

## MEMORANDUM

To: Mr. E. E. Davis,  
Bridge Engineer,  
Bridge Office,  
Admin. Bldg.  
ATTENTION: Mr. S. McCombie

FROM: Foundation Section,  
Materials & Testing Office,  
Room 107, Lab. Bldg.

DATE: March 14, 1969

MAR 16 1969

OUR FILE REF.

IN REPLY TO

SUBJECT:

## FOUNDATION INVESTIGATION REPORT

For

Proposed Crossing at Bar Creek and  
Hwy. #638, Township of MacDonald  
S.W. 1/4 Sect. 24 and N.W. Sect. 25  
District of Algoma  
District No. 18 (Sault Ste. Marie)  
W.J. 68-F-77 -- W.P. 263-66-02

Attached, we are forwarding to you, our detailed foundation investigation report on the subsail 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.

AGS/MdeF

Attach.

cc: Messrs. E. R. Davis (2)

H. A. Tregaskes

D. W. Farren

H. Hurrell

J. H. Blevins

S. B. Davidson

E. R. Saint

B. A. Singh

Foundations Files

Gen. Files

*A. G. Stermac*  
A. G. Stermac  
PRINCIPAL FOUNDATION ENGINEER

## TABLE OF CONTENTS

1. INTRODUCTION.
  2. DESCRIPTION OF SITE.
  3. FIELD AND LABORATORY WORK.
  4. SUBSOIL CONDITIONS:
    - 4.1) General.
    - 4.2) Clay.
    - 4.3) Clayey Silt, Sand and Traces of Gravel.
  5. GROUNDWATER CONDITIONS.
  6. DISCUSSION AND RECOMMENDATIONS:
    - 6.1) General.
    - 6.2) Approaches, Scour Protection and Dewatering.
    - 6.3) Bridge.
    - 6.4) Multi-Plate Pipe Arch Culvert.
  7. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION REPORT  
For  
Proposed Crossing at Bar Creek and  
Hwy. #638, Township of MacDonald  
S.W. 1/4 Sect. 24 and N.W. Sect. 25  
District of Algoma  
District No. 18 (Sault Ste. Marie)  
W.J. 68-F-77 -- W.P. 263-66-02

1. INTRODUCTION:

A request to carry out a foundation investigation for the proposed new bridge to carry Hwy. #638 over Bar Creek, was received from Mr. F. De Visser, Regional Bridge Location Engineer, in a memo dated October 15, 1968.

An investigation was subsequently carried out by this Section to determine the subsoil conditions existing at the site of the proposed bridge.

This report contains the results of our field and laboratory investigation, together with our recommendations for the foundations of the new structure.

2. DESCRIPTION OF SITE:

The new structure is proposed to be located at the same site as the existing one. The site is about 5.3 miles east of Echo Bay on Hwy. #638. The existing bridge is a 27.6 ft. span timber bridge, supported on piles whose lengths are not known. The structure is in poor condition.

The creek, at the bridge site, flows in a north to south direction. The topography is flat to gently rolling. Hillocks can be seen about half a mile north of the site. The land on both sides of the creek is covered with pasture.

### 3. FIELD AND LABORATORY WORK:

The field work at the site consisted of six sampled boreholes and two dynamic cone penetration tests. All boreholes were advanced using conventional diamond drilling equipment adapted for soil sampling purposes. A driving energy of 350 ft.-lbs. per blow was used for the dynamic cone penetration tests.

Disturbed samples were obtained using a 2-inch O.D. split-spoon sampler driven according to the specifications of the Standard Penetration Test. Undisturbed samples were obtained by means of 2-inch I.D. Shelby tubes which were pushed into the soil manually.

In-situ vane tests were carried out, wherever possible, at elevations 12 inches below various soil samples.

Samples were visually examined in the field and subsequently in the laboratory. The following tests were carried out on selected samples:

- 1) Grain-Size Distribution.
- 2) Atterberg Limits.
- 3) Natural Moisture Content.
- 4) Bulk Density.
- 5) Unconfined Compression Tests.
- 6) Quick Triaxial Tests.
- 7) Consolidation Tests.

The results of field and laboratory tests are summarized on the Record of Borehole sheets, which are contained in the Appendix to the report.

The locations and the elevations of boreholes are given on Drawing No. 68-F-77A, which is also contained in the Appendix to this report.

The borehole locations and elevations were surveyed by the Sault Ste. Marie District Office of the D.H.O.

4. SUBSOIL CONDITIONS:

4.1) General:

In general, the subsoil consists of a deposit of soft, sensitive clay, underlain by a mixture of clayey silt, sand and traces of gravel.

The boundaries between the different deposits are shown on the attached Record of Borehole sheets. The estimated stratigraphical profiles, shown on Drawing No. 68-F-77A, are based upon this information.

From ground level downwards, the different soil deposits are described as follows:

4.2) Clay:

This is the original soil at the site and was observed from ground level downwards. All the boreholes, except borehole 1, were terminated in this deposit. In borehole 1 this deposit extends to a depth of 69.0 ft., while the cone penetration test adjacent to borehole 4 indicates its depth to be about 73.0 ft. at that location. The material is highly plastic clay of red colour. The Atterberg Limits carried out on the samples from this stratum, show the following range:

Liquid Limit .....	63	-	91%
Plastic Limit .....	25	-	33%
Plasticity Index .....	30	-	60%
Natural Moisture Content ....	65	-	88%

The undrained shear strength was determined in the field by vane tests, and in the laboratory by means of undrained triaxial tests and unconfined compression tests. The shear strength, as determined from the field vane tests, ranged from 180 to 1,200 lbs./sq.ft., but, generally, was found to be between 300 - 500 lbs./sq.ft. The results from undrained triaxial and unconfined

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

4.2) Clay: (cont'd.) ...

compression tests ranged from 170 to 520 lbs./sq.ft. These results indicate, in generally, a soft consistency and, in places, a stiff consistency. It is felt that the field vane results are more reliable than the laboratory results for assessing the shear strength to be used in computations.

