Ministry's geologist advised that the rock encountered was as described in the Contract documents. If this is true the Contractor would have no claim.

W D Ham

W.D. Ham,
Dist. Construction Engr.

WDH/es
c.c. Mr. J.W. MacDougall.

~~Mr. Davis~~
Mr. Mirza

Mr. L. R. Eadie,
Executive Director,
Operations Division.

J. W. MacDougall,
Claims Engineer.

May 8, 1974.

Re: Claim on Contract 73-125
O. J. Gaffney Limited
Huntsville District

Attached please find for your information, copy of Notification of Intent to Claim dated April 15, 1974 from O. J. Gaffney Limited regarding the above contract.

J. W. MacDougall,
Claims Engineer.

JWM:dk
Attach.
c.c. - J. B. Wilkes
C. R. Wilmot
A. McConnell
A. C. Lennox
J. M. Crannie
R. S. Chapman

RECEIVED
MAY 15 1974
Executive Director of Design
M. T. C.

D. H. O.
TORONTO
RECEIVED
MAY 16 1974
BRIDGE
OFFICE

71-11074

73-125

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO:	Mr. M. Devata, Supervising Foundation Engineer, Soils Mechanics Section, Geotechnical Office, West Bldg.	FROM:	Pavement Structure Design Section, Geotechnical Office, West Building.
ATTENTION:		DATE:	July 30, 1974.
OUR FILE REF.		IN REPLY TO	

SUBJECT: Bascule Pier Foundation
Hwy. #118, Indian River, Port Carling, Ontario
W.P. 401-64 W.O. 71-11074

The problem encountered with artesian water flow while drilling holes for anchor tie-rods in the Bascule pier foundation appears to have originated from the drill encountering an open joint in the rock structure that was making water and was under pressure from adjacent canal and river water bodies.

The water flow was blocked off and the problem appears to be of a local one hole occurrence. Additional holes have since been drilled with good results having no water artesian flow difficulties.

The rock in this specific area as indicated by the geological report and core logs is characterized by a set of four joints, vertical and inclined. A similar water artesian flow pattern could occur again if the drilling encountered another open water pressured joint in the gneissic rock.

B K Glassford /sd

B. K. Glassford,
Geologist.

BKG/sd

C.C. Harry Martin.

Mr. Rutka + Mr. Davis

Mr. L. R. Eadie,
Executive Director,
Operations Division.

J. K. Livingston,
Assistant Claims Engineer.

August 16, 1974.

Re: Claim on Contract 73-125
O. J. Gaffney Limited
Huntsville District

Attached please find for your information, copy of Notification of Intent to Claim dated August 12, 1974 from O. J. Gaffney Limited regarding the above contract.

ORIGINAL SIGNED BY J. K. LIVINGSTON

J. K. Livingston,
Assistant Claims Engineer.
(for) J. W. MacDougall,
Claims Engineer.

JKL:dk
Attach.
c.c. - J. B. Wilkes
E. J. Orr
C. R. Wilmot ✓
G. F. Wetherall
A. C. Lennox
J. M. Crannie
R. S. Chapman

RECEIVED AUG 22 1974 MGR. GEOTECHNICAL OFFICE

D. H. O. TORONTO RECEIVED AUG 20 1974 BRIDGE OFFICE

Ministry of Transportation and Communications
Ontario

Memorandum

✓ M. Devata — would you
B. Glassford please get together
look at the data you
have, summarized
necessary & attend the meeting on
Mar 18. If you require any
further information you
might contact Peter
Peacock.
A.R.

To: Mr. A. Rutka
Geotechnical Office

From: P. H. Peacock
Construction Branch

Attention:

Date: February 24th, 1975

Our File Ref.

In Reply to

Subject:

Contract 73-125
W.P. 401-64-03-04
Highway 118 Indian River Bridge
Port Carling

I am in receipt of numerous claims on the above, two of which have specific reference to the Geotechnical Office.

(1) Intent dated February 28th concerning "sound rock encountered in the Bascule Pier foundation. As a result the character of the work is changed from that indicated in the contract documents".

We have now received a claim from the Contractor in the amount of $52,503.96. This is about $32,000 more than he will be paid under the contract. The site was visited and a report made by Mr. G. K. Glassford dated March 13, 1974 and also by Mr. M. Davata dated March 21st.

