

#  
64-F-238 C

W.P. #139-64

HWY # 631

HORNE PAYNE  
CREEK

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION			
<h1>HORNEPAYNE CREEK</h1>			
KING'S HIGHWAY NO. 631		LINE A	DIST. NO. 18
DIST. ALGOMA			
TWP. WICKSTEED		LOT	COR.
<b>PROJ. 1409 WILLIAM A. TROW AND ASSOCIATES LIMITED</b>			
SUBWD	CHECKED DMS	WP. NO. 139 - 64	FOR DRAWING NO.
DRAWN: EPK	CHECKED	JOB NO.	
DATE		SITE NO.	FOR DRAWING NO.
APPROVED		CONT. NO.	
<small>REGISTERED PROFESSIONAL ENGINEER</small>			

23-64-365

Mr. A. M. Teye,  
Bridge Engineer,  
Bridge Division.

Foundation Section,  
Materials & Testing Div.,  
Room 107, Lab. Bldg.

Attention: Mr. S. McCombie

July 3, 1964

FOUNDATION INVESTIGATION REPORT BY:  
William A. Trow Associates Limited,  
Proposed Bridge, Hwy. 631 Hornepayne  
Creek, 25 Miles South of Hornepayne,  
Ontario. District 16, W.P. 139-64.

W.P. 139-64

Attached, please find the above-mentioned report submitted by Wm. A. Trow & Associates. We have reviewed the report and found the factual information adequate and well presented. We are also in agreement with the conclusions and recommendations put forward by the Consultant.

Should additional information be required, please feel free to call on our Office.

AGS/Wdef

Attach.

cc: Messrs. A. M. Teye (2)  
H. A. Tregaskes  
H. D. McMillan  
H. Hurrell  
J. D. Foster  
E. P. Saint  
A. Watt

Foundations Office  
Gen. Files

*A. G. Sternac*  
A. G. Sternac,  
PRINCIPAL FOUNDATION ENGINEER

1850 Jane Street  
Weston, Ontario  
241-4644

**William A. Trow**

Project: J1409

Soil Mechanics  
Consultants  
W. A. Trow  
MSc. MEIC. P. Eng.  
K. Peaker  
PhD. MEIC. P. Eng.  
D. H. Shields  
PhD. MEIC. P. Eng.



**Associates Ltd.**

July 1, 1964

Mr. A. Rutka, P. Eng.,  
Materials and Research Engineer,  
Materials and Research Division,  
Department of Highways of Ontario,  
Downsview, Ontario.

Attention: Mr. A.G. Stermac, P. Eng.  
Principal Foundations Engineer.

Re: Foundation Investigation  
Proposed Bridge  
WP 139-64, Sec. Hwy. 631 Hornepayne Creek  
25 Miles South of Hornepayne, Ontario

Dear Sirs:

In conformance with your authorization of April 2, 1964, we have carried out an investigation of the subsoil conditions at the above bridge site. The field work was performed between April 20th, and May 5th, and consisted of 5 sampled borings, one of which went to a maximum depth of 110 feet. In view of the straightforward soil conditions, we take the liberty to be brief in the submission of our findings to you.

SOIL CONDITIONS

The soil conditions at the various test locations are outlined in the borehole logs, Dwgs. 1 to 5, appended to this report. The locations of the holes and an estimated subsoil profile of the



site are contained in the site plan Dwg. From these it is seen that, in general, a deposit of medium dense to dense silt overlies a deposit of sand and gravel till at a depth of 15 to 17 feet. The upper few feet of silt is organic and a surface mantle of muskeg exists at this site. Six feet of disturbed material was found beneath the creek bottom. This probably delineates the depth of scour.

#### SITE DESCRIPTION

As the photographs with this report illustrate, the site is in an area of flat bushland. The creek has no banks to speak of and the water seems to follow the course of a slight depression in the ground surface. At the time of the investigation the creek was in flood with a water level of 1054.6. Even in flood, the flow was only of the order of 36 feet per minute (0.4 miles per hour).

