

G.I.-30 SEPT. 1976

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 40P4-9

DIST. 2 REGION Southwestern

W.P. No. \_\_\_\_\_

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W. O. No. 60-F-94c

STR. SITE No. \_\_\_\_\_

HWY. No. \_\_\_\_\_

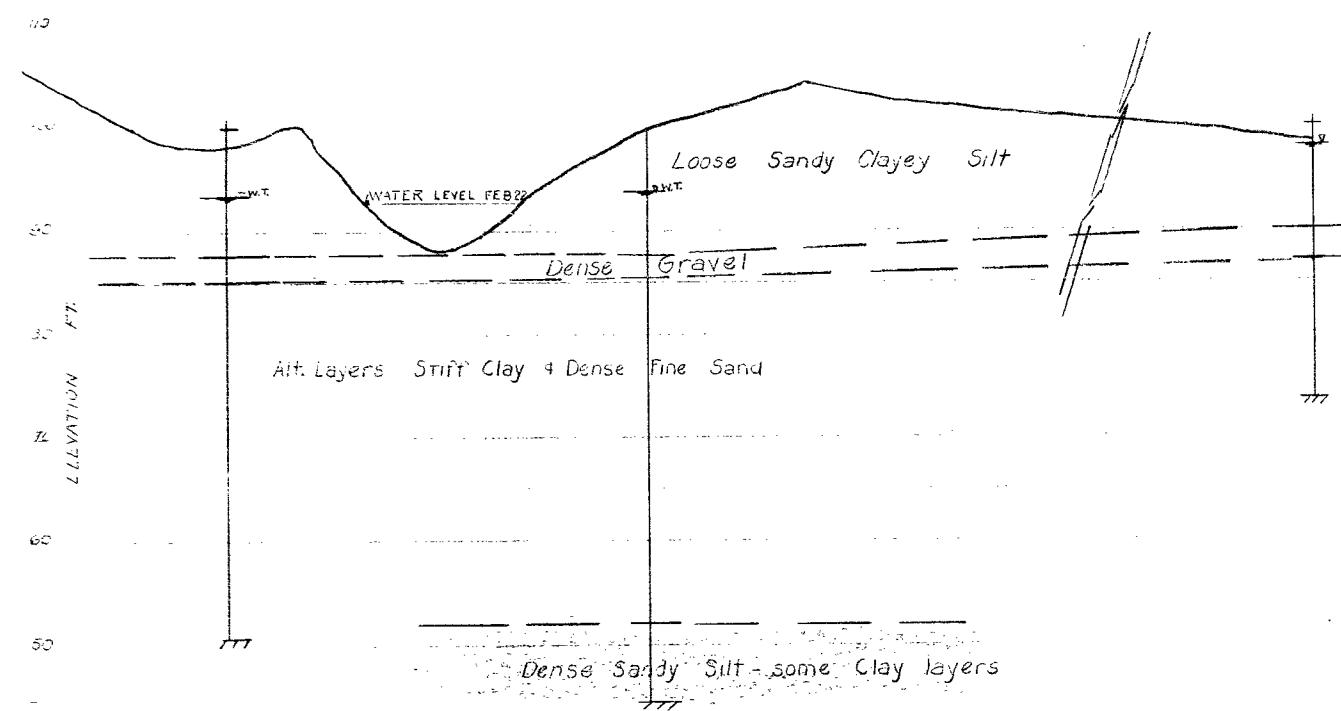
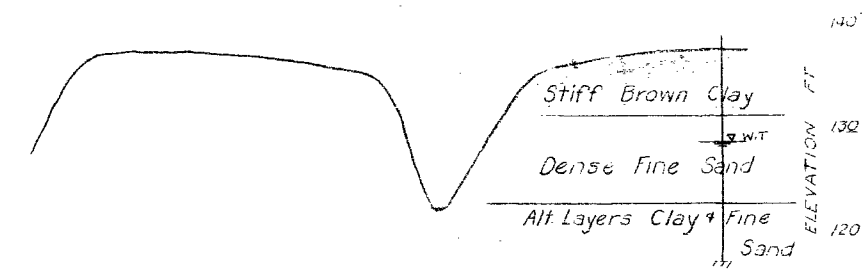
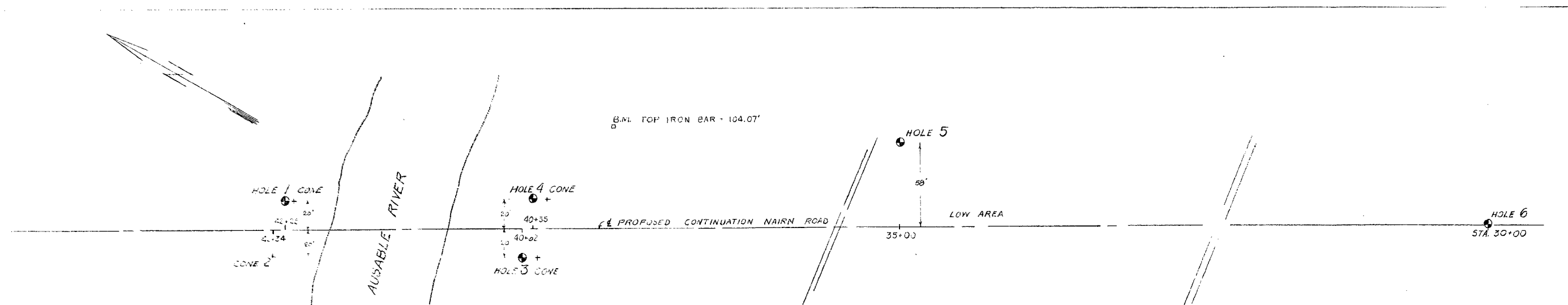
LOCATION PROP. COUNTY BRIDGE

NAIRN TWP

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 1

REMARKS: TO BE ADDED TO EXISTING

MICROFICHE [40P4-9]



PROPOSED AUSABLE RIVER CROSSING NAIRN, ONT.  
 BOREHOLE LOCATION PLAN AND ESTIMATED STRATIGRAPHY  
 William A. Trow & Assoc. Ltd. Feb 26/60 Scale 1" = 40'

PILE LOADING TEST  
COUNTY BRIDGE  
NAIRN, ONTARIO  
60-F-94  
21 NOV. 60

LOAD IN TONS

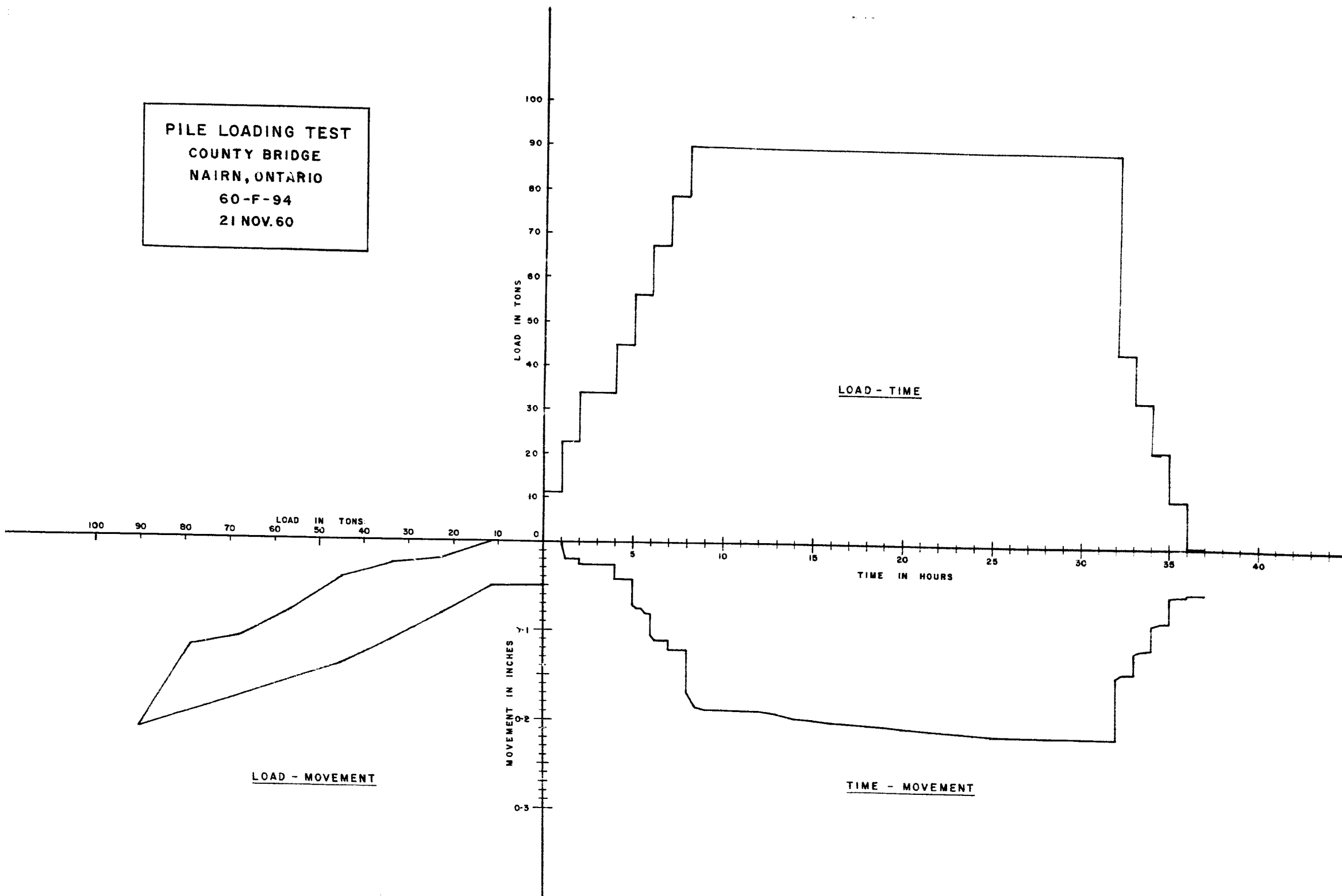
LOAD - TIME

TIME IN HOURS

MOVEMENT IN INCHES

LOAD - MOVEMENT

TIME - MOVEMENT



Mr. A. M. Toye,  
Bridge Engineer.

Materials & Research Section.

November 16, 1960.

FOUNDATION INVESTIGATION REPORT

by: H.Q. Golder & Associates, Ltd.

Attention: Mr. K. Kleinsteinber.

Re: Proposed County Bridge,  
Nairn, Ont., District #2.

Enclosed, for your information, are two  
copies of the soils investigation report, prepared by  
H. Q. Golder & Associates, Ltd., for the above site.

The remaining copies of this report have  
been forwarded as per the distribution list shown therein,  
on the first page.

*for Mr. de Looze*

LGS/MdeF  
Encls. (2)

L. G. Soderman,  
PRINCIPAL FOUNDATIONS ENGINEER

cc: Foundations Office ✓  
Gen. Files.

Amable

# C. C. PARKER AND ASSOCIATES LIMITED

LONDON HAMILTON EDMONTON

JOB No. 911 PAGE 1

Des' By J. Thaler Date July 25/62

Ckd. By Date

JOB McCUBBIN BRIDGE EXTENSION

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO.

13 12 11 10 9 8 7 6 5 4 3 2 1  
Z

Under side of Abut. Ftg.