(2) Intent dated August 12, 1974 concerning the "inflows of water and sand through the drill holes for the reinforcing stool dowels". Contractor further states that this condition revealed all rock below this pier foundation is not sound as assumed by contract drawings. Mr. Glassford's memo of July 30th refers to this but the work was not completed when he visited the site. Claim has now been received in the amount of $7,827.18.

These two claims did not appear to me to have much virtue and in fact appear to be to some extent self contradictory. There is no evidence that the Contractor took the trouble to look at our borhole information. Nonetheless I would appreciate your

..../2

examining all the information afresh and it would be appreciated if you could have a representative at a meeting to discuss these claims with the district and consultant provisionally fixed for March 18th. At some later date we shall have to have a meeting with the Contractor and it is necessary that we be suitably prepared. I should be glad to discuss this matter further with you if you see fit.

P. H. Peacock,
Assistant Construction
Engineer.

PHP/aln

cc: Mr. J. MacDougall

Ministry of Transportation and Communications
Ontario

Memorandum

To: Mr. R.S. Chapman,
District Engineer,
District #11, Huntsville.

From: P.H. Peacock.

Attention: Mr. W.D. Ham

Date: March 12, 1975.

Our File Ref.

In Reply to

Subject: Contract 73-125, Highway 118, Port Carling

MD please, Cam.

Further to my conversation with you, a meeting will be held to discuss outstanding claims on the above contract on Thursday, March 20, 1975, commencing at 10:00 a.m. in the Construction Boardroom in ~~this office~~. This supersedes previous suggestions of Tuesday and Wednesday of the same week.

The following will be discussed:-

1 - North abutment - rock above top of footing.
2 - Bascule pier - rock above top of footing.
3 - Bascule pier - rock below top of footing.
4 - Grouting of cavities in rock.
5 - Changes in concrete quantities.

[illegible]ve previously sent comments on all the above except the last. Regarding this, I should like to know the reasons for these changes. If they are corrections of arithmetical errors, the 5% is applicable. If they are changes due to site conditions on the instructions of the consultant, I do not think 5% would be applicable and we would pay for all of the concrete extra costs. This would obviously not be a proportion of the lump sum which includes formwork etc., and would be the cost of concrete and placing.

Would you please look into this matter and let me have your recommendations.

P.H. Peacock,
Assistant Construction Engineer,
Construction Branch.

PHP/bc

c.c. - A. Rutka
L. Fisher
J.M. Crannie
A. McKim

Would you please have a rep present with Foundation Report

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. P.H. Peacock,
Assistant Construction Engineer,
Construction Branch.

FROM: Pavement Structure Design Section,
Geotechnical Office,
1st Floor, West Building.

ATTENTION:

DATE: June 12, 1975.

OUR FILE REF.

IN REPLY TO

SUBJECT: Bascule Pier Foundation
Indian River Bridge, Port Carling, Ontario
Highway 118, Contract 73-125 - Claim of O.J. Gaffney

Mr. M. Devata of the Soil Mechanics Section asked me to clarify the references to 'solid' and 'sound' rock as requested in your "Minutes of Meeting" dated March 20, 1975 regarding the above claim.

The following are definitions of 'solid rock' as taken from appropriate references:

Solid Rock - a British term for a consolidated rock.
Glossary of Geology,
American Geological Institute, 1972.

Solid Rock - rock which is both consolidated and in situ.
Dictionary of Mining and Minerals and related terms,
U.S. Department of the Interior, 1968.

Solid Rock - is defined as sound rock which cannot be broken down by hand picks and includes boulders of 1 cubic yard size.
Dictionary of Applied Geology,
Mining and Civil Engineering, 1967.

Solid - of uniformly close, coherent and compact texture, not loose or spongy; having no break or interuption.
Webster's Dictionary, 1974.

Solid - homogenous, alike throughout.
Oxford Dictionary, 1964.

Sound - synonym for solid as applied to rocks and diamonds.
Oxford Dictionary, 1964.

These descriptions of 'solid rock' are closely related and convey the same meaning. My use of the term 'solid rock' in the memo to Mr. Devata of March 13, 1974, was to imply that the rock as described by Mr. K. Ingham in a previous memo of October 25, 1971 was a homogenous, consolidated, mass of rock and not being in a condition of broken fragments of various sizes

cont'd.../2

Mr. P.H. Peacock June 12, 1975

Bascule Pier Foundation
Indian River Bridge, Port Carling, Ontario
Highway 118, Contract 73-125 - Claim of O.J. Gaffney

of rock loosely compacted and bearing upon each other. This description of 'solid rock' was confirmed later on during the construction period by visits to the site on May 2, 1974 and July 26, 1974.