The sensitivity of the soil varies from 2.8 to 8.8 and lies, generally, between 4.0 and 6.0, indicating a sensitive clay. This is further borne out by the natural moisture contents which are very close to the liquid limits. Because of its higher sensitivity, the soil is susceptible to loss of strength due to disturbance.

Consolidation tests were carried out on two samples. They show that the soil is slightly overconsolidated (by 0.25 - 0.35 t.s.f.) and highly compressible ( $C_c = 1.4 - 1.7$ ).

4.3) Clayey Silt, Sand and Traces of Gravel:

This deposit was encountered only in borehole 1 which was terminated in this layer. The material consists of a mixture of red, clayey silt, sand and traces of gravel. The only Atterberg Limits test on this material gave the following values:

Liquid Limit .....	25%
Plastic Limit .....	14%
Natural Moisture Content .....	11%

The Standard Penetration Test gave 'N' values ranging from 70 blows per foot to 150 blows for 4-1/2 inches, indicating a very hard consistency.

5. GROUNDWATER CONDITIONS:

..... 5

The water level observed at the time of investigation, was as follows:

..... 5

5. GROUNDWATER CONDITIONS: (cont'd.) ...

Upstream of the bridge : El. 644.4

Downstream of the bridge : El. 643.2

The water level in the boreholes ranged from El. 643.8 to El. 645.9, except in borehole 3, where it was at ground level, elevation 648.7, because of the presence of perched water. Therefore, it may be assumed that the groundwater level in the vicinity of the creek, is equal to or slightly higher than the prevailing water level in the creek.

Slight artesian pressure was observed in borehole 1, when the underlying clayey silt and sand layer was intersected.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to replace the existing bridge with a new structure. Two alternative types of structures have been proposed:

- 1) Multi-plate pipe arch culvert
- 2) Multi-span timber bridge

The new grade will be some 6 ft. higher than the existing grade, resulting in a maximum height of the embankment of about 16 ft. above the river bed.

6.2) Approaches, Scour Protection and Dewatering:

Because of the low shear strength of the underlying clay layer, stability problems exist for embankment heights in excess of 10 ft., as measured from the river bottom. For embankments in excess of 10 ft., therefore, berms will be required to ensure stability.

Total stress analyses were carried out to determine the length of the berm required for stable 2:1 forward slopes of the approaches with the required 6-ft. additional fill. The soil

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

6.2) Approaches, Scour Protection and Dewatering: (cont'd.) ...

properties used in the foregoing analyses are as follows:

Angle of Friction	- fill	: $\phi$ = $35^{\circ}$
Undrained Shear Strength	- subsoil	: $C$ = 340 p.s.f.
Bulk Density	- fill	: $\gamma$ = 120 p.c.f.
Bulk Density	- subsoil	: $\gamma$ = 100 p.c.f.
Depth of Tension Crack		: = 6 ft.

It was computed that, in order to ensure the stability of 2:1 forward slopes for the proposed 16-ft. high approaches, it would be necessary to provide mid-height berms 30 ft. in length. Berms would be needed for side slopes, also, wherever the total height of embankment above the original ground level is more than 10 ft. For every 1-ft. height of embankment in excess of 10 ft., the length of berm required would be 5 ft. For details regarding the berms, see Fig. 1.

It has been reported by the Senior Soils Engineer, Northern Region, that in this area, a contractor had a granular stockpile fail after it was built to a height of 20 ft.

The forward slopes will be stable only if the designed section is maintained all the time. Any scour of the toe of the slopes, or the waterway, will endanger their stability. Therefore, it is recommended that adequate protection should be provided against scour of the slopes, or the waterway. Failure of the river bank, because of scour, is clearly evident immediately south of the existing bridge.

Because it is proposed to raise the grade by about 6 ft., some settlements are expected to take place under the fill. It is anticipated that the maximum settlement behind the abutment will be about 8 inches.



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

6.2) Approaches, Scour Protection and Dewatering: (cont'd.) ...

If it is decided to construct a multi-plate arch culvert, a dewatering scheme will be necessary, although no major dewatering problems are foreseen. On the other hand, if a multi-span timber bridge is to be constructed, supported on piles, no dewatering scheme will be required.

6.3) Bridge:

As stated in the previous sub-section, the stable 2:1 forward slopes will require 30-ft. long berms. This would mean constructing a relatively long multi-span bridge.