Plumb Line

10 14 ft. Intervals

Table of Offsets from Plumb Line

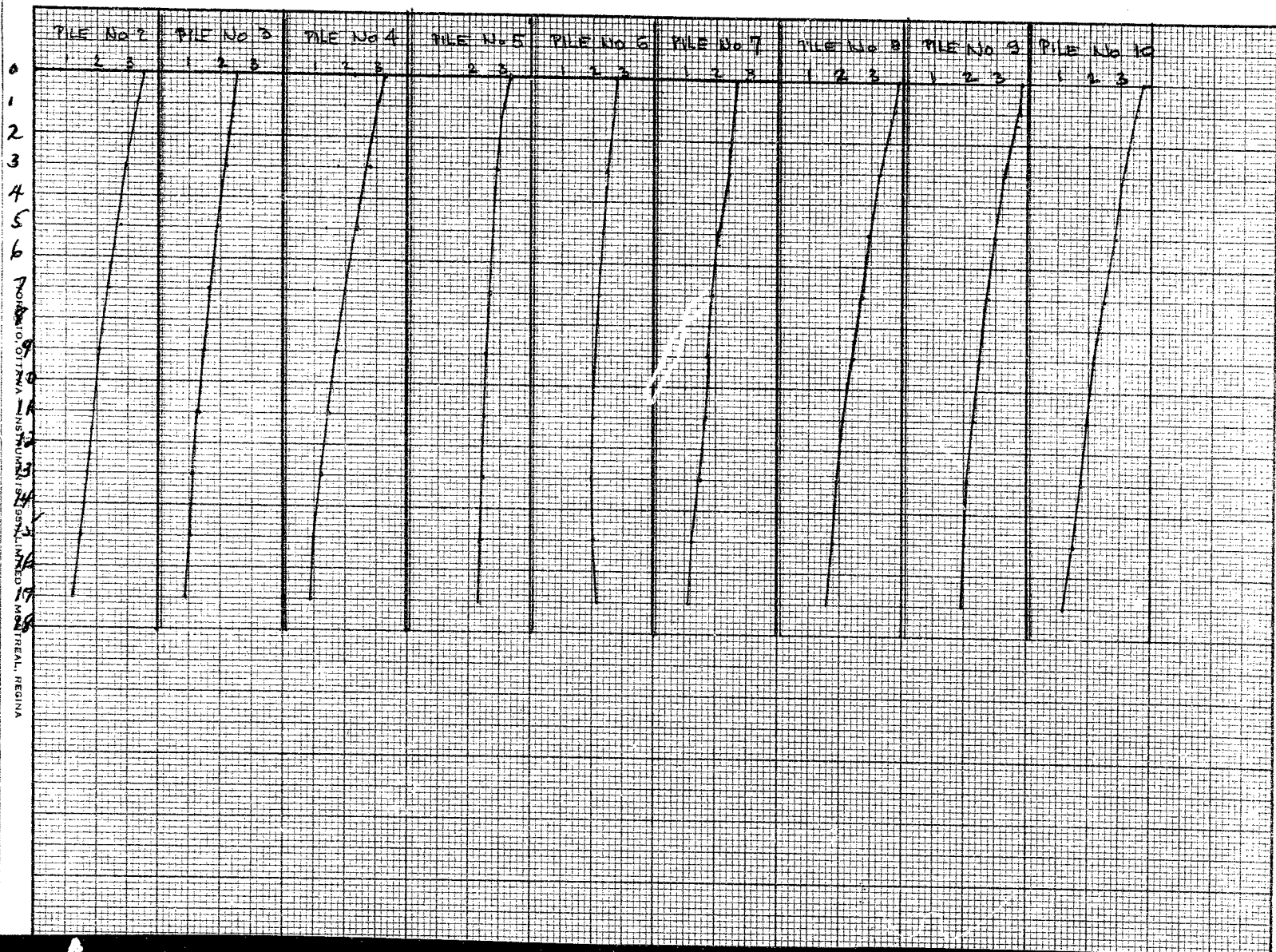
Pile Numbers

	2	3	4	5	6	7	8	9	10
0	3'-5	2'-5½	3'-3	3'-2½	2'-7½	2'-7¾	3'-9½	5'-8	3'-8½
1	3'-2½	2'-4½	3'-0	3'-1½	2'-6½	2'-6½	3'-8	3'-7	3'-6½
2									
3	2'-11½	2'-2	2'-8½	2'-11	2'-4½	2'-3¾	3'-3	3'-3½	3'-0
4									
5	2'-8	1'-10½	2'-4	2'-9	2'-3¾	2'-0½	2'-10½	3'-0	2'-10½
6									
7	2'-4	1'-7¾	2'-0	2'-7½	2'-2	1'-10¼	2'-7	2'-8¾	2'-6½
8									
9	2'-0¾	1'-5½	1'-8	2'-5½	2'-1	1'-8¼	2'-4	2'-6	2'-2½
10									
11	1'-9½	1'-3½	1'-5	2'-5	2'-0½	1'-7	2'-1	2'-4	1'-11
12									
13	1'-7½	1'-1½	1'-3	2'-5	2'-0	1'-5	1'-11	2'-2½	1'-8
14									
15	1'-5½	1'-0½	1'-0	2'-4	2'-1	1'-3½	1'-9	2'-0¾	1'-6
16									
17	1'-2½	0'-11	0'-9½	2'-3	2'-1½	1'-2½	1'-7	1'-11½	1'-3½
18									

Note: Offsets taken perpendicular to face of Abutment

RELATION TO PLUMBLINE OF FRONT PILING  
WEST ABUTMENT FTG.

McCUBBIN BRIDGE EXTENSION



FOR INFO. OF THE INSURANCE COMPANY, THE FOLLOWING INFORMATION IS BEING FURNISHED TO YOU FOR YOUR RECORDS.

**H. Q. GOLDER & ASSOCIATES LTD.**

**CONSULTING CIVIL ENGINEERS**

**H. Q. GOLDER  
V. MILLIGAN**

**2446A BLOOR ST. W.  
TORONTO 9  
RO. 7-9201**

**REPORT  
TO  
DEPARTMENT OF HIGHWAYS, ONTARIO  
ON  
SITE INVESTIGATION, PROPOSED COUNTY BRIDGE  
NAIRN, ONTARIO**

**Distribution:**

- 2 copies - Mr. S. B. Arnold, Engineer,  
County of Middlesex,  
London, Ontario.**
- 4 copies - Department of Highways, Ontario,  
Toronto, Ontario.**
- 1 copy - Mr. T. S. Caldwell,  
Department of Highways, Ontario,  
London, Ontario.**
- 1 copy - M. M. Dillon & Company Limited,  
London, Ontario.**
- 2 copies - H. Q. Golder & Associates Ltd.,  
Toronto, Ontario.**

**October, 1960**

**6018**

### ABSTRACT

This report covers the results of an investigation of foundation conditions at the pier locations for a bridge which is currently under construction across the Ausable River at Nairn, Ontario. The purpose of the investigation was to determine the soil conditions at the exact pier locations and to review the foundation design criteria for the structure.

The results of the field investigation generally confirmed the soil conditions determined previously by a preliminary investigation. In light of these the foundation design criteria are considered to be satisfactory.

Recommendations are made concerning the construction of the east pier of the bridge, and it is recommended that the sheeting used in the cofferdams for both piers be left in place as scour protection.



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## INTRODUCTION

1.

H. Q. Golder and Associates have been authorized by the Department of Highways, Ontario to investigate soil conditions at the pier locations for the proposed County Bridge crossing the Ausable River at Nairn, Ontario. The purpose of the investigation was to confirm the soil conditions at the bridge pier locations and to review the foundation design criteria for the structure.

## PROCEDURE

Two boreholes in BX size were put down between September 29th, 1960 and October 3rd, 1960, one at each pier location, using a standard skid-mounted machine drillrig. The samples obtained in these boreholes were returned to Toronto for laboratory testing, and those remaining after testing will be stored until March 31st, 1961 at which time you will be notified regarding their disposal.

All elevations in this report are referred to a previously established local datum.

The results of the field investigation are plotted on the Records of Boreholes at the end of this report. The results of the laboratory testing are plotted on the borehole logs and on the figures, also at the rear of the report. Borehole locations and the inferred soil stratigraphy at the site are shown on Drawing 1.

From a preliminary foundation investigation by William A. Trow and Associates it was known that in the vicinity of the bridge the site is underlain by a stratum of loose clayey silt followed by a thin layer of dense gravel, and then a stratum of dense stratified silt and clay. The results of our investigation generally confirmed this stratigraphy. A detailed description of the various strata encountered follows:

Organic Clayey Silt

The uppermost stratum at Boreholes 1 and 2 was found to be a brown sandy, clayey silt containing appreciable organic matter, principally in the form of decayed vegetation. The thickness of the stratum was about 3 feet in Borehole 1, and 2 feet in Borehole 2. The consistency of this stratum was estimated to be soft to firm.

Grey-Brown Sand and Gravel

Underlying the silt is a stratum of grey-brown silty sand and gravel containing considerable fresh-water shells, some organic matter, and a trace of clay. As indicated by the previous investigation, this stratum is very thin with a thickness of about 3 feet in Borehole 1 and about 5 feet in Borehole 2.

Standard penetration resistances of 2 in Borehole 1 and 15 in Borehole 2 indicated that the relative density of the stratum can vary from very loose to compact.

Stratified Silt and Clay

The same stratum of grey and brown stratified silt and clay encountered beneath the sand and gravel in the previous investigation was encountered throughout the remainder of each of the boreholes to a depth of 72 feet in Borehole 1 and 62 feet in Borehole 2.

The individual layers or stratifications vary widely in composition, but are composed predominantly of either silty clay or sandy silt. Grain size distributions for the non-plastic silt layers are shown on Figure 1 and the Atterberg limits for the clay layers are plotted on the borehole logs. The liquid limits range from 29.3 to 34.4 with an average of 31.6 per cent and the plastic limits range from 16.4 to 20.8 with an average of 18.7 per cent. Natural water contents of the clay layers vary generally from 18.2 to 25.2 with an average of 21.8 per cent. Wet unit weights ranged from 121 to 129 with an average of 125 pounds per cubic foot.

The individual layers vary randomly and widely in thickness from  $\frac{1}{4}$  inch to a foot or more. Specifically, it was observed that in the upper 20 feet of Borehole 2 the clay layers predominated whereas in Borehole 1 the silt layers formed the greater proportion of the stratum.

Stratified Silt and Clay (continued)

Six unconfined compression tests were carried out on some of the thicker clay layers and shear strengths of 1,260 to 3,650 with an average of 2,200 pounds per square foot were measured. However, axial strains at failure ranged from 13 to 20 per cent suggesting considerable sample disturbance. Stress-strain curves for these tests are shown on Figure 2.

The standard penetration resistances obtained in the stratum ranged from 21 to 91 with an average of 37 blows per foot. These, together with the shear strengths quoted above indicate that the relative density of the silt layers varies from compact to very dense but is generally dense, and the consistency of the clay layers varies from stiff to very stiff and is generally very stiff.

Groundwater

Groundwater levels in the boreholes at the time of the investigation corresponded approximately to river level which was at about elevation 89.

DISCUSSION

The proposed bridge, which is presently under construction, will consist of 3 structural steel spans with a reinforced concrete deck. The abutments are of the spill-through type and will be founded on 12 inch diameter steel pipe piles driven to

refusal in the stratified silt and clay. The design load for the piles is 45 tons subject to the results of a pile loading test to be carried out during construction. The two piers are to be founded on spread footings in the stratified silt and clay at about elevation 80.5 with net design loads of about 3,000 pounds per square foot.

In view of the shear strengths quoted above, these design values are reasonable and require no comment except that the specification of a load test for the abutment piles is a wise precaution.

With regard to the construction of the piers, it is understood that the contractor has been required to drive a steel sheet pile cofferdam to protect the excavation for the footings. It is further understood that for the west pier the cofferdam sheeting has been driven to practical refusal at about 2 to 3 feet below proposed footing level and that the excavation has been successfully completed to grade. A copy of a letter to Mr. L. G. Soderman dated October 26th, 1960, giving recommendations concerning cofferdam excavations at both west and east piers is enclosed in Appendix I to this report.

For the construction of the east pier, where the results of Borehole 1 indicated that the material in the upper 20 feet of the stratum is predominantly silt, it is recommended that the cofferdam sheeting be driven to a depth of 6 to 8 feet below

DISCUSSION (continued)

6.

footing level in order to prevent a possible blow due to an unbalanced hydrostatic head of about 10 feet. If the minimum penetration of 6 feet cannot be attained due to the density of the substratum, it is requested that we be contacted before the contractor proceeds with the excavation.

In the case of both piers it is recommended that the steel sheeting be left in place following construction in order to provide additional scour protection for the piers. With respect to this, it is considered that the present penetration of the sheet piling at the west pier is sufficient and need not be increased.



A. A. Gass, P. Eng.



H. Q. Golder, P. Eng.

AAG:IMB  
6018

October, 1960

APPENDIX I



APPENDIX I

C O P Y

October 26th, 1960.

Department of Highways,  
Materials and Research Section,  
Parliament Buildings,  
Toronto 2, Ontario.

Attention: Mr. L. G. Soderman,  
Principal Foundation Engineer

Re: Pier Construction,  
Nairn Bridge,  
Nairn, Ontario.

Dear Sirs:

We were contacted on October 25th, 1960 by Mr. Stermac of your office regarding a problem which had arisen in connection with the construction of the west pier of the above bridge, and, at his request, we called Mr. J. H. Kearney of M. M. Dillon & Company Limited in order to discuss the problem directly. The purpose of this letter is to inform you of the results of this discussion.

According to Mr. Kearney the steel sheet pile cofferdam for the west pier had been driven to about 2 feet below footing level, the excavation had been completed, and the contractor was prepared to pour the concrete. Mr. Kearney wondered whether the sheet piling was to be left in place permanently following construction of the pier, and if it was, whether the 2 foot penetration of the piling below footing level was sufficient. We stated that it was our recommendation that the sheeting be left in place as scour protection for the pier and that the 2 feet of penetration was, in our opinion, sufficient. In addition, we suggested that, before the pier footing was poured, the excavation be inspected to see whether there were any signs of disturbance of the underlying soil due to seepage of water into the bottom of the cofferdam.

The construction of the east pier was also discussed. The boring which we recently put down at this pier indicated that the soil was predominantly silt at foundation level as opposed to the west pier where there was considerably more clay. We recommended to Mr. Kearney that the cofferdam sheeting be driven to a penetration of at least 6 and preferably 8 feet below footing level in order to protect against a possible blow due to unbalanced

GOLDER & ASSOCIATES

Department of Highways,  
October 26th, 1960,  
Page 2.

C O P Y

hydrostatic head. If the contractor finds it impossible to drive the sheeting to the minimum recommended penetration, we requested that we be informed before the excavation is carried out.

The above points will be covered in detail in our report for the investigation which we recently carried out at the pier locations, but, if you have any questions regarding them, we would be pleased to discuss them with you.

Yours very truly,

H. Q. GOLDER & ASSOCIATES LTD.

A. A. Gass.

AAG:IMB

cc: Mr. J. K. Kearney,  
M. M. Dillon & Company Limited,  
144 Maple Street,  
London, Ontario.

Mr. S. B. Arnold,  
Engineer, County of Middlesex,  
County Building,  
London, Ontario.

GOLDER & ASSOCIATES

## ABBREVIATIONS

The standard abbreviations commonly employed on each "Record of Borehole", on the figures, and in the text of the report are as follows:

## SAMPLE TYPES

A.S. - Auger Sample	H.C. - Rock Core
C.S. - Chunk Sample	S.T. - Slotted Tube
D.O. - Drive Open	T.O. - Thin-walled, Open
D.S. - Denison Type Sample	T.P. - Thin-walled, Piston
P.S. - Poff Sample	W.S. - Wash Sample

## PENETRATION RESISTANCES

**Dynamic Penetration Resistance** - The energy required to drive a 2 inch diameter, 60 degree cone attached to the end of the drilling rods into the ground; expressed in blows per foot, where each blow represents 4200 inch-pounds of energy.

**Standard Penetration Resistance, N** - The number of blows by a 140 pound hammer dropped 30 inches required to drive a 2 inch drive open sampler one foot into the ground.

$W_h$  - Sampler advanced by static weight of sampling hammer

$P_h$  - Sampler advanced by an hydraulic pressure

$P_m$  - Sampler advanced by leveraging on drill rods

## SOIL DESCRIPTION

The standard terminology for the descriptions of the consistency of cohesive soils and the relative density of cohesionless soils is as follows:

<u>Relative Density</u>	<u>N, Blows/ft.</u>	<u>Consistency</u>	<u>C, lb./sq.ft.</u>
Very Loose	0 to 4	Very Soft	30 to 250
Loose	4 to 10	Soft	250 to 500
Compact	10 to 30	Firm	500 to 1,000
Dense	30 to 50	Stiff	1,000 to 2,000
Very Dense	over 50	Very Stiff	2,000 to 4,000
		Hard	over 4,000

## SOIL TESTS

C - Consolidation Test	Q - Undrained Triaxial
H - Hydrometer Analysis	Qc - Consolidated Undrained Triaxial
M - Sieve Analysis	S - Drained Triaxial
MH - Combined Analysis, Sieve and hydrometer	U - Unconfined Compression
	V - Field Vane Test

Note: Undrained triaxial tests in which pore pressures are measured are shown as  $Q'$  or  $Q'_c$ .