Referring to the Oxford Dictionary, 1964, the word 'sound' is used as a synonym for solid when applied to rocks or diamonds. Thus, it would appear that 'solid rock' and 'sound rock' are of the same description.

B.K. Glassford

B.K. Glassford,
Geologist.

BKG/sg
cc: - G.A. Wrong
M. Devata✓
B.K. Glassford

Ministry of Transportation and Communications

Ontario

M. Devata

Memorandum

To: Mr. J. W. McDougall,
Claims Engineer,
Engineering Claims Office,
Central Building, Downsview.

Attention: Mr. L. D. Fisher.

From: Structural Office,
West Building, Downsview.

Date: November 26, 1975.

Our File Ref.

In Reply to

Subject: Contract 73-125,
Indian River Bridge at Port Carling,
Highway 118, District #11.
Claim regarding pressure grouting of dowels for Bascule Pier columns.

Please refer to your memo of November 3, 1975.

Our answers to your questions are as follows:

1. Had we been aware of the situation encountered, the contractor's attention would have been drawn to this fact by a special note on the Structure Drawings, stating that additional grouting might be necessary in order to seal off the bedrock in the vicinity of the pier columns outside the perimeter of the footing prior to grouting the dowels.

2. On page six of the Foundation Report the fourth paragraph describing the bedrock states:
"Fractures are common and apparently form a fairly close pattern. Evidence of four sets of fractures is present in the cores, one sub-parallel to the lineation, a horizontal set, a vertical set and less common a steeply inclined set. These joints are generally tight except where noted. In some instances open joints are sandfilled near the surface."

9 Although the Foundation Report has indicated presence of horizontal and inclined fractures in the bedrock, the extent and the difficulties encountered during construction were not anticipated.

The bedrock, in addition to being fractured, was probably also shattered during construction of the canal, locks and pier of the old structure.

....2

3. We feel the contractor has a legitimate claim and is entitled to an additional payment.

A. Radkowski

AR/CSG/cf

A. Radkowski,
Regional Structural Design Engineer.

for: C. S.Grebski,
Structural Design Engineer.

c.c. M. Devata
A. McKim

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

To: Mr. M. Devata,
Soil Mechanics Section.

FROM: L. D. Fisher

ATTENTION:

DATE: November 28, 1975

OUR FILE REF.

IN REPLY TO

SUBJECT: Contract 73-125, Indian River Bridge at Port Carling
Highway 118, District #11
Claim re: Pressure Grouting of Dowels for Bascule Pier Columns

Under the contract, the contractor was to drill through the bascule pier foundation slab which is at elevation 733 ± to elevations in bedrock varying from 714.0 to 720.3. Reinforcing bars were to be inserted in the drill holes and grouted in as noted in General Note 3, page 40. There is an S.P. concerning grouting in the contract. 24 holes were to be drilled, 12 in each of two locations. When the contractor went to place the rebars in the holes he found he could not get to the bottom of the hole due to sand. This sand was carried in to the previously drilled hole through a seam in the rock, which I understand was at elevation 725.5.

The contractor has claimed for additional costs he incurred as a result of the inflow of sand and water through this seam. Your bore holes 5 - 9 inclusive are adjacent to or within the bascule pier foundation area. Your Foundation Investigation Report W.O. 71-11074, W.P. 401-64 gives information on the bore holes. My question is, from the information shown on these bore holes, and now knowing the elevation of the seam, would a prudent contractor have expected an abnormal inflow of water and sand into his bore hole? The Bridge Office have advised me that had they been aware of the situation encountered, the contractor's attention would have been drawn to the fact by a Special on the structural drawings. Since such a Special was not included it is obvious that neither the Bridge Office nor their consultant, who has verbally confirmed this fact, anticipated such a problem, it would then seem that it would not necessarily be logical to expect a contractor to anticipate the problem. In any event, I would appreciate your comments to my question above.

L. D. Fisher,
Assistant to Claims Engineer,
(for) J. W. MacDougall,
Claims Engineer.

LDF/jm

Ministry of Transportation and Communications

Ontario

Memorandum

To: Mr. M. Devata,
Supervising Engineer,
Soil Mechanics Section,
West Building.

Attention:

From: Pavement Structure Design Section,
Geotechnical Office,
West Building.