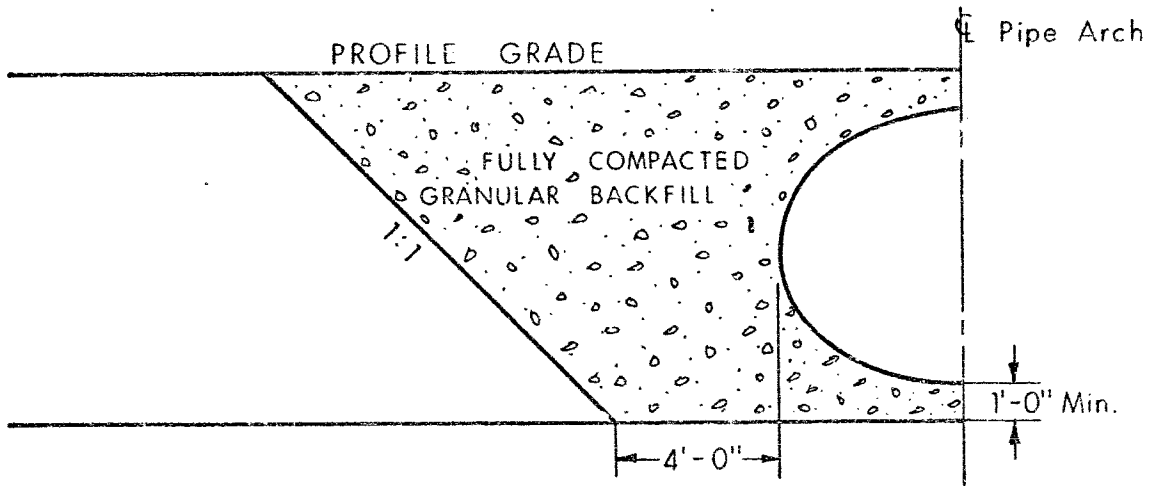
Since the underlying clay is unable to provide adequate bearing capacity because of its low shear strength, a spread footing type foundation is considered to be unsuitable. Therefore, it is recommended that the entire structure be supported by means of H-piles driven to approximate El. 465 into the hard silty clay stratum. The allowable maximum load for design purposes, should be 70 tons per pile in the case of 12 BP @ 53. If it is decided to construct a trestle bridge, then the single piles should be utilized as vertical columns, in which event, no dewatering scheme will be necessary to construct the pile caps below the water level. The portion projecting above the river bed should be encased in concrete for aesthetic purposes.

6.4) Multi-Plate Pipe Arch Culvert:

If it is decided to construct a pipe arch culvert at this site, then it is recommended that the pipe should be founded on at least a 1-ft. thick, compacted granular pad. The granular pad can be placed at or below El. 640.0. The granular backfill should be placed against the sides, as shown on the following page:

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

6.4) Multi-Plate Pipe Arch Culvert: (cont'd.) .....



Since the underlying soil is very soft, it will be necessary to provide berms in order to achieve stable side slopes. A diversion of the stream will be required on the south side. The spoil should be placed in the old stream bed.

7. MISCELLANEOUS:

The field work for this project was carried out during the period November 2 to 5, 1968, under the supervision of Mr. A. Prakash, Project Foundation Engineer, who also prepared this report.

The equipment used was owned and operated by Canadian Longyear Co. Ltd.

This report was reviewed by Mr. K. G. Selby, Supervising Foundation Engineer.

March 1969.

ASTORIA I

\*\*\*\*\*

DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

MATERIALS &amp; TESTING DIVISION

JOA 68-F-77

LOCATION Sta. 282 + 14 o/s 17<sup>th</sup> Lt.

ORIGINATED BY AP

W P 263-66-02

BORING DATE Nov. 2 - 3, 1968

COMPILED BY \_\_\_\_\_ AP

DATUM \_\_\_\_\_ Geodetic

BOREHOLE TYPE Washboring, NY Casing & Cone

CHECKED BY                     

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 68-F-77

LOCATION Sta. 282 + 00 o/s 49' Lt.

ORIGINATED BY AP

W P 263-66-02

BORING DATE Nov. 4, 1968

COMPILED BY            AP

DATUM Geodetic

BOREHOLE TYPE Washboring & NX Casing

CHECKED BY                     

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

JOA 68-F-77

LOCATION Sta. 281 + 55 o/s 54' Lt.

ORIGINATED BY AP

W. P. 263-66-02

BORING DATE Nov. 4, 1963

COMPILED BY            AP

DATUM \_\_\_\_\_ Geodetic

BOREHOLE TYPE Washboring & NX Casing

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT _____	L IQUID LIMIT _____ W <sub>L</sub> PLASTIC LIMIT _____ W <sub>P</sub> WATER CONTENT _____ W	BULK DENSITY Y P C F.	REMARKS
ELEV DEPTH	DESCRIPTION	SIRAT PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F. + Field Vane o Unconfined	W <sub>P</sub> ———— W <sub>L</sub> WATER CONTENT % 25 50 75		
648.7 0.0	Ground Level									
	Clay Highly plastic Very soft to firm Red		1	SS	3	640	+3.8		94	
			2	TW	PM		+5.2			
			3	SS	-	630	+4.4			
			4	TW	PM		+3.4			
			5	SS	-		+5.1			
620.7 28.0	End of Borehole					620				
							0 15% strain at failure 10			

DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

JOB 68-F-77

LOCATION Sta. 282 + 71 o/e 18' Rt.