## SOIL PROPERTIES

$\gamma$ - Total Unit Weight	$k$ - Coefficient of Permeability
$\gamma_d$ - Dry Unit Weight	$c$ - Undrained Shear Strength
$\gamma_t$ - Submerged Unit Weight	( $\pm$ Compressive Strength)
LL - Liquid Limit	$S_t$ - Sensitivity
PL - Plastic Limit	$\phi'$ - Effective Angle of Shearing Resistance
w - Natural Water Content	$c'$ - Effective Cohesion Intercept
G - Specific Gravity	$C_c$ - Compression Index
e - Void Ratio	$C_v$ - Coefficient of Consolidation

## RECORD OF BOREHOLE I

LOCATION SEE DRWG. No. 1

BORING DATE SEPT. 29 - OCT. 1, 1960

DATUM

LOCAL

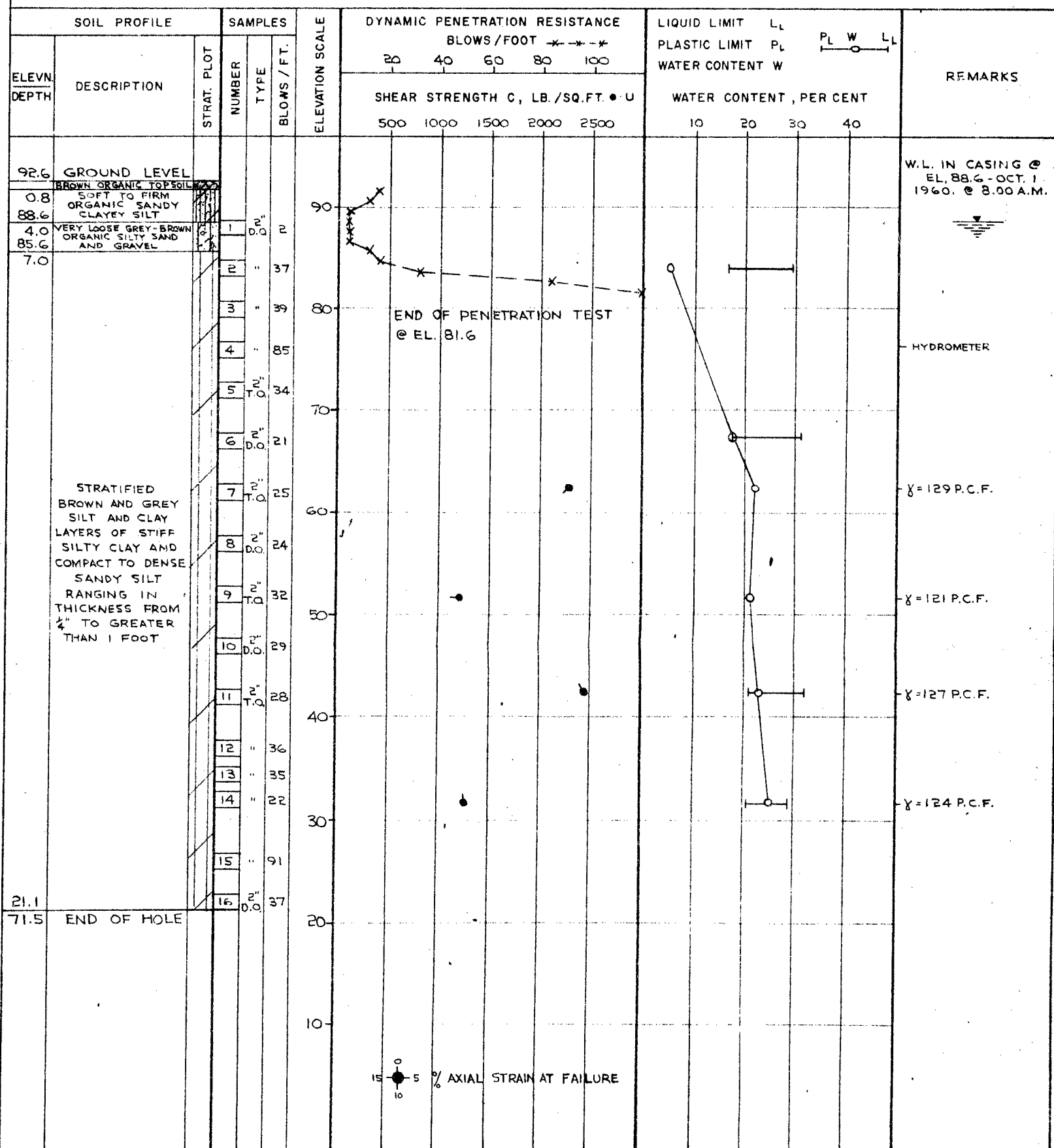
BOREHOLE TYPE WASHBORING

BOREHOLE DIAMETER

BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



(a) Dynamic penetration resistance converted to 4200 inch lb. energy

(b) Abbreviations listed on page 7

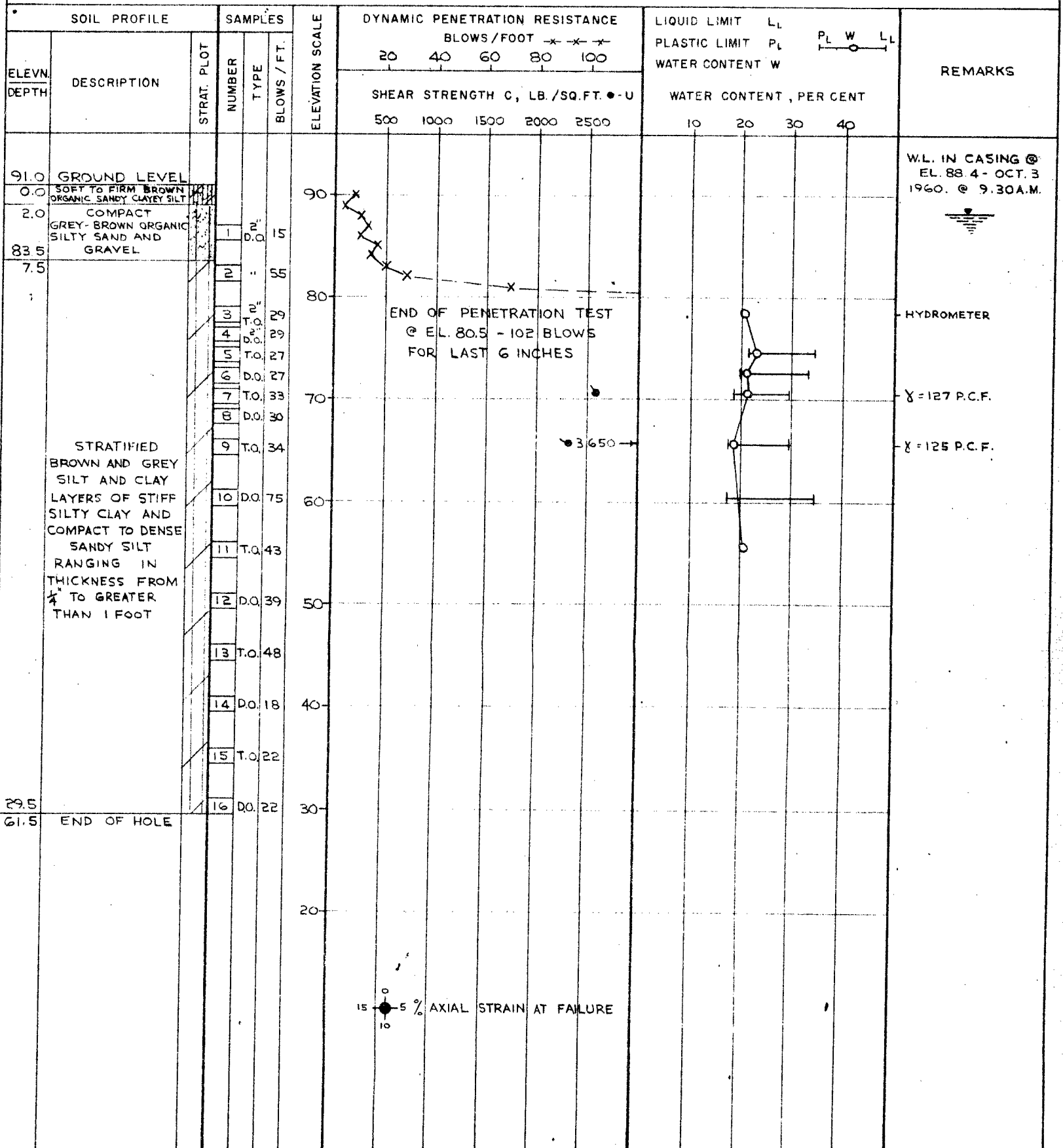
VERTICAL SCALE  
1 INCH TO 10 FEET

GOLDER &amp; ASSOCIATES

DRAWN J.A.  
CHECKED *ms*

# RECORD OF BOREHOLE 2

LOCATION SEE DRWG. No. 1 BORING DATE OCT. 1 - OCT. 3 1960 DATUM LOCAL  
 BOREHOLE TYPE WASHBORING BOREHOLE DIAMETER BX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



(a) Dynamic penetration resistance converted to 4200 inch lb. energy  
 (b) Abbreviations listed on page 7