Date: December 24, 1975.

Our File Ref.

In Reply to

Subject: Contract 73-125, Hwy. 118, District 11
Indian River Bridge - Port Carling
Claim Re: Pressure Grouting of Dowels
for Bascule Pier Columns

I have reviewed the correspondence, memos and meeting notes concerning the above claim. The following are my comments:-

Under the Geological and Log of Holes report made by Mr. K. W. Ingham for this project the following pertinent facts are noted.

1. Rock type is a granite gneiss with pronounced lineation dipping at 50°.

2. Fractures in this rock type are common. Four sets are noted from the core logging. These fractures or joints are generally tight, except where noted. In some instances joints are sand filled.

3. Unusual rock conditions were noted in the vicinity of borehole #6 (Bascule Pier Vicinity). Here the rock appears to dip toward the river, coupled with several open fractures. There may be an unstable area of rock in the immediate vicinity (Bascule Pier Vicinity). The Log indicates a 1.7 footage of fine and medium sand at 725.00 elevation for hole #6.

4. The log of holes cite fractures or joints throughout the drilling outline. Such rock structural features give an indication as to the rock characteristics.

These initial facts should have indicated to the contractor before his bidding and after work began that water and sand probably would be encountered in the drill program due to the adjacent water flows in the river and canal. The 1.7 feet of sand in hole #6 at 725+00 elevation is a serious and unusual structural feature.

Also a great flow of water with sand was encountered in the excavation work for the pier pit. This again should have warned the contractor that water and sand could be encountered with the drilling through the Bascule Pier foundation tremie for the anchoring dowels.

Cont'd.../2

Contract 73-125, Hwy. 118, District 11
Indian River Bridge - Port Carling
Claim Re: Pressure Grouting of Dowels
for Bascule Pier Columns

Contract documents stated that no blasting of the rock would be allowed. The contractor was required to bid this way in his tender. This restriction was on account of the proximity of the canal and the lock mechanism, also because of the structure of the rock involved with its inherent fracture and joint pattern characteristics. Blasting was undertaken without knowledge of the MTC. The vibrations from this blasting probably opened up many fractures or joints as well as causing a further water and sand flow pattern through the openings. At the time of my visit to the site on July 26/74 water with some sand was flowing from depth through the bored holes in the Bascule pier foundation pad. This could be the direct effect from the blasting on the rock or from a normal water flow as an in-situ feature of the joint-fracture system, especially from the large open seam noted at 725+00 elevation on the log of hole #6, which was sand filled for a total of 1.7 feet.

The contract documents show that the overburden in the immediate area of the site is granular and that the undulated rock surface is in direct contact with this material, hence a water-sand flow pattern would be present through the joints in the rock, water coming from the Indian River and canal system.

During the excavation construction of the pit for the Bascule pier, water was present and the pit area flooded. The contractor was using the services of a diver to locate and position the drill bit for rock removal purposes. The contractor should have seen then that a serious water flow existed. This was some weeks before the foundation pad of tremie was poured. At the meeting on March 20, 1975 regarding various claims on the contract, attended by the consultants Wyllie and Ufnal Limited and MTC personnel it was generally agreed that water was to be expected in the drilling of the holes through the Bascule Pier foundation pad. The sand content appearing in the holes appears to have originated from an open sand filled joint at an approximate elevation of 725.00 (which was indicated in the log of hole #6, at or near this elevation for 1.7 feet).

During the grouting procedure of the 24 holes in the pier foundation pad, the pressure grout at one time was seen coming to the river's surface down stream a few hundred feet from the point of origin at the Bascule Pier foundation pad. Such a feature suggests an open fracture or joint of such dimension that would easily carry sand from a higher elevation.

Cont'd.../3

Contract 73-125, Hwy. 118, District 11
Indian River Bridge - Port Carling
Claim Re: Pressure Grouting of Dowels
for Bascule Pier Columns

In conclusion, it is noted that information supplied to the contractor in the documents indicated conditions in the rock similar to those encountered during the excavation construction work. I contend that the contractor should have anticipated such a problem of sand and water flow. The contractor, their consultant and the Bridge Office at MTC should have been aware and anticipated a sand-water problem in the Bascule Pier foundation pad area as it is well noted under the log of holes. Hole #6, which states unusual rock conditions are present and 1.7 feet of fine to medium sand with some wood particles at elevation 725+00. "There may be an unstable area of rock in the immediate vicinity of hole #6" appears in the geological report. This important feature seems to have been overlooked by the contractor, consultant and the Bridge Office.