FOUNDATION SECTION

ORIGINATED BY AP

W.P. 263-66-02

BORING DATE Nov. 5, 1968

COMPILED BY AP

DATUM Geodetic

BOREHOLE TYPE Washboring, NY Casing & Cone

CHECKED BY

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 68-F-77 LOCATION Sta. 283+04 o/s 19' Rt. ORIGINATED BY AP  
 W P 263-66-02 BORING DATE Nov. 5, 1968 COMPILED BY AP  
 DATUM Geodetic BOREHOLE TYPE Washboring & NX Casing CHECKED BY AK

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY Y P.C.F.	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F. + Field Vane o Unconfined					WP	WL	W		
648.5	Ground Level						200	400	600	800	1000	25	50	75		
0.0	Clay Highly plastic Firm to soft  Red		1	SS	2	640				+8.8					94	▼ 643.8
			2	TW	PM		σ	+5.7								
			3	SS	-	630		+4.9								
			4	TW	PM			+4.0								
620.5			5	SS	-			+5.0								
28.0	End of Borehole					620										
							0 15 10	5 % strain at failure								





DEPARTMENT OF HIGHWAYS - ONTARIO

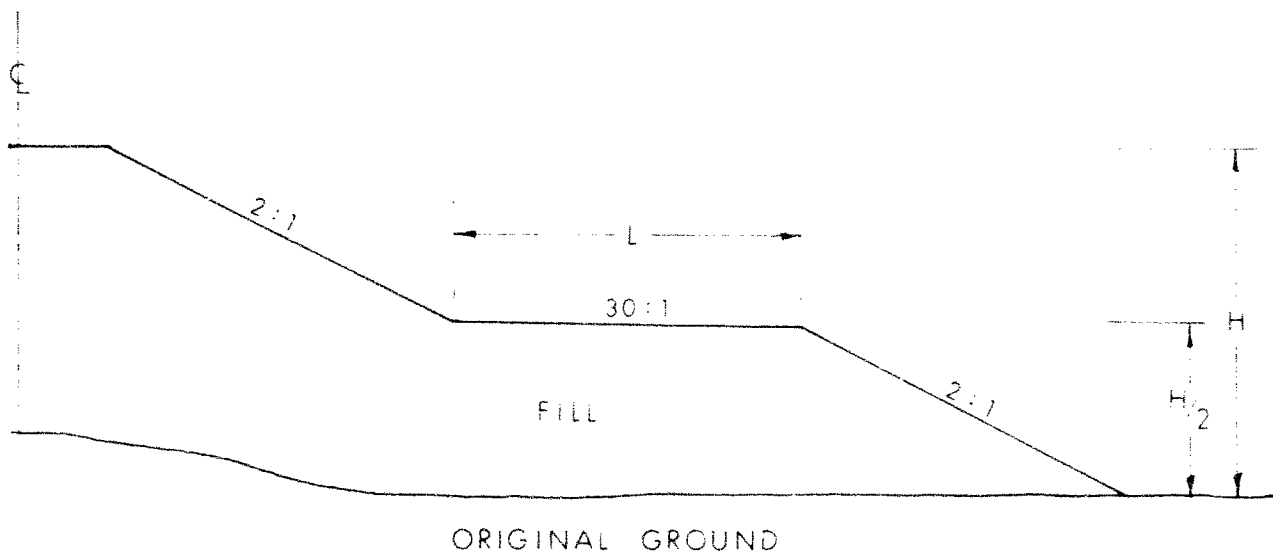
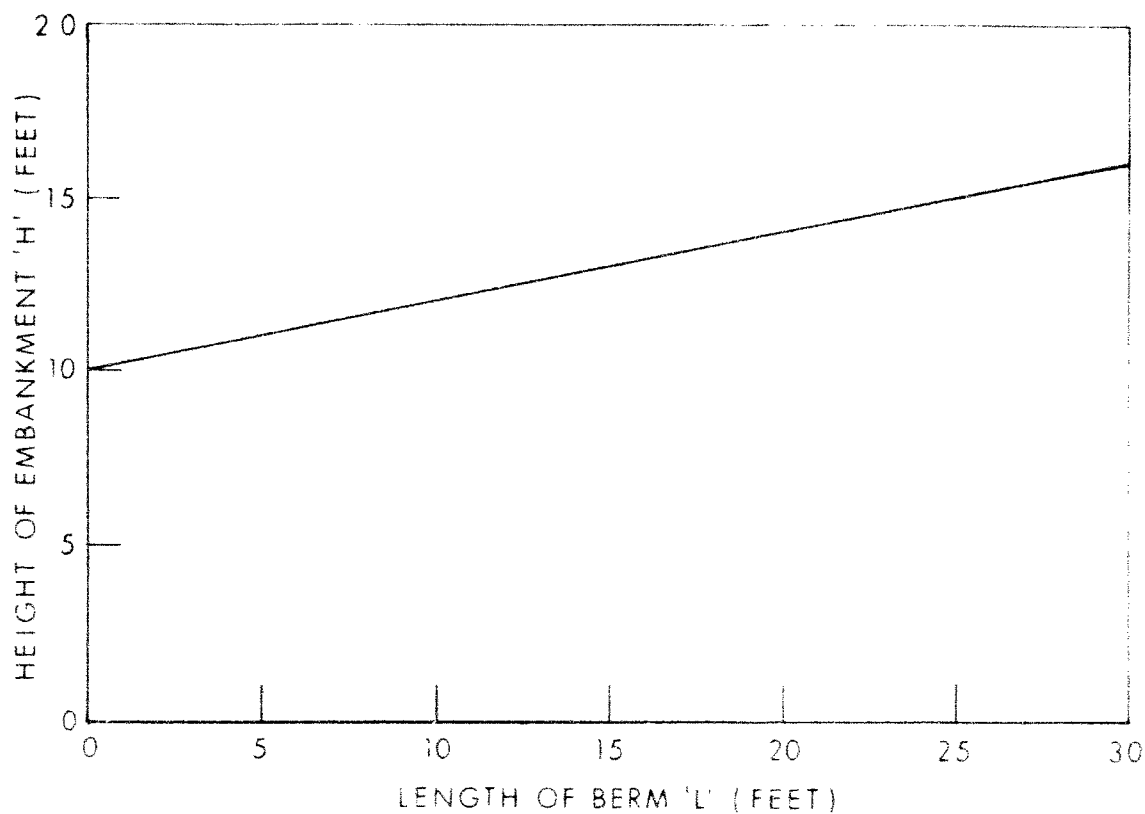
## MATERIALS &amp; TESTING DIVISION