VERTICAL SCALE  
 1 INCH TO 10 FEET

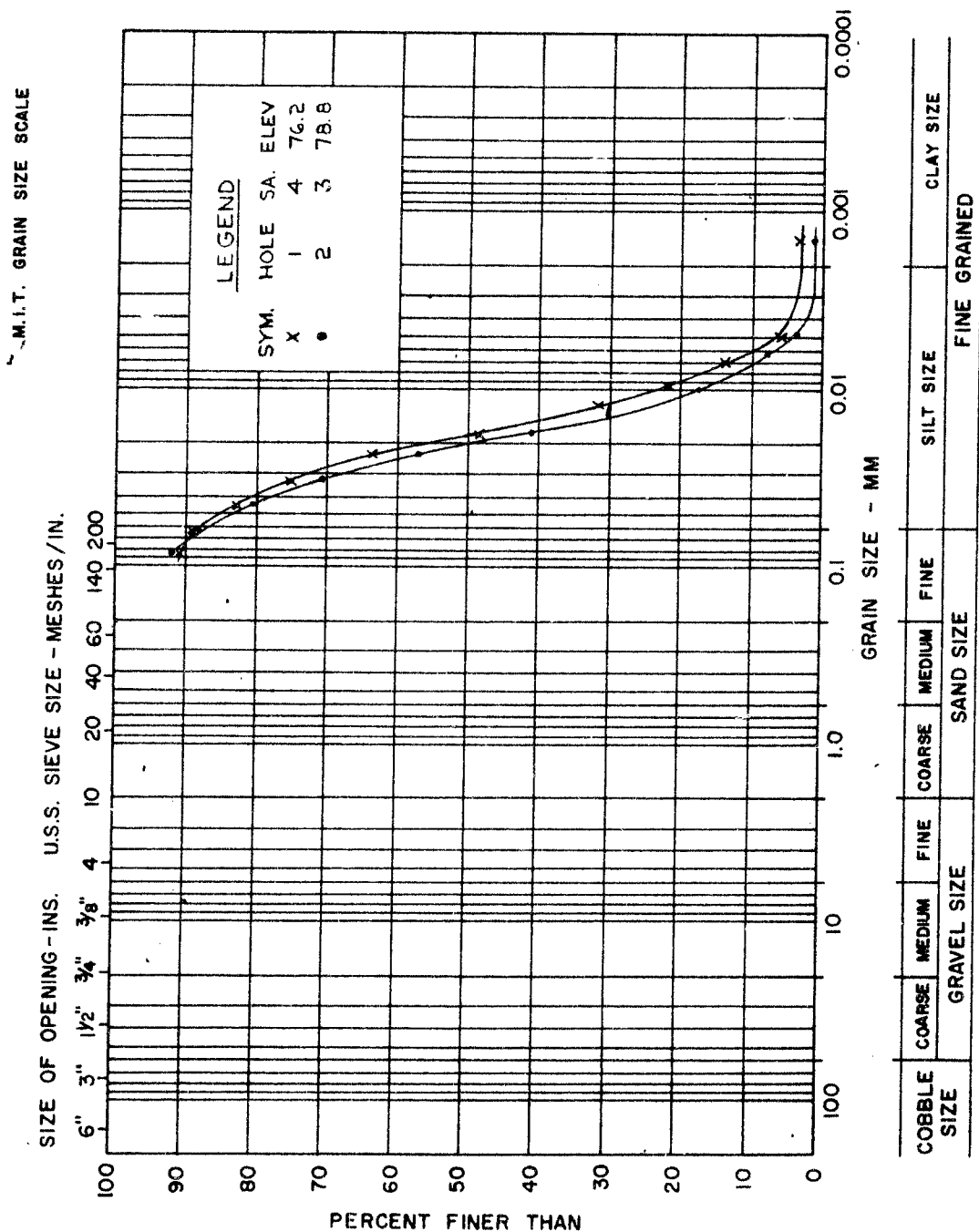
GOLDER & ASSOCIATES

DRAWN J.A.  
 CHECKED *ab*

## GRAIN SIZE DISTRIBUTION

FIGURE

I

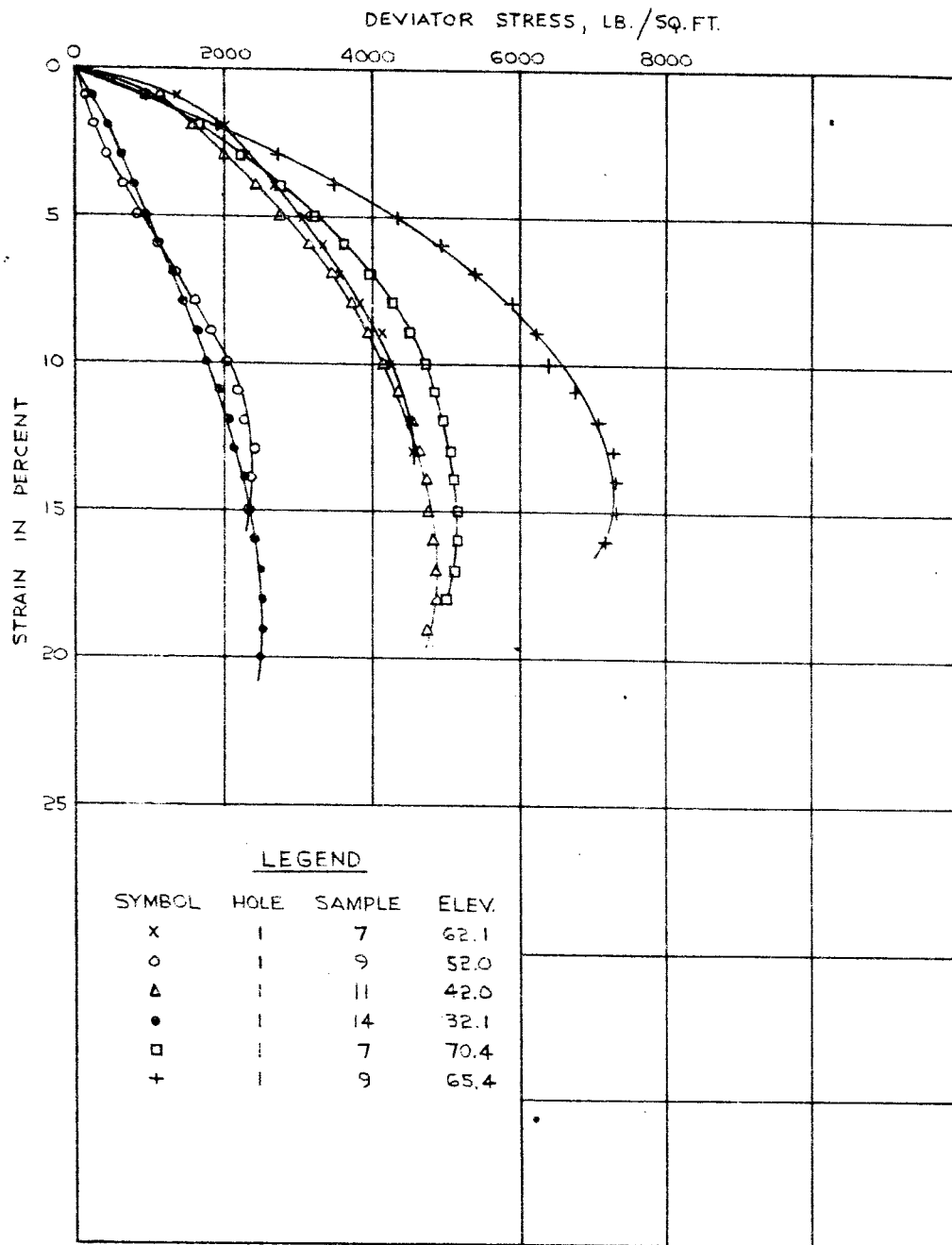


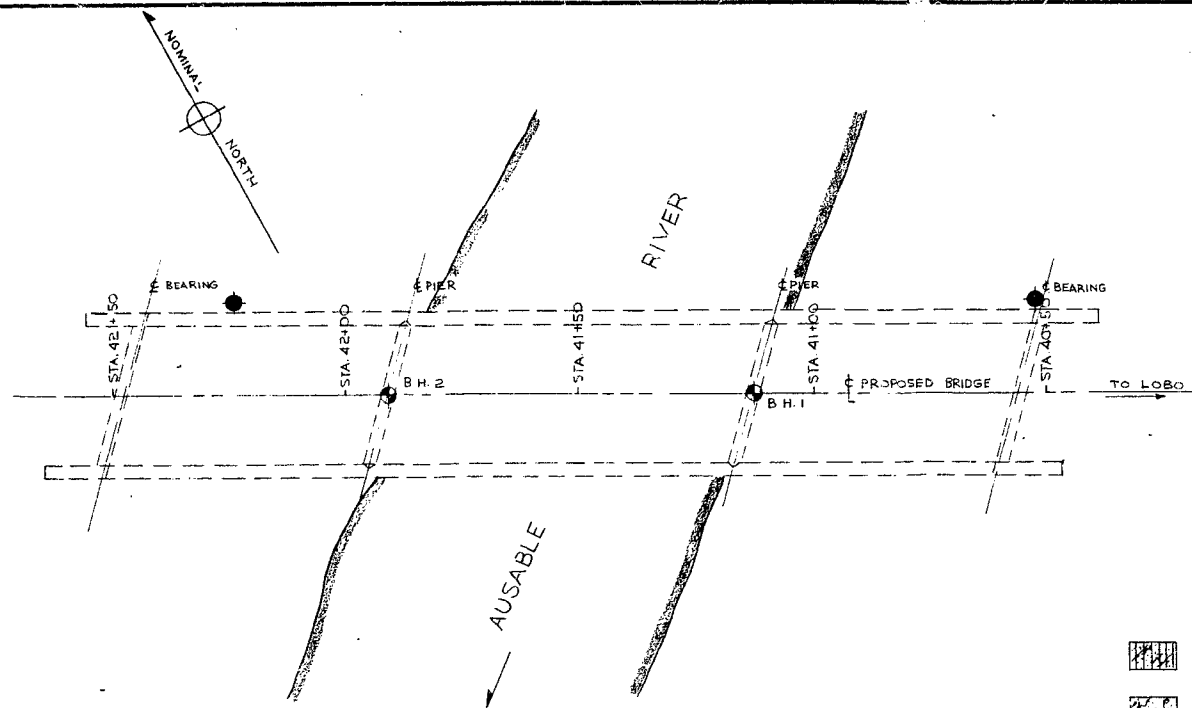
GOLDER &amp; ASSOCIATES

# UNCONFINED COMPRESSION TESTS

STRESS-STRAIN CURVES  
STRATIFIED SILT AND CLAY




FIGURE 2

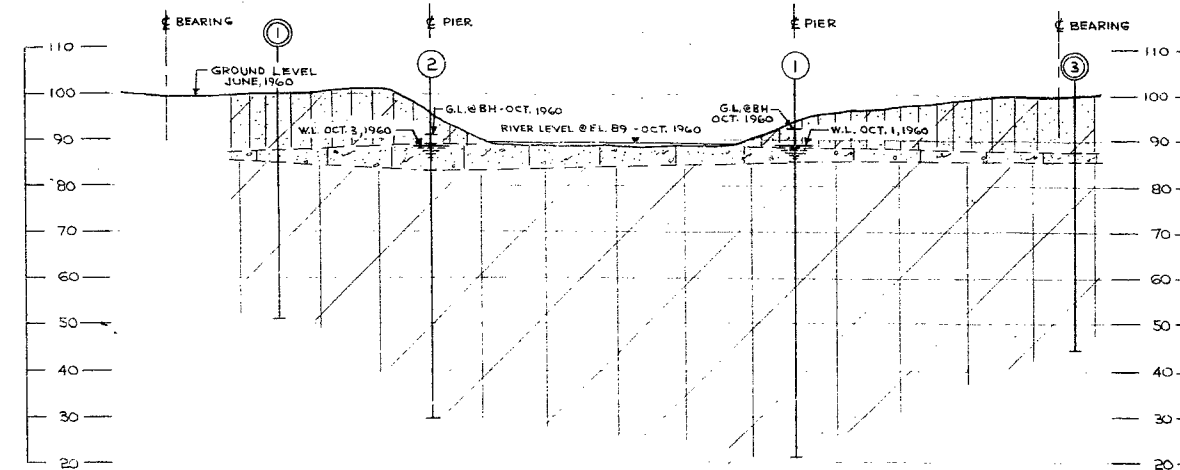




PLAN  
SCALE: 1" = 20'-0"





#### STRATIGRAPHY

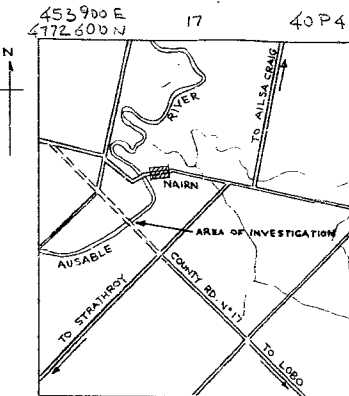
-  SOFT TO FIRM BROWN ORGANIC SANDY CLAYEY SILT
-  VERY LOOSE TO COMPACT GREY-BROWN ORGANIC SILTY SAND AND GRAVEL
-  STRATIFIED BROWN AND GREY SILT AND CLAY



SECTION ALONG CENTRE LINE  
SCALE: 1" = 20'-0"

#### LEGEND

-  BOREHOLE WITH PENETRATION TEST IN PLAN
-  BOREHOLE IN ELEVATION
-  BOREHOLE FROM PREVIOUS INVESTIGATION IN PLAN
-  BOREHOLE FROM PREVIOUS INVESTIGATION IN ELEVATION



KEY PLAN

SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. BETWEEN BOREHOLES THE SOIL STRATIGRAPHY HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

REFERENCE		DEPARTMENT OF HIGHWAYS, ONTARIO		GOLDER & ASSOCIATE	
DRWG. No.	DESCRIPTION	TORONTO		CONSULTING CIVIL ENGINEERS	
1 JOB 5703-1	M.M. DILLON & COMPANY LIMITED CONSULTING ENGINEERS - NAIRN BRIDGE GENERAL LAYOUT, DATED JUNE 2, 1960	NAIRN		DATE: OCT. 7, 1960 SCALE: AS SHOWN	
		ONTARIO		MADE CHKD. APPD.	
		BORING PLAN AND SOIL STRATIGRAPHY		J.A. A.V. H.E.	
				DRWG. No. 1	



28-2

NAIRN BRIDGE

## LOAD TEST

Elev top = 104.9

$$H = \frac{82.5}{22.4}$$

Pile length: 22.4'

Assumed datum = .000 - no load

LOADING			READINGS		Time			Differential	Total Settlement
Percentage of total	Jack gauge pressure	Load			Gauge reading - inches				
25%	600	22.5 Kips 11 1/2 TONS	9.00 AM	9.15	9.30	9.45	10.00	0	0
			.000	.000	.000	.000	.000		
50%	1350	45 K 22 1/2 TONS	10.00	10.15	10.30	10.45	11.00	.020	.020
			.003	.020	.020	.020	.020		
75%	2000	67.5 33 3/4 TONS	11.00	11.15	11.30	11.45	12.00	.006	.026
			.026	.026	.026	.026	.026		
100%	2700	90.0 45 TONS	1.00 PM	1.15	1.30	1.45	2.00	.017	.043
			.043	.043	.043	.043	.043		
125%	3350	112.5 56 1/4 TONS	2.00	2.15	2.30	2.45	3.00	.038	.081
			.071	.075	.075	.080	.081		
150%	4050	135.0 67 1/2 TONS	3.00	3.15	3.30	3.45	4.00	.030	.111
			.105	.111	.111	.111	.111		
175%	4700	157.5 78 3/4 TONS	4.00	4.15	4.30	4.45	5.00	.011	.122
			.122	.122	.122	.122	.122		
200%	5400	180 K 90 TONS	5.00	5.30	6.00	7.00	8.00		
			.167	.185	.188	.188	.189		
			9.00	10.00	11.00	12.00	1.00 AM		
			.189	.193	.198	.199	.201		
			2.00	3.00	4.00	5.00	6.00		
			.202	.204	.205	.207	.208		
			7.00	8.00	9.00	10.00	11.00		
			.210	.211	.213	.214	.214		
			12.00	1.00 PM	2.00	3.00	5.00		
			.214	.214	.214	.214	.214	.092	.214"

\* Pile - 12" I.D tube - not filled with concrete wall thickness: 1/4"

$$\text{Area of pile} = 12.125 \times \pi \times \frac{1}{4} = 9.52 \text{ sq in.}$$

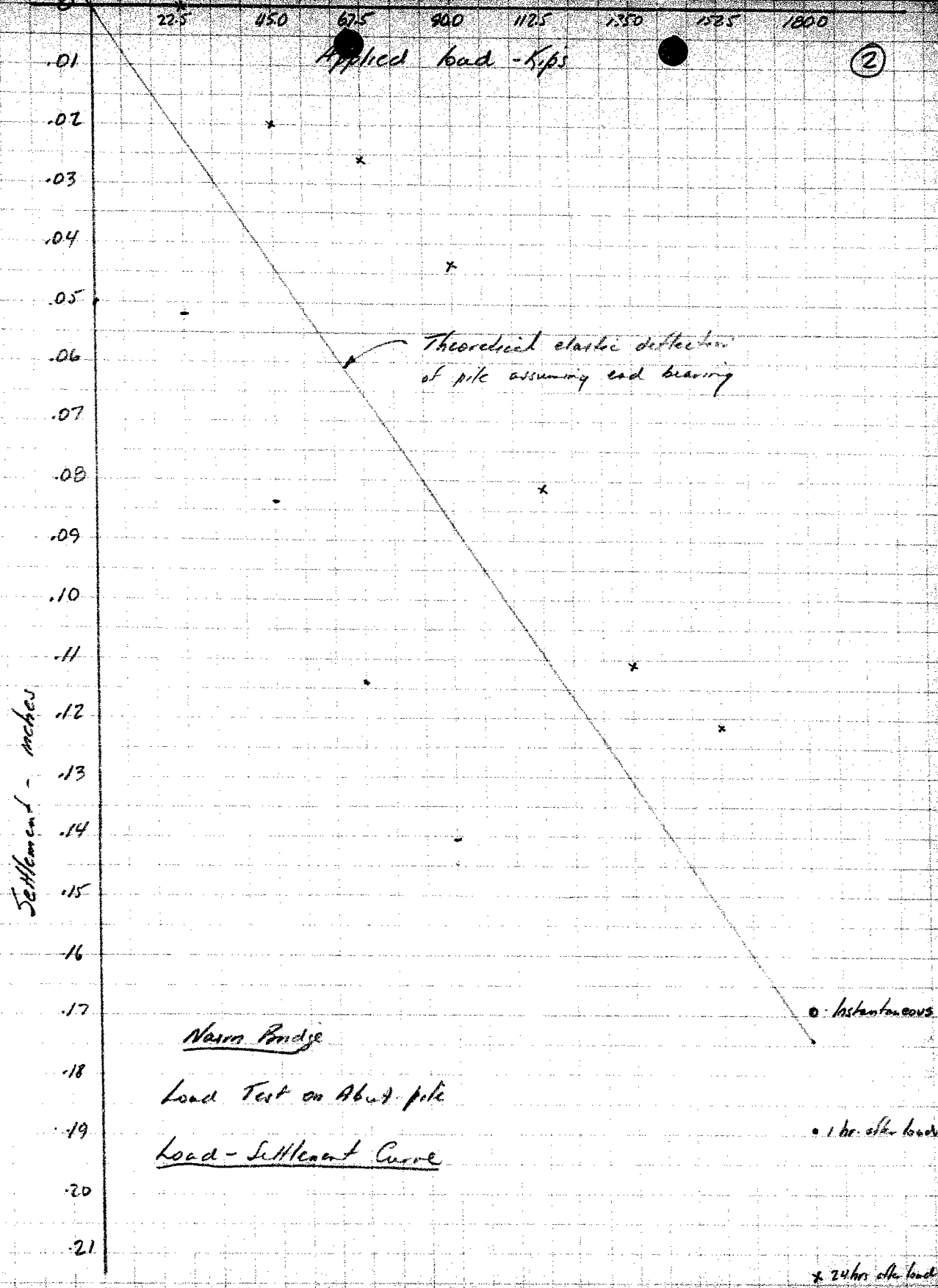
$$\text{Stress at max load} = \frac{180}{9.52} = 18.9 \text{ ksi}$$