B.K. Glassford

B.K. Glassford,
Geologist.

BKG/sd
cc:- A. Rutka
G.A. Wrong
B.K. Glassford
Files

Mr. J.W. MacDougall
Claims Engineer
Engineering Claims Office
Central Building, Downsview

Mr. L.D. Fisher

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

January 8, 1976

Indian River Bridge at Port Carling
Claim Re: Pressure Grouting of Dowels
for Bascule Pier Foundations
Cont. 73-125, Hwy. 118, District 11
W.P. 401-64-03

The contractor has claimed for additional costs as a result of the inflow of sand and water into the drilled holes while placing dowels for the bascule pier foundations. In your memorandum dated November 28, 1975, you have requested the Soil Mechanics Section to review this claim and provide our comments. We have reviewed all the available foundation data, together with related construction information and submit the following:

I. CONSTRUCTION REQUIREMENTS AS PER TENDER

1. The special provisions for item 63 "Cofferdam and Unwatering" indicates that "the sheet piling shall be installed through the overburden to the depth of the existing rock surface, but not higher than the minimum elevation shown on the drawings.

"Pre-excavation of the rock to the required depth will be necessary to install the sheet piling in the east wall and a portion of the south wall. No blasting will be permitted for rock excavation in the bascule pier foundations." "Excavation, installation of sheet piling, and placing of concrete for the foundation slab shall be carried out before the cofferdam is unwatered." In addition to this, on page 40 of the contract drawings, a general note states that "the concrete in the foundation slab shall be cast to elevation 732.5 by tremie methods."

2. A total of 24 dowels for the north wall of the bascule pier foundation will be required and are as follows:

West Side - 6-5" ø dowels to elev. 717.0
6-5" ø dowels to elev. 714.0

East Side - 6-5" ø dowels to elev. 720.5
6-5" ø dowels to elev. 717.5

cont'd....

II. SEQUENCE OF CONSTRUCTION

The construction sequence was initiated by the removal of the more weathered and fractured portion of the granite gneiss bedrock to the design elevation. A sheeted cofferdam was then driven to the newly established bedrock surface. A seal of tremie concrete was then poured under water inside the cofferdam to elevation 732.5 to counterbalance the hydrostatic head existing at the base of the excavation. (Indian River controlled water elevation 739.5)

The bascule pier area enclosed within the sheeted cofferdam was then unwatered. The installation of the dowels for the north wall was then initiated. This work consisted of 5" ∅ holes drilled by means of a drilling machine located at the surface of the tremie concrete seal (elevation 732.5). Once the drilling extended through the tremie concrete into the bedrock, the flow of water was observed from the drilled hole. Subsequently, the sand which filled the drilled hole bottom negated attempts by the contractor to install the dowels to the required elevation.

III. DISCUSSIONS

A detailed foundation report containing factual data such as subsoil, bedrock and groundwater conditions was available to the contractor as well as to the designer (structural consultant Wyllie and Ufnal Ltd.). The foundation report describes in detail the bedrock conditions at various borehole locations emphasizing the fractured nature of the rock. The following portion of the text of the foundation report clearly illustrates the nature of the bedrock.

"Fractures are common and apparently form a fairly close pattern. Evidence of four sets of fractures is present in the cores, one sub-parallel to the lineation, a horizontal set, a vertical set and less commonly a steeply inclined set".

"In some instances, open joints are sandfilled near the surface". (See B.H. #6)

In addition, the condition of the in situ rock can be seen wherever it is exposed in the existing canal area which is only 14 feet away from the bascule pier foundations. Furthermore, the contractor might have experienced himself the fractured or open-jointed nature of the bedrock during the removal of the rock for pier construction. The specific requirements for a tremie concrete seal within a sheeted cofferdam in the contract for the construction of the bascule pier foundation indicates that the footing excavation could not be unwatered due to the pervious nature of the bedrock. Since the bedrock is pervious, it is in direct communication with the river water through the granular overburden.

The water level outside the cofferdam was 7 feet higher than the top of the tremie concrete seal. The drilling of the dowel holes broke the watertight seal; consequently a flow of water was created due to the excess hydrostatic head when the drilled hole extended into the pervious bedrock.

cont'd....

This flow of water resulted in the migration of the fine grained particles from the granular overburden of the subsoil or sand from the sand filled open joints of the bedrock.