RECORD OF BOREHOLE NO. 7

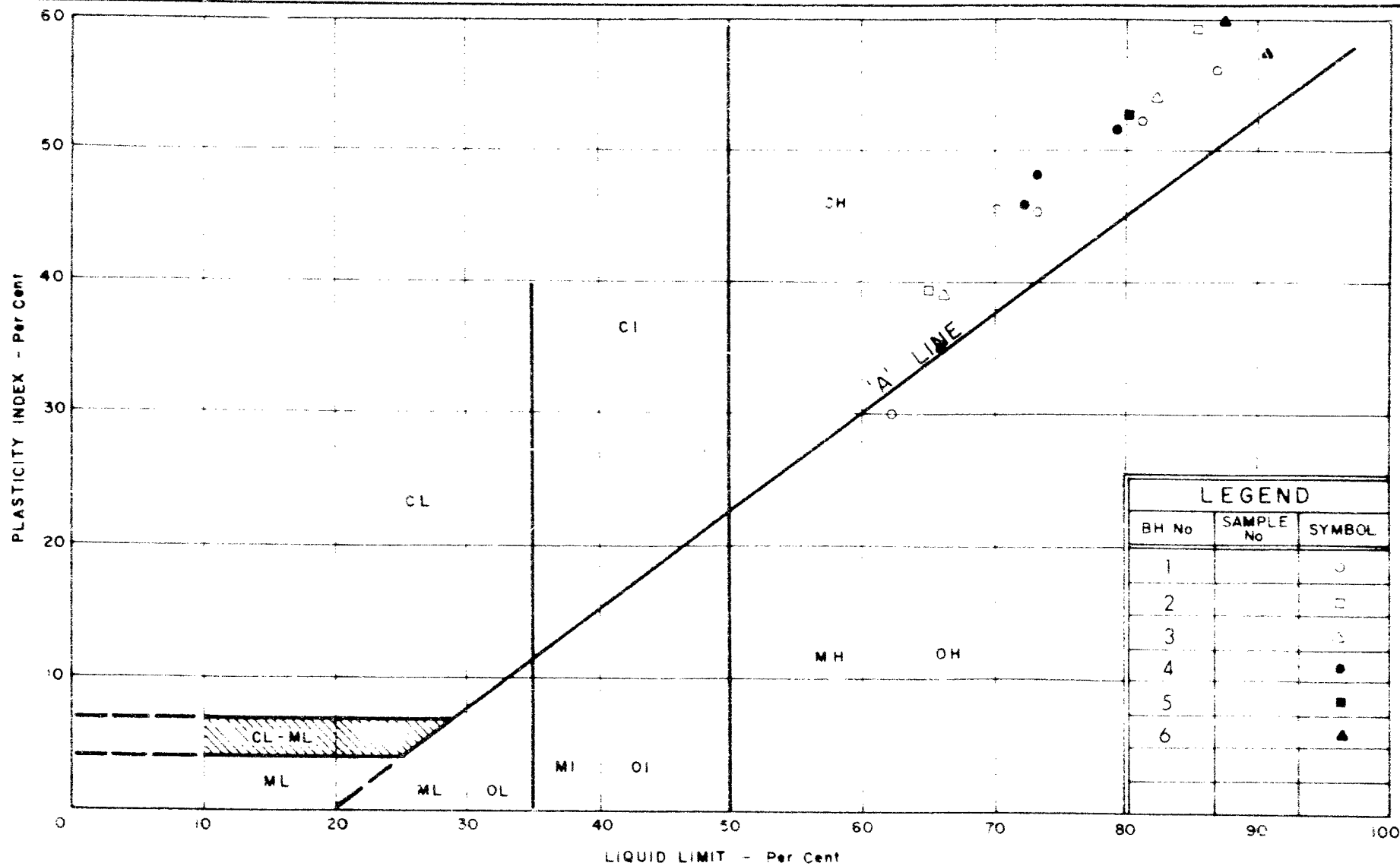
FOUNDATION SECTION

JOB	68-F-77	LOCATION	Sta. 61 + 50 o/s 12' Rt.	ORIGINATED BY	AP
W P	263-66-01	BORING DATE	Nov. 6, 1968	COMPILED BY	AP
DATUM	Geodetic	BOREHOLE TYPE	Washboring & NX Casing	CHECKED BY	<i>[Signature]</i>

[illegible]



TYPICAL SECTION THROUGH EMBANKMENT



DEPARTMENT OF HIGHWAYS  
 MATERIALS and  
 TESTING  
 DIVISION

# PLASTICITY CHART SENSITIVE CLAY

WP No 266 - 66 - 02  
 JOB No. 68 - F - 77

## 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 -

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
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

SS	SPLIT SPOON	TW	THINWALL OPEN
WS	WASHED SAMPLE	TP	THINWALL PISTON
SB	SCRAPER BUCKET SAMPLE	OS	CESTERBERG SAMPLE
AS	AUGER SAMPLE	FS	FOIL SAMPLE
CS	CHUNK SAMPLE	RC	ROCK CORE
ST	SLOTTED TUBE SAMPLE		
	PH	SAMPLE ADVANCED HYDRAULICALLY	
	PM	SAMPLE ADVANCED MANUALLY	