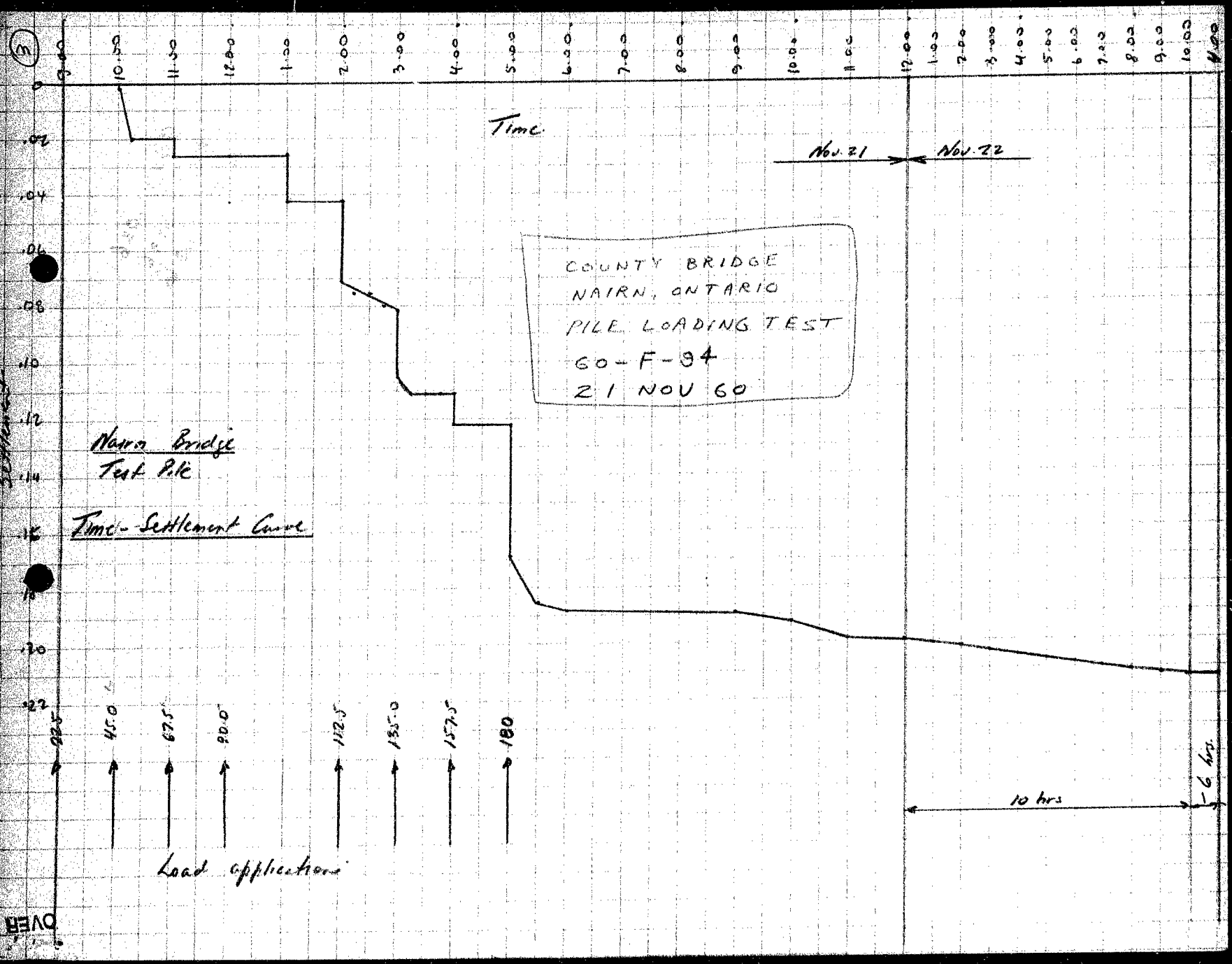
Elastic deflection (theoretical) of pile at max load

$$\Delta = \frac{180,000 \times 22.4 \times 12 \times \frac{1}{2}}{9.52 \times 29 \times 10^6} = .174"$$

CB

*100%	2700	90.0 <sup>K</sup> 45 Tons	5.00 PM .145	5.15 .142	5.30 .141	5.45 .141	6.00 .141
75%	2000	67.5 33 <sup>3</sup> / <sub>4</sub> Tons	6.00 .117	6.15 .115	6.30 .114	6.45 .113	7.00 .113
50%	1350	45 <sup>K</sup> 22 <sup>1</sup> / <sub>2</sub> Tons	7.00 .086	7.15 .085	7.30 .083	7.45 .083	8.00 .083
25%	600	22.5 11 <sup>1</sup> / <sub>2</sub> Tons	8.00 .053	8.15 .052	8.30 .052	8.45 .052	9.00 .052
0%	0	0	9.00 .050	9.15 .050	9.30 .050	9.45 .050	10.00 .050





DATE

930 - .036  
945 - .036  
10.00 - .036

10.10  
10.10  
10.10

0.24 ←  
0.18 ←  
0.08 ←  
0.55 ←  
0.02 ←  
0.55 ←  
0.01 ←

10.10

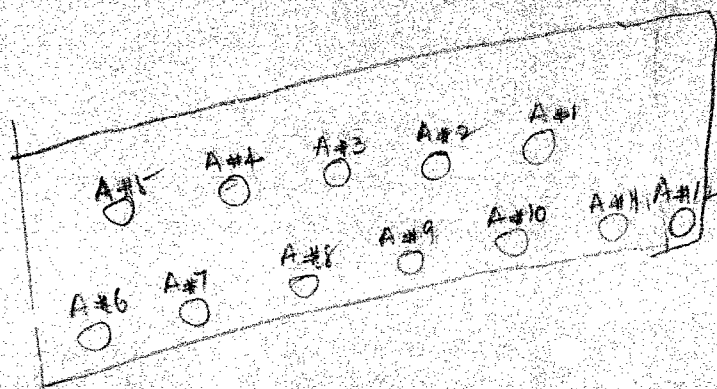
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W. I. MON CO.  
NOTE - at  
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W. I. MON CO.

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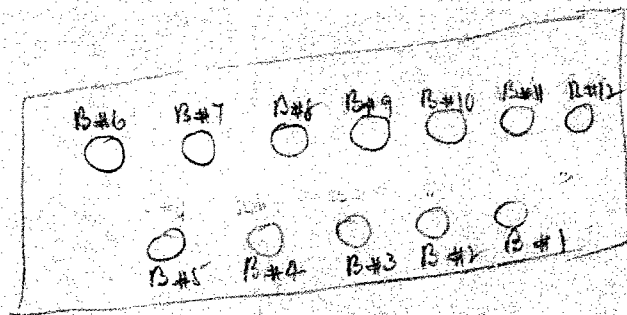
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West  
Abutment

River



East  
Abutment

Name --- Ausable River Bridge; Nairn, Ontario

Job Number --- 60-F-94      Contract No. 64-60

Hwy --- county road

District --- London

County --- Middlesex

Township --- East Williams

Contractor ---

Date Started ---

Date Completed ---

} 1 pile

Subsoil Conditions --- 5 ft. of silty sand and  
gravel over stratified silt and clay.

Type of Pile 12" Steel Pipe Pile x 22.4 ft.

Method of Loading

Cost of Test

Salaries, travelling and living expenses --- 114.06

Driving, skilled labour and materials ---

Total ---



ONTARIO

DEPARTMENT OF HIGHWAYS

London-----Ontario

Nov. 25, 1960

P. O. Box 217

Mr. L.G.Soderman,  
Foundation Engineer,  
Materials and Research  
Dept. of Highways,  
Parliament Bldgs.  
Toronto, Ontario.

Att. Mr. M.Devetta

Dear Sir:

RE: MIDDLESEX COUNTY  
Nairn Bridge  
Ausable River

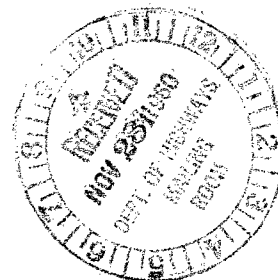
Enclosed are the remaining 7 pile driving record sheets  
for the above bridge.

Yours truly,

*D.A.O. White*

D.A. O. White  
ASSISTANT DISTRICT MUN. ENGINEER

DAOW:DP  
ENCL.





Cut-off elevations ✓

Location to test pile

Bridge drawing

# General Information

Dept. of Highways, Ontario

Bridge Construction. Pile Driving Record

List. no - 2

Contract no. 64/60 Hain Bridge (County of Middlesex)

Type of Hammer - D 12

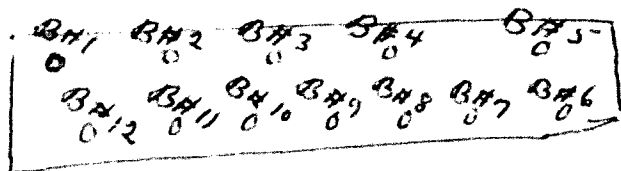
Weight of Hammer - and head  $3\frac{1}{2}$  ton.

Type of Cap -  $18" \times 18"$  with  $3" \times 24"$  base ~~plate~~ <sup>Steel (wooden)</sup>

Description of Pile -  $12" \times \frac{1}{4}"$  tubular ~~type~~ <sup>Steel</sup> type

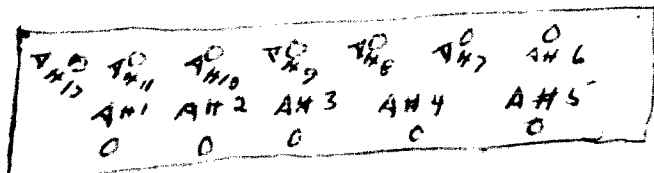
Weight of Pile per foot  $12" \times \frac{1}{4}" = \underline{\underline{33.38 \text{ lbs/ft.}}}$

Footings plan for piles <sup>I.D.</sup>



East Abutment

Arch



West Abutment

General Information  
Dept. of Highways, Ontario

Bridge Construction. Pile Driving Record

Dist. No. -- 2

Contract No. 64/60 Nairn Bridge (County of Middlesex)

Type of Hammer -- D. 12

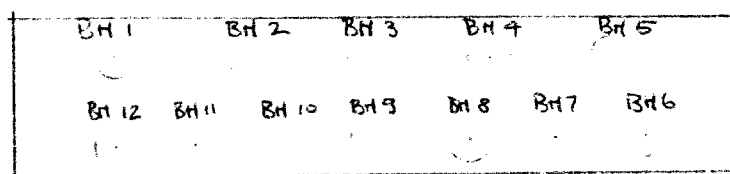
Weight of Hammer and head  $3\frac{1}{2}$  ton

Type of Cap -- 18" x 18" with 3" x 24" base (wood cushion)

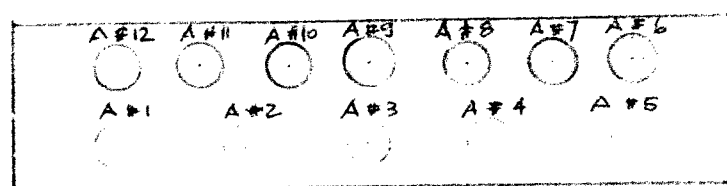
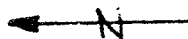
Description of Pile -- 12" x 1/4" tubular Steel type

Weight of Pile per foot 12" (I.D.) x 1/4" = 33.38 lbs/ft.

Footing plan for piles



E - ABUTMENT



W - ABUTMENT

BA. 1147

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS  
AND  
SOIL MECHANICS CONSULTATION

INCLUDE WITH 40P4-9

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

884 WILSON AVE.,  
DOWNSVIEW, ONT.  
ME. 5-5921

Project: J 473

February 25, 1960.

M. M. Dillon & Co. Ltd.,  
Consulting Engineers,  
141 Maple St.,  
London, Ont.



Attention: Mr. J.H. Kearney  
Project Engineer

Foundation Investigation  
Proposed County Road Crossing  
The Ausable River, Nairn, Ont.

Dear Sirs:

Enclosed herewith is our report on the soil conditions underlying the proposed Ausable River Crossing and its easterly approaches, at Nairn, Ont.

As stated in telephone conversations, as the work progressed, the site is underlain by clayey silt alluvium, probably laid down by the river during periods of flooding. This material extends to a depth of about 12 feet at which level a thin stratum of gravel was encountered. We feel that this latter material again was placed by the river as it progressed across its flood plain in geologically recent times.

The lower limit of the gravel, at approximate Elev. 85 feet, is believed to be an indication of the maximum depth of scour of this river. Very dense sand and very stiff clay lies in layers below the gravel.

After examining these conditions, we feel that the bridge structure can be supported either on simple footings bearing at Elev. 87 feet or on cylindrical steel piles driven to refusal in the underlying dense sand-clay composite. The safe bearing value in the former proposal has been estimated to be 6000 p.s.f. Excavation work, in this instance, will be about 6 feet below the water table and therefore it will be necessary to support the footing trench using sheet piling driven into

the underlying clay. In view of the high penetration resistance of this sand-clay, we expect that cylindrical piles should encounter refusal at a depth of about 20 feet.