Flow conditions created by the contractor due to the excess hydrostatic head could have been eliminated by modified drilling techniques. One method of achieving this is by drilling from a raised platform so that the top of the drilling casing would be slightly above the prevailing hydrostatic head (river water level 739.5). If the casing is kept full of water at all times, little or no inflow of sand or water would occur. Such techniques are not uncommon since they are widely employed in our day to day drilling operations. If such techniques are employed the contractor could have installed the dowels without any serious problems associated with inflow of water and sand into the drilled holes for dowels.

IV. CONCLUSIONS

An experienced contractor who is presented with the various facts such as:

- the water level in the river
- the necessity of tremie concrete seal in the enclosed sheeted cofferdam
- the fractured nature of the bedrock as described in the foundation report

should have recognized the problems associated with the excess hydrostatic head during the installation of the dowels.

In addition, the contractor should have inferred from his own observations of the bedrock conditions at the site during construction, that the inflow of water and sand in the dowel holes would occur. A prudent contractor could have overcome this inflow of water and sand by employing proper drilling techniques as discussed previously. In our opinion there were no unforeseenor unusual conditions which could not have been anticipated by the contractor.

We have attached a copy of a memorandum prepared for this claim by Mr. B.K. Glassford, Geologist, Geotechnical Office.

The aforementioned comments summarizes our verbal discussions with you and should you require any further clarification, please contact our office.

M. Devata
Supervising Engineer

Enclosure

cc: C.S. Grebski
A. Radkowski
P.H. Peacock
W.D. Ham
A. Rutka
B.K. Glassford
A. McKim
Files
Record Services

L.D.

O. J. GAFFNEY LIMITED

P.O. BOX 700 - DIAL 271-8800 (Area Code 519) - STRATFORD, ONTARIO

Mr. J.W. MacDougall,
Claims Engineer,
Ministry of Transportation & Communications,
1201, Wilson Avenue,
Downsview, Ontario.
M3M 1J8

January 25th, 1977

Attention: Mr. L.D. Fisher.

Re: M.T.C. Contract 73-125, Port Carling, Ontario.

Dear Sir:

Further to our discussions last Tuesday, I wish to record with you some of the points we feel are central to the question of making payments to us for our Bascule Rock claim.

1. The Contract Drawings are misleading. These drawings, especially the "Soil Strata" drawings No. 42-1-2 and 42-1-3 clearly indicate the Contractor will be required to excavate something other than sound rock. 1

2. The rock we exposed and inspected in the area of Bore Hole no 5 on February 27th, 1974 was sound rock by whatever definition source you wish to consult. 2

3. The Soils Report in real terms supports the aboveviews when it recommends an allowable bearing pressure of up to 10 T.S.F for the moderately fractured rock. This loading is used by design experts for shattered, broken, bedrock or hard shale. 3

4. The verbal descriptions you referred to in the Soils Report are not clearly definitive to the point where they exclude our interpretation of their meaning. 4

Yours very truly,
O.J.GAFFNEY LIMITED.,

J.H.Hallam
J.H.Hallam.

JHH:cd

10-132

Ministry of Transportation and Communications
Ontario

Memorandum

Cont. 73-125

To: Mr. J. W. McDougall,
Claims Engineer,
Engineering Claims Office,
West Building, Downsview.

From: Structural Office,
West Building, Downsview.

Attention: Mr. L. D. Fisher
Asst. to Claims Engineer.

Date: January 27, 1976.

Our File Ref.

In Reply to

Subject: Indian River Bridge at Port Carling,
Contract 73-125, O. J. Gaffney Claim,
W. P. 401-64-03, Site 42-1.
District #11.

Please refer to your memorandum of January 20th, 1976 to Mr. C. Grebski.

The dowels placed for the bascule foundation were straight bars on the south side of the pier and bent bars at 90° on the north side of the pier.

Your assumption that the casings were to be removed during grouting operations is correct.

AR/cf

A Radkowski
A. Radkowski,
Regional Structural Design Engineer.

for: C. S. Grebski,
Structural Design Engineer.

c.c. M. Devata

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. C. Mirza,
Head, Soil Mechanics Section,
West Building.

FROM: L. D. Fisher

ATTENTION:

DATE: January 31, 1977

OUR FILE REF.