### SOIL TESTS

QU	UNCONFINED COMPRESSION	LV	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	FV	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

## ABBREVIATIONS USED IN THIS REPORT

### SOIL PROPERTIES

$\gamma$	UNIT WEIGHT OF SOIL (BULK DENSITY)
$\gamma_s$	UNIT WEIGHT OF SOLID PARTICLES
$\gamma_w$	UNIT WEIGHT OF WATER
$\gamma_d$	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
$\gamma'$	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
$\tau$	SHEAR STRENGTH
$c'$	EFFECTIVE COHESION INTERCEPT
$\phi'$	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
$c_u$	APPARENT COHESION
$\phi_u$	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
$\mu$	COEFFICIENT OF FRICTION
$S_t$	SENSITIVITY

### GENERAL

$\pi$	= 3.1416
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
$\sigma$	NORMAL STRESS
$\sigma'$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

### EARTH PRESSURE

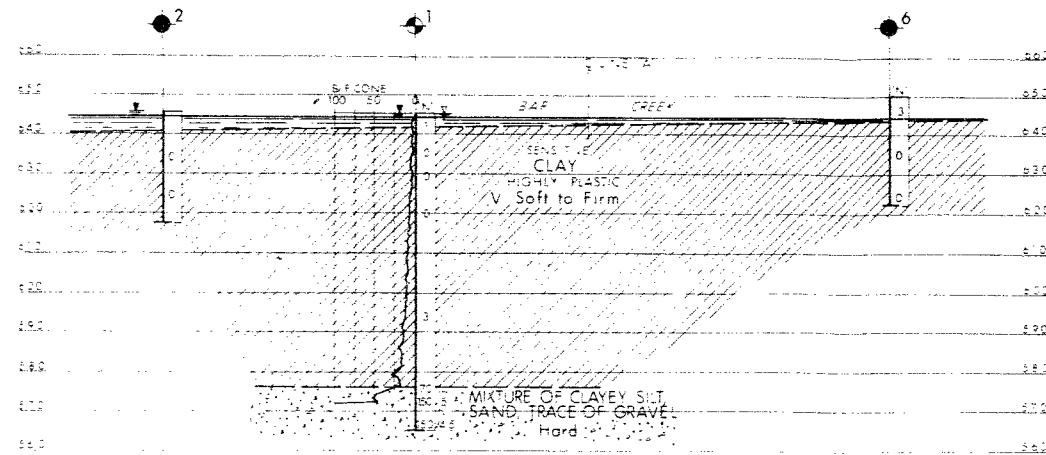
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
$K_0$	COEFFICIENT OF EARTH PRESSURE AT REST

### FOUNDATIONS

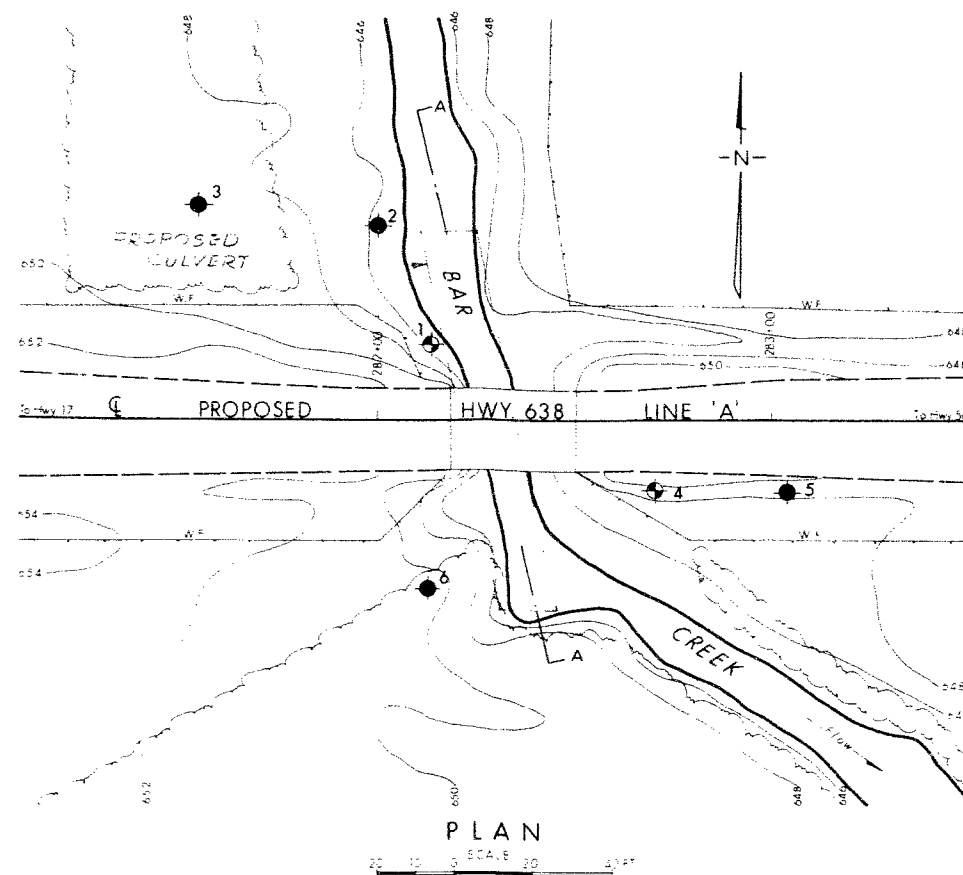
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
$k_s$	MODULUS OF SUBGRADE REACTION