Although the upper levels of alluvial clayey silt are soft, this material is strong enough to support 10 feet of embankment fill. However, in the vicinity of the river, rip rap protection must be provided.

We hope that the information contained in this report is sufficient to permit you to proceed with the design of this river crossing. Please contact us if any queries arise concerning the enclosed comments, or if other foundation proposals come to mind as the design work progresses.

Yours very truly,

*W. Trow*

William A. Trow (P. Eng.)

WAT/lt  
Encl.

WILLIAM A. TROW AND ASSOCIATES LTD.

M. M. DILLON & CO. LTD.,  
CONSULTING ENGINEERS  
141 MAPLE ST., LONDON, ONT.

FOUNDATION INVESTIGATION  
PROPOSED COUNTY ROAD CROSSING  
THE AUSABLE RIVER, NAIRN, ONT.

Project: J 473

February 25, 1960  
William A. Trow and Assoc. Ltd.

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Borehole Location Plan and Est.Stratigraphy	Dwg. 1
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FOUNDATION INVESTIGATION  
PROPOSED COUNTY ROAD CROSSING OF  
THE AUSABLE RIVER, NAIRN

This report describes the soil conditions underlying the route of the proposed County Road extension across the flood plain of the Ausable River at Nairn, Ont. Recommendations for the support of the bridge structure have been made and the stability of the approach embankments is discussed. A description of the field investigation methods is given in the Appendix.

Site Description

The proposed extension of the County road from Hwy. No.22 to Nairn crosses the Ausable River in an area where its course is relatively straight. Farther upstream, to the north-east, its direction changes to the left where it passes under an existing bridge.

The flood plain of the river at this location is contained in a section of land carved out of the hillside for a distance of about 600 feet east of the crossing site. Along the centre line of the proposed county road approach across this flood plain, the ground is covered with shallow ponds of ice; probings in the vicinity of Station 35 indicated that the ground underneath was soft. In general this flat land is about 7 feet above the present river level. It is understood that the ground is covered with water during periods of spring flooding. The eastern bank of the valley rises about 38 feet to a flat plain which is more or less characteristic of this part of the county. West of the river the ground rises very gradually.

The existing bridge was examined in order to ascertain the extent of river scour. However, because of the accumulation of ice around each abutment, it was not possible to determine the amount of erosion. The current was quite fast under this structure and the river banks in the vicinity appeared to be undercut.

Subsoil Description

A total of 5 borings and 4 penetration tests were made at this project location. Two of these holes were made along the easterly approaches to the river crossing and the remainder of the work was carried out at the bridge site.

The logs for each boring are presented as Dwg. 2 to 6 of this report. Each log contains a description of the soil profile from the ground surface to the maximum depth investigated, together with a record of laboratory and field test measurements.

In order to assist in the overall appraisal of the subsoil conditions, the information from these borehole logs has been summarized into the estimated stratigraphical profile shown in Dwg. 1. Reference to this drawing shows that the flood plain of the Ausable River is underlain by loose or



medium stiff deposits of stratified fine sandy clayey silt. In the vicinity of the stream, this fine-grained loose soil extends down to approximate Elev. 87.5, which corresponds more or less to the maximum depth of the river bed at the present time. Farther to the south-east in the ox bow area, at one time occupied by the river, this weaker soil terminates at Elev. 90 ft., or about 2 feet higher up. Some partially decayed leaves were noted in Hole 3 at a depth of about 9 feet, but in most instances the soil was relatively clean.

Underlying this compressible material, in all instances, is a thin layer of wet fine to coarse gravel which ranges in thickness from about 2 to 4 feet. It has a penetration resistance of the order of 28 blows per foot. Below the gravel a very dense or very stiff deposit of grey clay and silty fine sand exists. The sand appears to occur as intrusions and thick interbeds within the clay and it becomes the pre-dominant soil type below a depth of about 18 feet. Below about Elev. 52 feet the soil changes again into a dense sandy silt with some thin layers of clay contained in it.

Hole 6 was made just back of the crest of the hill which forms the east boundary of the river flood plain. The profile here consists of about  $6\frac{1}{2}$  feet of very stiff brown clay which is underlain by 8 feet of very dense fine sand and then at Elev. 123 feet by stiff grey clay with interbeds of sand. The water table lies about 9 feet below ground surface at this hill location.

In the valley, the water table is practically at the surface in the ox bow area east of the bridge and it is about one foot above river level adjacent to this stream.

#### Discussion of Subsoil Characteristics

The foregoing description indicated that the soil strata below the flood plain can be considered to consist of 3 distinct types.

The material found at and above the elevation of the existing river bed comprises alluvial deposits which probably have been deposited each spring as the stream overflows its banks. The stratification of this material and the presence of organics at various levels in the soil would appear to support this view. For the most part this flood plain deposit can be classified as a sandy clayey silt, although layers of coarser sand and gravel were noted in samples from some of the borings. Although the area is flat and poorly drained, this soil has been stiffened somewhat by surface drying and by removal of water through tree roots. As a consequence, it has a shear strength of the order of 400 p.s.f. or more, and its moisture content is well below the liquid limit value of about 39%. Because of the presence of layers of medium to coarse sand some of the field and laboratory measurements of shear strength are not considered to be applicable.

The dense wet fine to coarse gravel underlying this flood plain silt probably represents the coarse deposits formed in the river bed during spring flooding. They extend to an average depth equivalent to Elev. 85 feet which level could be taken as the approximate maximum depth of river bed scour. This material is quite permeable and it exists in a medium dense to dense condition.

The original soil at this location begins about Elev. 85 feet and it consists of alternate layers of dense very fine sand and very stiff grey clay. This layered composite has a penetration resistance in the order of 40 to 60 blows per foot and the clay phases of it exist at a moisture content close to the plastic limit. The liquid and plastic limits of the clay are approximately equal to 35 and 17 percent respectively; its shearing strength is at least equal to 2600 p.s.f. Both this high strength and low moisture content are to be expected in view of the fact that a much greater weight of soil covered the area at one time, before the present valley was carved out.

The results of boring No. 6 indicate that the soil, beyond the eastern slope of the valley, consists either of very dense fine to medium sand or of very stiff clay. This latter material exists at a moisture content slightly above the plastic limit. Both soil types are suitable as compacted fill for the bridge approaches although the clay is probably slightly wet of the optimum placement condition. This state should improve if removal and compaction is undertaken during hot dry summer months.

The water table lies near the upper third part of the uniform fine sand and therefore some drainage of this material may be required before proper placement can be carried out. This drainage should take place naturally as the excavation into the hillside progresses.

#### Discussion of Foundation Requirements

In the design of this river valley crossing, consideration must be given to the following matters:

- (a) The method of placing the earth fill approaches and the stability of these approaches.
- (b) The type of bridge foundation to use and the design bearing value to apply.
- (c) The measures necessary to prevent erosion or undermining of these foundations during periods of high, fast river flow.

Since the approach embankments will reach a height only of the order of 10 feet, the danger of failure due to overloading the supporting alluvial deposits seems remote. Although circular arc analyses would be required to determine the factor of safety against failure, an approximate guide to this problem is to consider the embankment as a continuous footing.

An embankment 10 feet high will exert a pressure on the underlying soil in the order of 1200 p.s.f. The shearing resistance required at the instant of failure is equal to  $1200/5 = 240$  p.s.f. With the exception of one laboratory test result, which has been discredited because of handling disturbances, the shear strength of the clayey silt is at least of the order of 400 p.s.f. As a consequence, there appears to be a sufficient potential of shearing resistance to support 10 feet of fill safely. This remark does not apply in the area adjacent to the river in the event that its banks are eroded and undermined. Adequate rip rap protection must be provided to prevent this possibility.

In view of the wet nature of the existing ground along the easterly approach to the bridge site, excavation of topsoil and placement of fill should be carried out, if possible, during dry summer months. In addition the first 1 to 2 feet of soil should consist of the fine to medium sand encountered below Elev. 131 in Hole 6. In this way, a dry base will be provided for the placement of clay.

With regard to the support of the bridge structure, reference to the borehole logs indicates that material having a high capacity exists just below the bed level of the existing river. The thin layer of alluvial gravel was found to have a penetration resistance of the order of 28 blows per foot; the underlying clay-sand composite offered even higher resistances to sampling. The shear strength of one sample of the clay was found to be 2600 p.s.f. and, as stated previously, the moisture content was near the plastic limit of this material.

One alternative for bridge support would be to place the bridge abutments on the top surface of the alluvial gravel. On the basis of the penetration resistance measurements for the gravel and of the shear strength of the clay, the safe bearing value for a deep footing founded at Elev. 87 feet, or about 6 feet below river surface level is 6000 p.s.f. The objection to this method of support is that considerable water will flow into the excavation as digging work progresses. This unsatisfactory condition can be avoided by driving light interlocking steel sheet-piling around the perimeter of the proposed abutments prior to digging. This piling should be driven at least 5 feet into the underlying clay and it should be adequately braced. It could be left in place after construction as additional protection against river scour. An alternative and less positive method of controlling the water flow is to depress the water table by pumping from sumps dug to the gravel adjacent to the abutment sites. A large volume of water seepage should be anticipated.

The alternative type of foundation which would avoid most of the ground water problems is one incorporating the use of cylindrical piles. In view of the high penetration resistances encountered during sampling it is probable that cylindrical piles would meet refusal at a depth of about 20 feet, or Elev. 80. In this instance, the capacity of each pile will be determined by its structural properties when considered as a short column.

In the unlikely event that the piles penetrate below the very dense soil at a depth of 20 feet, their ultimate capacity can be estimated assuming a shaft friction of 2000 p.s.f. Present field experience indicates that this is the maximum dependable value generated in stiff clay soils. This resistance would begin at a depth of 14 feet; a factor of safety or about 2 should be applied to the resulting computations.

As suggested in earlier comments consideration must be given to measures for protecting the river bed against scour and lateral erosion. The fine grained flood plain materials encountered above river bed level will not offer much resistance to erosion, particularly after the natural flood plain is blocked by the country road embankment and the rate of river flow is consequently increased under the bridge. The amount of protection upstream and downstream of the bridge and the size of the rip rap are hydraulic problems beyond the scope of this report. However, any rip rap protection that is placed should be backed by a bed of well-graded pit-run gravel at least 2 feet thick. This gravel will act as a filter to prevent the "sucking-out" of natural alluvial fines through the large spaces between the rip rap. Protection at least up to high water level, Elev. 108 feet, is required.

#### Summary of Comments and Conclusions

The foregoing observations and comments can be summarized briefly as follows:

- 1) The bridge site and its easterly approach is underlain by about 12 feet of loose or medium stiff sandy clayey silt alluvium, the result of the yearly flooding of the Ausable river. It is underlain by a thin layer of dense gravel and then by alternate layers of very stiff clay and of dense sand.
- 2) The easterly bank of the Ausable river flood plain is composed of very stiff clay and of uniform fine to medium wet sand. Both materials should be suitable for use as fill, although the clay exists at a moisture content slightly above the plastic limit and therefore will compact to a density slightly wet of the optimum condition. The wet sand should drain as excavation into the hillside progresses.
- 3) Although the upper levels of the alluvial clayey silt in the flood plain are soft, the material is sufficiently strong to support 10 feet of fill. In order to provide a dry base and to assist the consolidation of the underlying soil, the bottom one or two feet of the embankment should consist of sand.
- 4) The bridge abutments can be supported either on simple footings or on piles. Support in the former instance can be obtained at Elev. 87 feet, and the safe bearing value to apply at this level is 6000 p.s.f. Control of ground water can be obtained by driving interlocking steel

sheet piling around the perimeter of the footing excavation, at least to Elev. 80 feet. This piling will provide long term protection against undermining. Alternatively, cylindrical steel piles can be used to carry the weight of the bridge. Refusal to driving at shallow depth below Elev. 80 feet is anticipated.

5) Rip rap protection of the bridge structure is required since the alluvial clayey silt will have little resistance to erosion in the restricted parts of the river adjacent to the bridge. The rip rap should be placed on a bed of well-graded pit-run gravel 2 feet thick.

WAT/lt  
Encl.  
J473



*W. Trow*

William A. Trow (P. Eng.)

APPENDIXField Investigation Methods

The borings of this investigation were performed using continuous flight auger equipment. The holes were 5 inches in diameter and were uncased to full depth.

Samples generally were taken at 5 foot intervals of depth although closer sampling was carried out within the first 10 feet at the bridge site. In most instances, the soil was recovered in the disturbed state using the conventional 2 inch O.D. split spoon. This was done because the soil, in most cases, was very dense, or examination of material exposed on the auger indicated the presence of sand. In some instances, particularly at upper levels, undisturbed samples were recovered in 2 inch I.D. shelly tubes. In one or two cases sand or gravel was recovered in these tubes, although augered soil immediately above consisted of clay.

Field vane tests were performed in the first 12 feet of alluvial soil, in order to obtain in-situ measurements of the shear strength of the soil. Here again, the results were of little value at some levels because the soil tested was found to be too granular. No vane tests were made in the underlying natural clay because its strength was well in excess of the capacity of this device.

Cone penetration tests were performed at the bridge site in order to confirm split spoon resistance measurements and to determine more exactly where dense soil began.

Water level observations were made in the uncased borings as the field program progressed. The elevations of the holes were referred to the top of an iron peg located 32 feet right of Station 40+00 on the east bank of the river. The elevation of this pin has been given as 104.07 feet.