IN REPLY TO

SUBJECT: Claim Contract 73-125, O. J. Gaffney Limited,
Rock Excavation Bascule Pier Foundation,
Indian River Bridge, Port Carling

Based on your reports of March 21, 1974, Devata to Chapman, March 13, 1974, Glassford to Devata, and June 12, 1975, Glassford to Peacock, this office has continued to deny any liability towards additional costs expended by the contractor in removing rock which he claims to be sound rock, from the lower portions of the bascule pier foundation.

The contractor continues to assert that the rock was not moderately fractured as the drawings led him to anticipate, but was sound bedrock.

Attached is a copy of his latest submission of January 25, 1977. Would you please provide this office with your comments on the four points that he makes.

L. D. Fisher,
Assistant to Claims Engineer,
(for)J. W. MacDougall,
Claims Engineer.

LDF/jm
attach.

Mr. J.W. MacDougall
Claims Engineer
Engineering Claims Office
Central Building, Downsview

Soil Mechanics Section
Engineering Materials Office
West Building, Downsview

February 16, 1977

Mr. L.D. Fisher
Cont. 73-125

your memo of Jan. 31/77

Re: Gaffney Claim - Cont. 73-125
Indian River Bridge, Port Carling
District 11, Huntsville

This responds to the four points raised in Mr. Hallam's memorandum of January 25, 1977 attached to your memorandum of January 31, 1977.

1. Contract Drawings No. 42-1-2 and 42-1-3 clearly indicate, by an asterisk notation, that for detailed bedrock descriptions the borehole log sheets should be consulted. For any contractor to ignore this notation and invitation to examine the detailed description of the bedrock is to scorn the value of the information willingly made available by this Ministry. Therefore, no further comment need be made either on point number 1 or point number 4 of Mr. Hallam's letter. Let's keep the additional fact in mind that all bidders were advised by special provision, of the availability of a complete soils investigation report on the contract.

2. There is an argument presented in Mr. Hallam's letter in point number 2 that the rock exposed and inspected in the area of Borehole 5 on February 27, 1974 was sound rock. We should ask or find out who inspected the rock? What competence did the person inspecting the rock possess to judge whether the rock was sound or otherwise? Were MTC personnel advised of such a finding and did they concur, at that time, with such an opinion? If not, why not?

3. The recommendation of 10 tsf allowable pressure in the Foundation Report is based on a multitude of soil mechanics, rock mechanics, structural design, construction, and structure performance considerations. There is no unique relationship between allowable bearing pressures and soil or rock types, as implied by point number 3 in Mr. Hallam's letter. Bearing pressures are often limited due to settlement considerations. It will be recalled by the contractor that the bascule pier was to support a rather settlement-sensitive piece of mechanical equipment to enable the bascule span to be raised and lowered as desired.

I trust that the above comments clarify our position on this claim and answer the four points raised in Mr. Hallam's letter.