### SLOPES

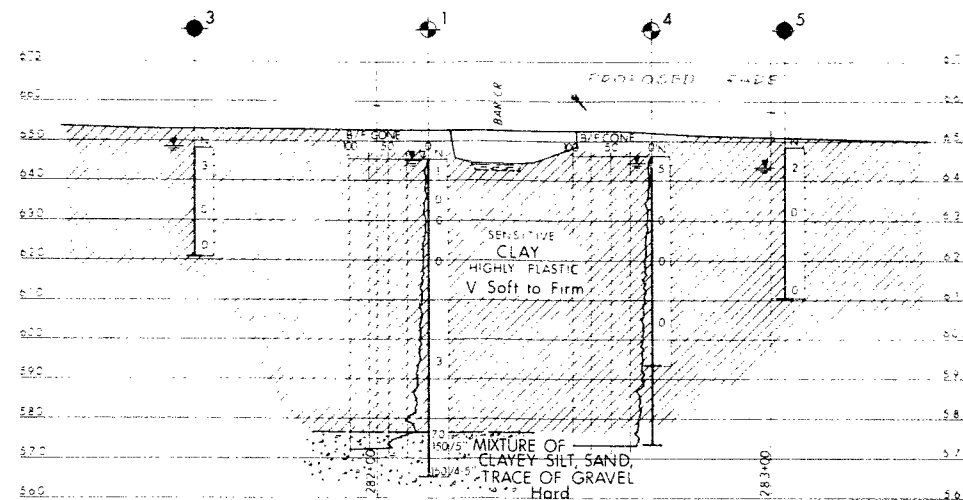
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
$\beta$	ANGLE OF SLOPE TO HORIZONTAL



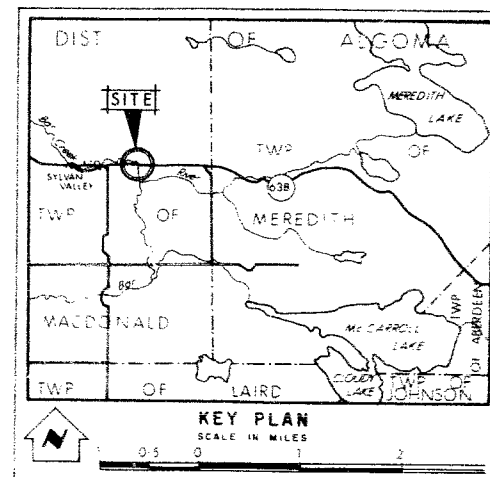
SECTION A-A



PLAN



PROFILE



LEGEND

- Bore Hole
- ⊕ Cone Penetration Hole
- ⊕ Bore & Cone Penetration Hole
- Water Levels established at time of field investigation. NOV. 1968
- Artesian Head

NO.	ELEVATION	STATION	OFFSET
1	645.8	282+14	15.0'
2	645.9	282+00	45.0'
3	648.1	281+55	54.0'
4	646.3	280+70	8.0'
5	648.5	283+04	19.0'
6	650.1	282+13	43.0'

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.

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

### BAR CREEK

KING'S HIGHWAY NO. 638 LINE 'A' DIST. NO. 18  
DIST. OF ALGOMA  
TWP. MACDONALD LOT CON.

### BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. A. F.	CHECKED BY	W.P. NO. 203-06-02	M.B.T. DRAWING NO.
DRAWN S. O.	CHECKER	JOB NO. 68-F-77	68-F-77A
DATE 16 JAN 1969	SITE NO.	BRIDGE DRAWING NO.	
APPROVED <i>[Signature]</i>	CONT. NO.		

1000000 00 9:30

D  
IBAR DOWN 2 MAY 29/70 955A VR  
S B DAVIDSON RGN BRIDGE PLANNING ENGR  
COPIES 100 B DAVIS BRIDGE OFF

S MCCOMBIE BRIDGE OFF

RE WP263-66-06-BRIDGE NUMBER 1

WP263-66-05-BRIDGE NUMBER 2

WP263-66-02-BRIDGE NUMBER 3

FOR ABOVE STRUCTURES RECOMMENDED DESIGN LOADS FOR TREATED TIMBER  
PILES ARE AS FOLLOWS.

BRIDGE NUMBER 1 AND BRIDGE NUMBER 2 ONE TON PER 3.0 FEET OF  
PENETRATION INTO ORIGINAL GROUND.

BRIDGE NUMBER 3 ONE TON PER 3.75 FEET OF PENETRATION INTO  
ORIGINAL GROUND. NOTE ALL SOFT ORGANIC SOIL MUST BE REMOVED  
AND REPLACED WITH SUITABLE GRANULAR FILL PRIOR TO DRIVING PILES.

X G SELBY SUPVR FOUND ENGR FOR A G STERMAC MAT AND TEST SECT

EB





Department of Highways Ontario

Copy for the information of

Foundation Office

Mr. A. Sternau,  
Principal Foundation Engineer,  
Room 107, Lab. Bldg.

C.S. Grebski,  
Bridge Office

March 30, 1971

W.P. 263-66-06 - Sylvan Valley Bridge #1 } 69-F-42  
W.P. 263-66-05 - Sylvan Valley Bridge #2 }  
W.P. 263-66-02 - Sylvan Valley Bridge #3 } 68-F-77  
Highway 638, District No. 18

Attached herewith we are submitting the final  
bridge drawings which show the foundation design for  
these structures.