SUMMARY OF LABORATORY AND FIELD TEST MEASUREMENTS (1)

Hole	Depth	Sample Type	Pene. Resis. Blows/ft.	Description	Shear Strength p.s.f.	Natural Mois. Con. % Dry wt.	Atterberg Lts. P.L. L.L.
1	3-4 $\frac{1}{2}$	Split spoon	5	Brown sandy clayey silt-some shell frag.			
	5-6 $\frac{1}{2}$	Shelby	push	as above	500 Qu	28.2	18.8 38.9
	8	Vane test			1050 V		
	8-9 $\frac{1}{2}$	Shelby	push	as above	800 Qu	27.9	
	11	Vane test			1008 V		
	13-14 $\frac{1}{2}$	Shelby	push	Wet fine to coarse gravel			
	18-19 $\frac{1}{2}$	Split spoon	52	Very stiff clay to 19 ft. -very fine sand below		16.7	
	23-24 $\frac{1}{2}$	Split spoon	28	Wet very fine sand			
	28-29 $\frac{1}{2}$	do	38	Wet fine sand to 29.2; stiff clay below			
	33-34 $\frac{1}{2}$	do	29	Stiff silty clay			
	38-39 $\frac{1}{2}$	do	23	" "			
	43-44 $\frac{1}{2}$	do	13	Wet silt and fine sand with thin layers clay			
	48-49 $\frac{1}{2}$	do	11	" "			
Hole 4	2-3 $\frac{1}{2}$	Split Spoon	4	Fine brown sandy clayey silt			
	4 $\frac{1}{2}$	Vane test			1260 V		
	5-6 $\frac{1}{2}$	Split spoon	2	Fine sandy silt with shell fragments			
	8	Vane test			924 V		
	8-9 $\frac{1}{2}$	Split spoon	5	Fine silty sand with fine gravel and shells below 9 feet			
	13-14 $\frac{1}{2}$	do	28	Fine to coarse gravel- some shells			
	19-20 $\frac{1}{2}$	do	86 for 10"	Very stiff clay with interbeds of fine silty sand		16.0	

LEGEND: Qu - Undrained triaxial test at overburden pressure  
V - Field vane test  
PL - Plastic Limit  
LL - Liquid Limit

## SUMMARY OF LABORATORY AND FIELD TEST MEASUREMENTS (2)

	Depth	Sample Type	Pene.Resis. Blows/ft.	Description	Shear	Natural	Atterberg Lt.	
					Strength p.s.f.	Mois. Con. % Dry. Wt.	P. D.	L. L.
Hole								
3	3-4 $\frac{1}{2}$	Split spoon	7	Brown sandy cohesive silt				
	5-6 $\frac{1}{2}$	Shelby	push	Alt. layers of silty fine sand and med. to coarse sand with shell fragments	No test (too granular)			
	8-9 $\frac{1}{2}$	Split spoon	2	Sandy fine gravel & shells to 8 $\frac{1}{2}$ ft.; sandy silt with decayed leaves below				
	13-14 $\frac{3}{4}$	Shelby	33	Gravel up to 2" size to 14 ft.; very stiff br. clay with intrusions fine sand below	2600	18.6	16.5	35.0
	18-19 $\frac{1}{4}$	Shelby	66	Dense very fine sand				
	23-24 $\frac{1}{2}$	Split spoon	32	Wet silty fine sand to 23.7; stiff clay below.		21.1		
	28-29 $\frac{1}{2}$	do	51	Stiff clay with interbeds of fine sand		22.6		
	33-34 $\frac{1}{2}$	do	59	Stiff clay to 33.9; silty sand to 34.2; stiff clay below.				
	38-39 $\frac{1}{2}$	do	50	Wet fine sand to 39.3 - silty clay below				
	43-44 $\frac{1}{2}$	do	107	Alt. layers stiff clay & dense very fine sand				
	48-49 $\frac{1}{2}$	do	19	Very fine sandy silt.				
	53 $\frac{1}{2}$ -55	do	50	Dense silt with some thin clay layers				
Hole								
5	5-6 $\frac{1}{2}$	Split spoon	6	Sandy clayey silt with shells				
	5-6 $\frac{1}{2}$	Shelby-5 ft. away	push	Very soft br. clayey silt to 5.2; becoming silty sand and then fine gravel below.	140 (not reliable)	35.2		
	8	Vane test			420			
	10-11.7	Shelby	push	Fine to coarse wet gravel				
	15-16 $\frac{1}{2}$	Split spoon	56	Stiff grey clay with interbeds of fine sand.				
	20-21 $\frac{1}{2}$	do	49	as above.				
	25-26 $\frac{1}{2}$	do	52	Predominantly very fine sand with some clay layers.				
Hole								
6	5-6 $\frac{1}{2}$	Split spoon	32	Stiff reddish br. clay - sand at 6 $\frac{1}{2}$ ft.		23.9	19.5	46
	10-11 $\frac{1}{2}$	do	82	Stratified fine to medium sand				
	15-16 $\frac{1}{2}$	do	28	Wet grey fine sand with some clay layers.				
	20-21.7	Shelby	push	Very stiff grey clay	3600	20.0	17.2	41.2



PROJECT NO. J473

## WILLIAM A. TROW &amp; ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT County Bridge Crossing Ausable R.

LOCATION Nairn, Ontario

HOLE LOCATION See dwg. 1

HOLE ELEVATION AND DATUM 100.0 - BM see dwg. 1

BOREHOLE NO. 1

FIELD SUPERVISOR

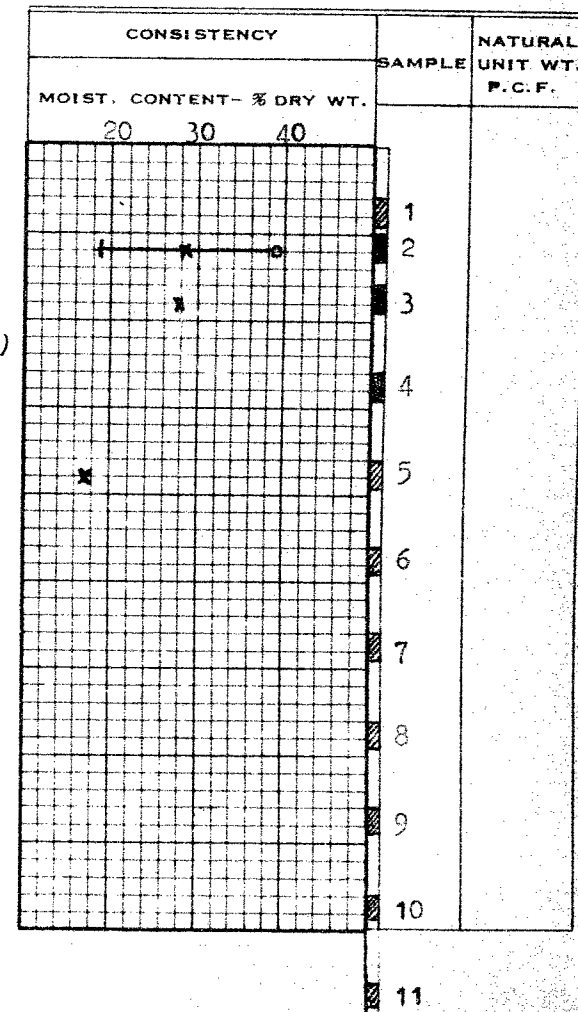
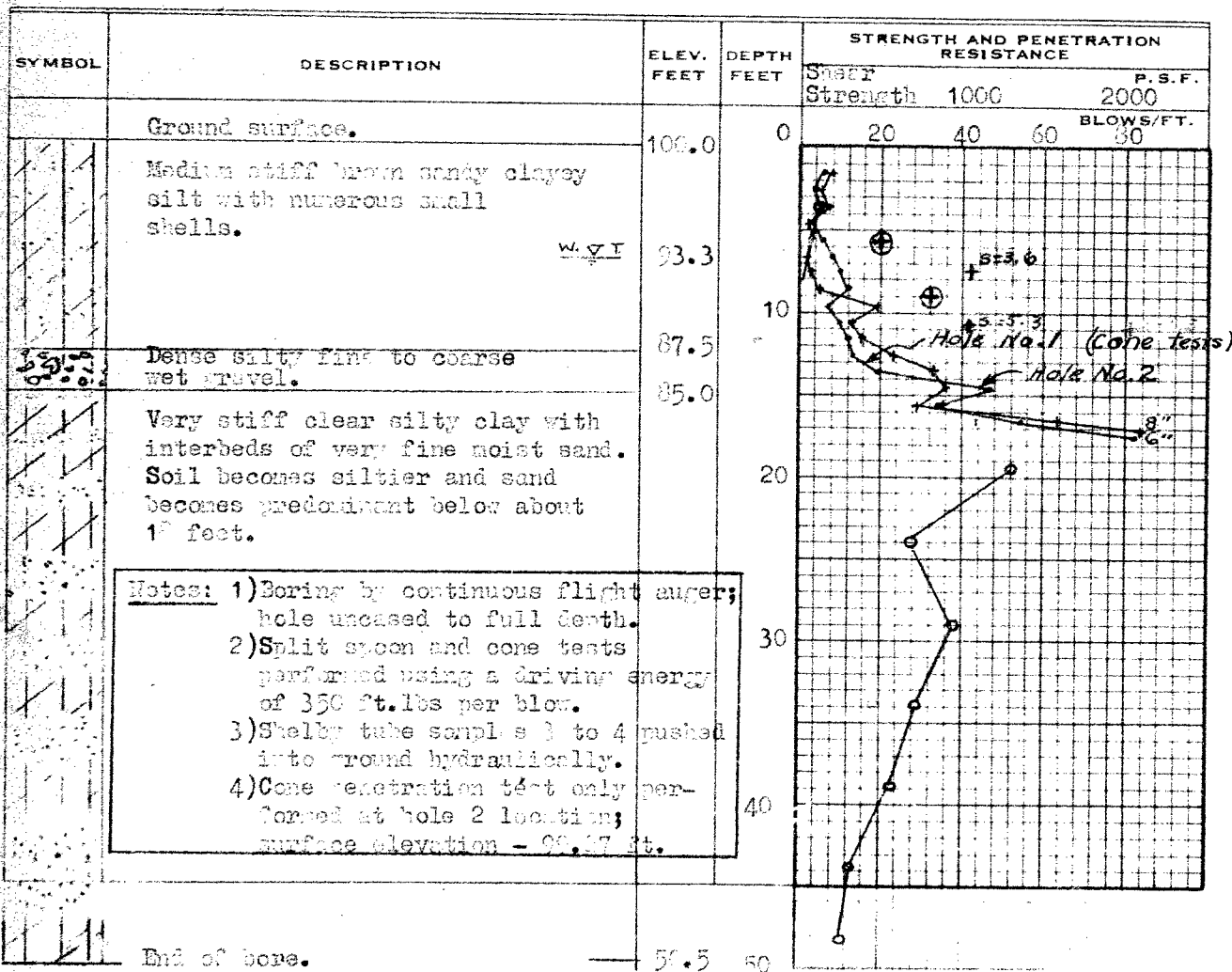
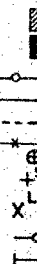
DRILLER

PREP.

DRAWING NO. 2

## LEGEND

2" DIA. SPLIT TUBE  
 2" SHELBY TUBE  
 2" SPLIT TUBE  
 2" DIA. CONE  
 CASING  
 2" SHELBY  
 1/2 UNCONFINED COMPRESSION (Qu)  
 VANE TEST (C) AND SENSITIVITY (S)  
 NATURAL MOISTURE AND  
 LIQUIDITY INDEX  
 LIQUID LIMIT  
 PLASTIC LIMIT



## WILLIAM A. TROW &amp; ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT County Bridge Crossing Ausable R.  
 LOCATION Nairn, Ontario  
 HOLE LOCATION See dwg. 1  
 HOLE ELEVATION AND DATUM 99.62

BOREHOLE NO. 3  
 FIELD SUPERVISOR  
 DRILLER  
 PREP.

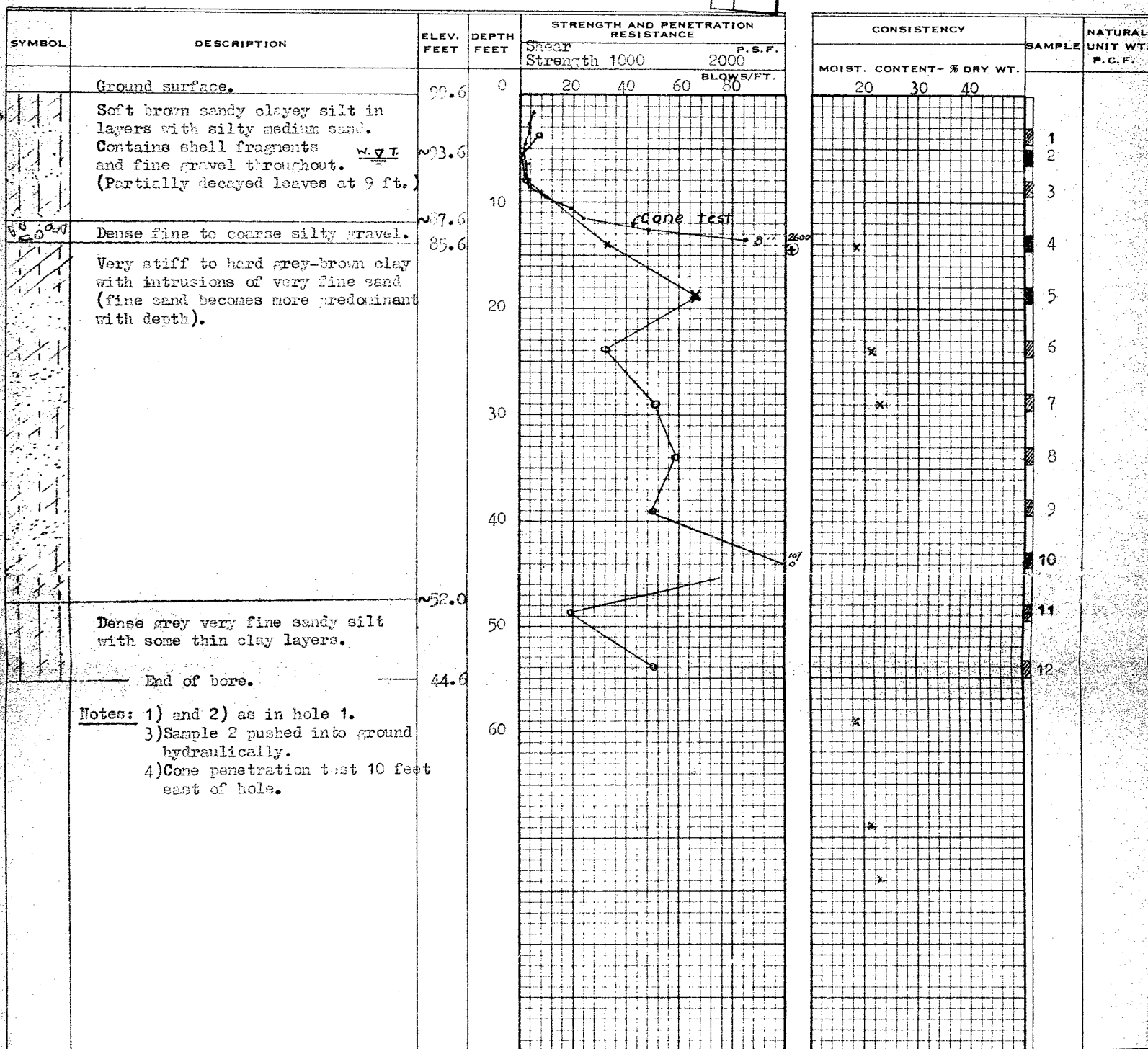
GEOCRIS No.