Original Signed by
C. MIRZA

C. Mirza, Head
Soil Mechanics Section

CM/gs

cc: B.K. Glassford
Files
Record Services

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31E-17

DIST. 11 REGION Northern

W.P. No. 401-64

CONT. No. 73-125

W.O. No. 71-11074

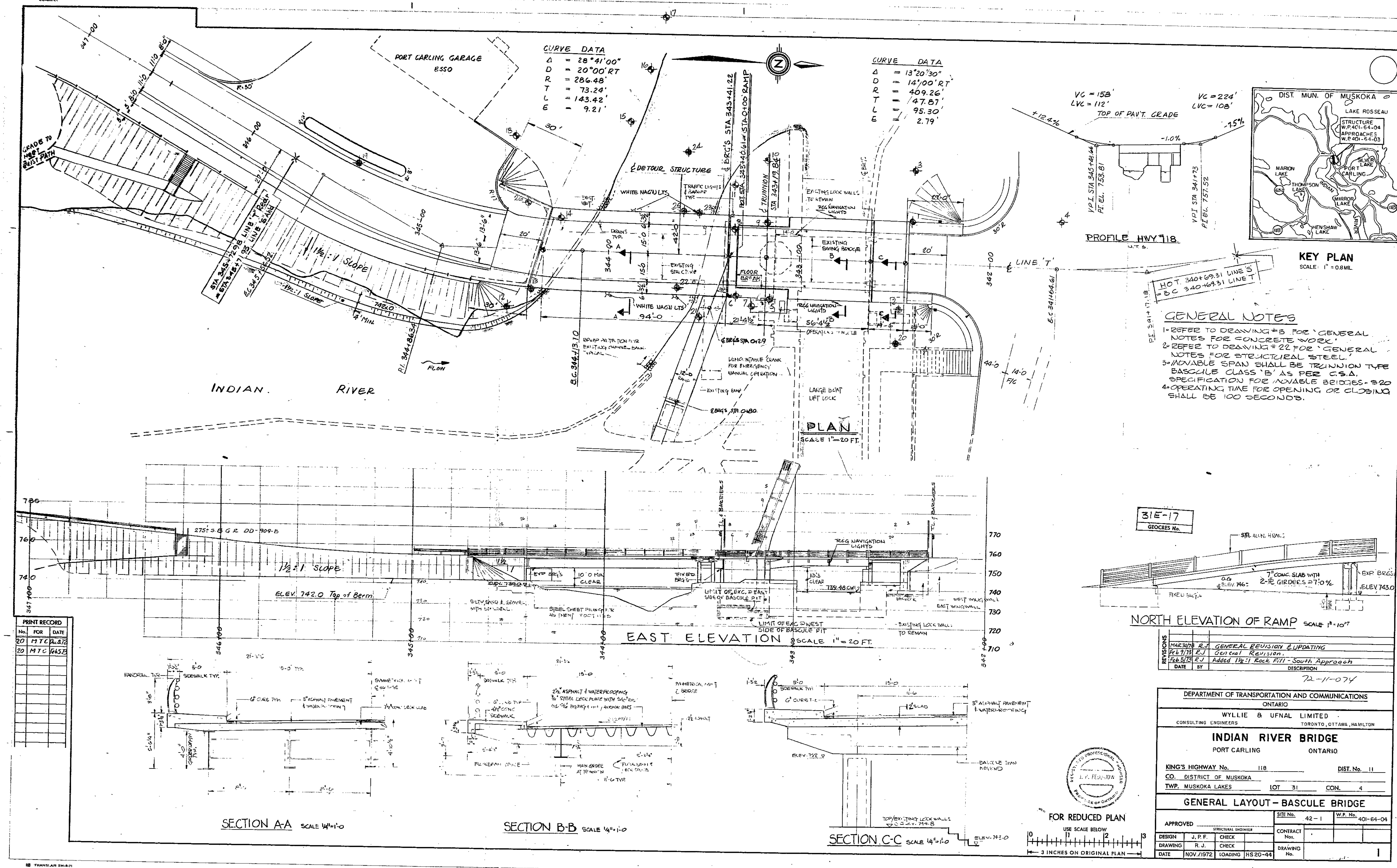
STR. SITE No. 42-001

HWY. No. 118

LOCATION Hwy 118 & Indian River

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 1

REMARKS: documents to be unfolded before microfilmed



GENERAL NOTES

1. REFER TO DRAWING #5 FOR 'GENERAL NOTES FOR CONCRETE WORK'
2. REFER TO DRAWING #22 FOR 'GENERAL NOTES FOR STRUCTURAL STEEL.'
3. MOVABLE SPAN SHALL BE TRUNNION TYPE BASCULE CLASS 'B' AS PER C.S.A. SPECIFICATION FOR MOVABLE BRIDGES - S20
4. OPERATING TIME FOR OPENING OR CLOSING SHALL BE 100 SECONDS.

PRINT RECORD

No.	FOR	DATE
20	MTC	[illegible]
20	MTC	[illegible]

REVISIONS

DATE	BY	DESCRIPTION
MAR 30/73	R.J.	GENERAL REVISION & UPDATING
Feb 9/73	RJ	General Revision.
Feb 5/73	RJ	Added 1½:1 Rock Fill - South Approach

72-11-074

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

WYLLIE & UFNAL LIMITED
CONSULTING ENGINEERS — TORONTO, OTTAWA, HAMILTON

INDIAN RIVER BRIDGE

PORT CARLING ONTARIO

KING'S HIGHWAY No. 118 — DIST. No. 11
CO. DISTRICT OF MUSKOKA
TWP. MUSKOKA LAKES — LOT 31 — CON. 4

GENERAL LAYOUT - BASCULE BRIDGE

APPROVED ______ STRUCTURAL ENGINEER				SITE No. 42-1	W.P. No. 401-64-04
DESIGN	J. P. F.	CHECK		CONTRACT Nos.	
DRAWING	R. J.	CHECK		DRAWING No.	1
DATE	NOV./1972	LOADING	HS 20-44		

Cont 73-125