Kindly give us your comments at your earliest  
convenience.

CSG:rd

C.S. Grebski,  
Bridge Design Engineer

Attach.

c.c. Foundation Office

COMMENTS:

Note for pile driving should read: "No. 14 creosoted timber piles  
driven to tip el. -- "

(Bridge #1 Tip El. 607.5)

(Bridge #2 Tip El. 608.0)

(Bridge #3 Tip El. 593.0)

Mention of design load might imply use of Hiley Formula which is  
not valid at this site.

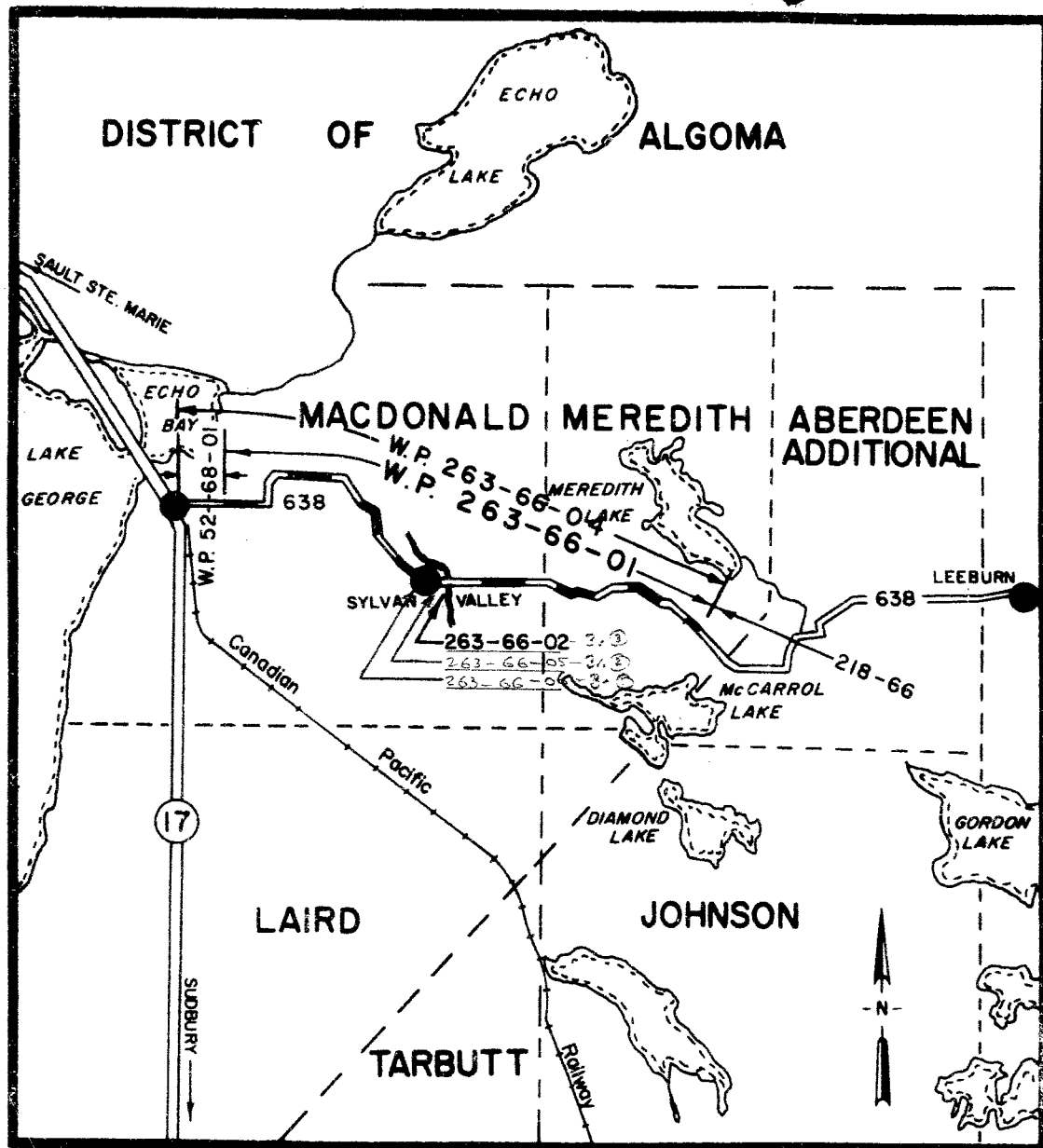
cc. A. Radkowski  
C. Grebski  
Found. Report

*Handwritten signature and date:*  
3 June 71

K. G. Selby,  
Supervising Foundation Engr.

April 15, 1971

*Handwritten signature:*  
K. G. Selby



Plan to accompany Design Criteria for W.P. 263-66-01

Scale 1" = 2 Miles

0022  
FEB 16 PM 2:48

68-F-77

MX TBAR FEB 16/70 238P

SAUL 3 R G GASCOYNE, DIST ENGR

DOWN 8 CC: K SELBY FOUNDATIONS MAT AND TEST

RE: WP 263-66-02 HIGHWAY 638, ECHO BAY NORTH

CORRECTION ON STRUCTURE REPORT W J 68F77, PAGE 7, 6.3 "H" PILES ARE  
TO BE DRIVEN TO ELEVATION 565 NOT TO ELEVATION 465 AS INDICATED IN  
THE REPORT.

PLEASE CORRECT YOUR REPORT ACCORDINGLY

R D GUNTER MAT AND TEST

JO