4084-9

## LEGEND

2" DIA. SPLIT TUBE  
 2" SHELBY TUBE  
 2" SPLIT TUBE  
 2" DIA. CONE  
 CASING  
 2" SHELBY  
 1/2 UNCONFINED COMPRESSION (Qu)  
 VANE TEST (C) AND SENSITIVITY (S)  
 NATURAL MOISTURE AND  
 LIQUIDITY INDEX  
 LIQUID LIMIT  
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PROJECT NO. J473

## WILLIAM A. TROW &amp; ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT County Bridge Crossing Ausable R.

LOCATION Nairn, Ontario

HOLE LOCATION See dwg. 1

HOLE ELEVATION AND DATUM 99.8

BOREHOLE NO. 4

FIELD SUPERVISOR

DRILLER

PREP.

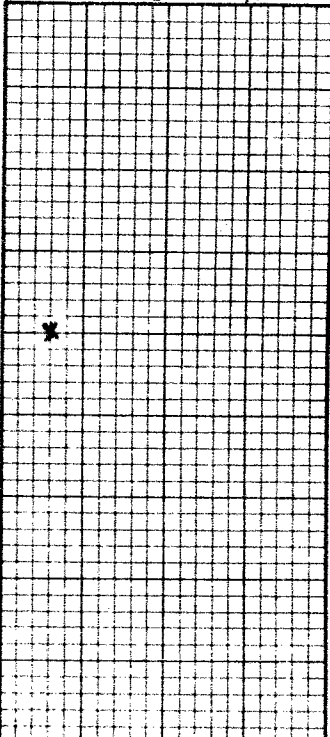





DRAWING NO. 4

## LEGEND

2" DIA. SPLIT TUBE  
 2" SHELBY TUBE  
 2" SPLIT TUBE  
 2" DIA. CONE  
 CASING  
 2" SHELBY  
 1/2 UNCONFINED COMPRESSION (Qu)  
 VANE TEST (C) AND SENSITIVITY (S)  
 NATURAL MOISTURE AND  
 LIQUIDITY INDEX  
 LIQUID LIMIT  
 PLASTIC LIMIT

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SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				Strength	P.S.F.
	Ground surface.	99.8	0	20 40 60 80	BLOWS/FT.
	Loose brown slightly cohesive silty fine sand; shell fragments and some fine roots; fine gravel and shells below 9 feet.	93.8	10		
	Dense wet medium to coarse gravel.	87.0			
	Very stiff grey clay with interbeds of fine silty sand.	83.0			
	End of bore.	79.4	20		
Notes: 1) and 2) as in hole 1. 3) Cone test 10 ft to east of borehole.					

CONSISTENCY			SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.				
20	30	40		
			 1	
			 2	
			 3	
			 4	
			 5	

PROJECT NO. J473

**WILLIAM A. TROW & ASSOCIATES LTD.**

## SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

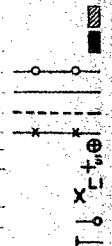
PROJECT County Bridge Crossing Ausable R.  
LOCATION Nairn, Ontario.  
HOLE LOCATION See diag. 1  
HOLE ELEVATION AND DATUM 29.9




BOREHOLE NO. 5  
FIELD SUPERVISOR  
DRILLER  
PREP.

DRAWING NO. 5

### LEGEND

- 2" DIA. SPLIT TUBE  
2" SHELBY TUBE  
2" SPLIT TUBE  
2" DIA. CONE  
CASING  
2" SHELBY  
1/2 UNCONFINED COMPRESSION [Qu]  
VANE TEST [C] AND SENSITIVITY [S]  
NATURAL MOISTURE AND  
LIQUIDITY INDEX  
LIQUID LIMIT  
PLASTIC LIMIT



SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				Shear Strength	1000	2000	P.S.F. BLOWS/FT.
	Ground surface.	99.9	0	20	40	60	80
	Six inches topsoil.						
	Loose brown wet sandy clayey <u>W. &amp; T.</u> silt with some organic matter and shells and considerable fine gravel below 6 feet.	98.0					
	Dense med. to coarse silty gravel.	90.0	10				
		87.0					
	Very stiff grey clay with interbeds of fine silty sand.						
	End of bore.	73.4					
<u>Notes:</u> 1) and 2) as in hole 1. 3) Ground too soft and wet to place boring along road centre line. 4) Shelby tube taken at 5 - 6 1/2 ft., 4 feet to west of boring. (This sample contained too much sand and gravel for satisfactory test; tended to slump with handling.)				30			

CONSISTENCY			SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.				
20	30	40		
			1	
			2	
			3	
			4	
			5	

PROJECT NO. J473

**WILLIAM A. TROW & ASSOCIATES LTD.**

## SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

**PROJECT** County Bridge Crossin - Ausable R.

**LOCATION** Hainn, Ontario.

HOLE LOCATION... See Enc. 1

HOLE ELEVATION AND DATUM. 138.1

BOREHOLE NO. .... 6 .....

FIELD SUPERVISOR...

## DRILLER

**PREP.** \_\_\_\_\_

DRAWING NO. .... 6

### LEGEND

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2<sup>11</sup> DIA. CONE

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1/2 UNCONFINED COMPRESSION [Qu]

### VANE TEST [C] AND SENSITIVITY [S]

## NATURAL MOISTURE AND

LIQUIDITY INDEX

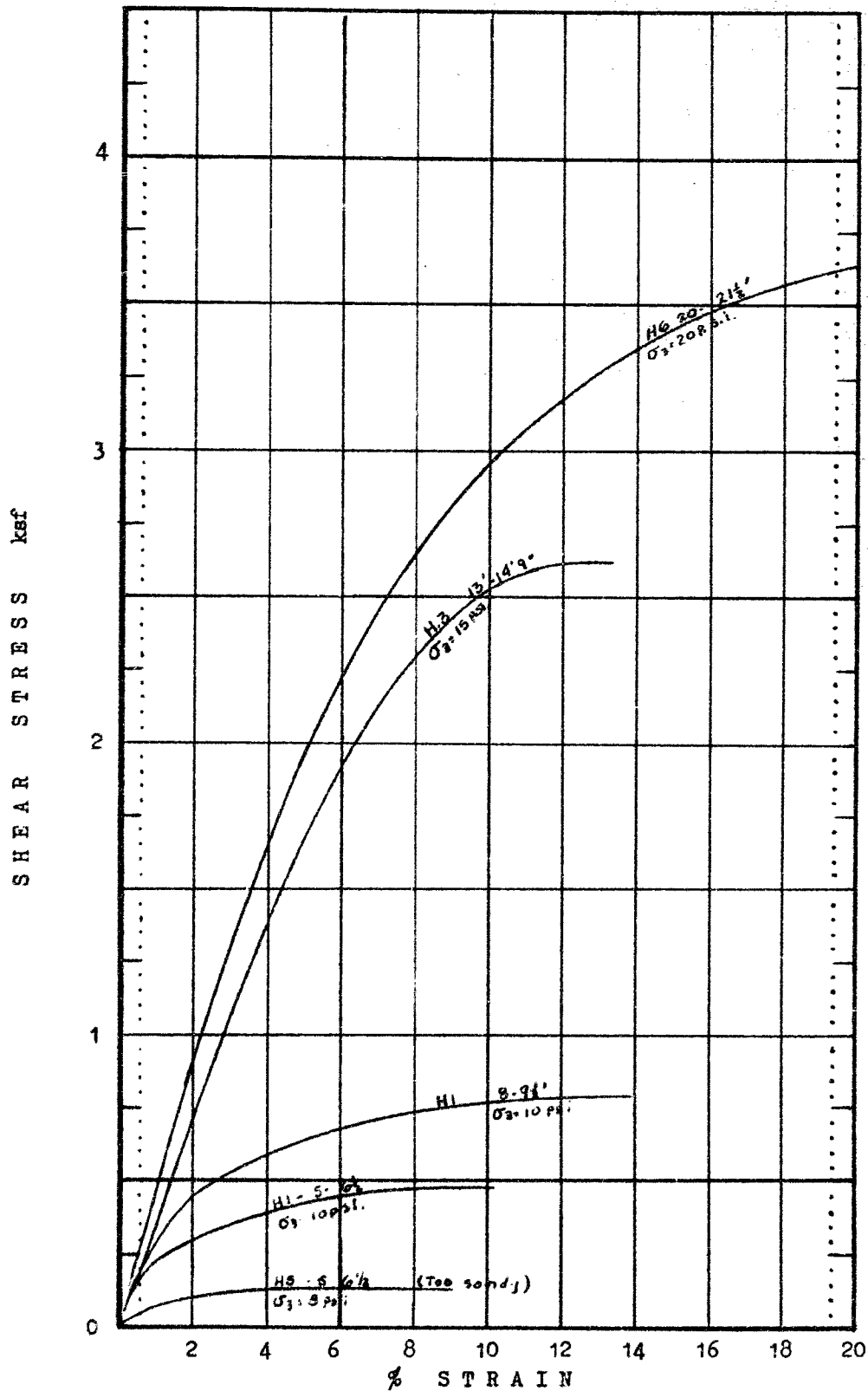
LIQUID LIMIT

PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				Shear Strength	P.S.F. BLOWS/FT.
	Ground surface.		0		
	1 Ft. topsoil.	138.1			
	Very stiff brown clay with some fine pebbles.	131.6			
	Very dense fine brown stratified sand (wet below 9 feet).	129.0	10		
	Very stiff grey clay with interbeds of fine sand.	123.0			
	End of bore.	117.4	20		

Notes: 1) and 2) as in hole 1.

CONSISTENCY		SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.			
20	30	40	
		1	
		2	
		3	
		4	

UNDRAINED TRIAXIAL TEST RESULTS - NAIRN BRIDGE

# WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS  
AND  
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

884 WILSON AVE.,  
DOWNSVIEW, ONT.  
ME. 5-5921

PROJECT: J 473

March 18, 1960.

M. M. Dillon & Co. Ltd.,  
Consulting Engineers,  
141 Maple St.,  
London, Ont.



Attention: Mr. J. H. Kearney

## Stability and Settlement Problems Nairn Bridge

Dear Sirs:

This letter confirms our recent telephone conversation in which the problems of embankment stability adjacent to the bridge abutments and of differential settlement of the multi-span structure were discussed.

With regard to embankment stability, it was noted that the most critical portion of the fill was a 40 foot section which reached a height of 17 feet adjacent to the west abutment. Assuming a compacted fill unit weight equal to 125 p.c.f., the maximum pressure exerted on the soil in this area was computed to be 2120 p.s.f. The lowest shear strength measurement in the alluvial sandy clayey silt in the vicinity of the bridge was the value of 500 p.s.f., determined by an undrained triaxial test on a sample from Hole 1. Higher shear strength values were obtained in this and in other holes.

Although stability analyses should be performed in order to determine the safety of this section, a conservative method of appraising the stability of the 17 foot embankment is to consider the fill load as equivalent to the pressure exerted by a continuous footing. The ultimate bearing capacity in this instance is equal to 5.1 times the cohesion of the soil, or equal to 2550 p.s.f., for  $C = 500$  p.s.f. The factor of safety against failure in this extreme condition therefore, is equal to  $2550/2120 = 1.2$ . When consideration is given to the fact that 2:1 embankment slopes will be provided, that a two-dimensional failure analyses is not strictly applicable for this restricted section, and that some consolidation of the alluvial soil will take place as each compacted lift is placed, this theoretical factor of safety would appear to be sufficient.

It should be appreciated, of course, that any loose organic pockets or excessively wet soft surface deposits should be removed before embankment fill is placed. It was suggested, also, that piles for the bridge structure should be driven before the embankment construction is begun. In this way, the danger of failure of the fill, due to temporary loss of strength of the alluvial clayey silt, will be avoided.

With regard to the matter of settlement of the structure, there will be a definite long-term tendency for greater movement or settlement at the abutments than at the bridge piers. This is because the weight of fill adjacent to the abutments will tend to produce deep-seated consolidation of the soil below foundation level. However, the abutment settlement should not be great since the pressure increment in the soil will only be about 700 p.s.f. and the foundation material is made up either of dense sand or very stiff clay. A long term movement of the order of 2 inches is anticipated. It is assumed, of course, that the abutments will be founded either on end-bearing cylindrical piles or on footings taken down below the alluvial silts. It is understood that the former proposal is economically more desirable and somewhat less settlement will result if piles are utilized for the abutments. The amount of settlement of the river piers should be of the order of one inch if a pressure of 6000 p.s.f. is utilized for simple footings. The piers must be taken below maximum anticipated scour level and the sides of the footings should be surrounded by gravel-backed rip-rap.

Some settlement of the embankment fill will take place as a result of the consolidation of the alluvial clayey silts. As a consequence, additional load will be transferred by friction to the end-bearing abutment piles as the alluvial soil compresses. Because of the tendency for this negative friction on the piles, the construction of the bridge superstructure should be delayed, if possible, until the embankment fill has had an opportunity to consolidate the alluvial materials. In addition, since horizontal pressures from the fill will be transmitted through the soft silt against the piles, it is recommended that fill be placed or spilled-through to the water side of the abutments.

Paving of the approach fill should not be undertaken until all consolidation has ceased. Because the alluvial soil contains thin layers of sand, drainage and hence consolidation, should take place within a few months.

We hope that the enclosed comments are of assistance to you in resolving the foundation problems at this site. If additional thoughts come to mind please do not hesitate to contact us.

Yours very truly,

*W. A. Trow*

WAT/lt

William A. Trow (P. Eng.)