#### FOUNDATION CONSIDERATIONS

Short timber piles driven to end bearing refusal at shallow depth in the very dense till appears to be the most suitable type of foundation for the proposed bridge structure. Piles of the order of 25 to 30 feet will be required. Untreated timber piles can be used and they should be capped below the permanent low water level i.e. the bottom of the shallow creek at elevation 1051.

Even though it is recommended that the piles be driven to refusal, the safe capacity of piles driven below elevation 1027 can be estimated from the expression:



$$Q_a = \frac{Y \cdot D \cdot N}{F} \times A$$

where:

- Qa = the allowable load per pile
- D = the depth of the pile tip below the level of the creek bottom, i.e. elevation 1051.
- Y = the submerged unit weight of the soil above the founding level, 70 pcf approximately.
- N = a bearing capacity factor, estimated to be equal at least to 150 for the very dense till
- F = the factor of safety, 3 is suggested.
- A = the area of the pile tip.

This expression reduces to:

$$Q_a = \frac{7 \cdot D \cdot A}{4}$$

where:

- D is in feet
- A is in square feet
- and Qa is in tons.

For example a pile with a 10 inch tip driven to elevation 1025, or about 25 feet below the creek bottom, would have a safe capacity of at least,  $Q_a = \frac{7}{4} \times 25 \times \frac{\pi}{4} \left(\frac{10}{12}\right)^2 = 24$  tons. Friction of the soil

along the sides of the pile would add slightly to this capacity. Because the value of 24 tons approaches the allowable limit for a timber pile, nothing would be gained by driving the piles butt down to enlarge the bearing area.



Because of the abrupt way the dynamic cone penetration tests reached refusal, the driving of each pile should be observed carefully so that the pile is not damaged by overdriving. As an indication of refusal, driving should not exceed 6 blows per inch under 8750 ft.lb. hammer blows.

An alternative does exist whereby spread footings could be placed on the medium dense to dense silt. Because of the difficulties that would be associated with excavation below the water level in the silt, this alternative is not recommended. Excavation would have to be to a depth of at least El. 1045, i.e. a foot or two below the depth of scour, and sheet piling would have to be used to provide additional protection against undermining.

The settlement of either spread footings on the silt or piles to the very dense till will be well within tolerable limits.

#### OTHER ENGINEERING CONSIDERATIONS

It is recommended that the shallow, surfacial deposit of organic soil and muskeg be removed from under the approaches to the bridge. The maximum depth of material to be removed is about 18 inches according to boreholes 2 and 4. There appears to be no stability problems associated with the low approach embankments over the essentially granular soil.

The abutments must be designed to resist the earth pressure which will be exerted by the approach fills and the natural ground.



The worst condition will arise if scour takes place in front of the abutment. The earth pressure,  $p$ , at any depth,  $h$ , below the top of the abutment can be estimated from the expression:

$$p = K (\gamma (h-h_1) + \gamma^1 h_1 + q)$$

where:  $K = 0.35$  the recommended earth pressure coefficient for the condition that a normal factor of safety of 3 is used in the structural design.

$\gamma = 130$  pcf the estimated unit weight of the retained soil

$\gamma^1 = 70$  pcf the estimated submerged unit weight of the retained soil

$h_1$  = the height of water table above the point being considered.

$q$  = the surcharge, if any, acting at the top of the wall

The above expression is limited to the condition that the water level behind the wall is always equal to the creek level. To ensure that this is so, free draining material must be used for the backfill and drains must be provided to carry away any water. In this way, there will be no build-up of hydrostatic pressure during rainstorms or spring thaw conditions.

Should any question arise regarding this work we would appreciate your call. Thank you for the opportunity to be of service to you.

Yours very truly,

DHS/bs.  
Encls.

D.H. Shields, P. Eng.





## APPENDIX A

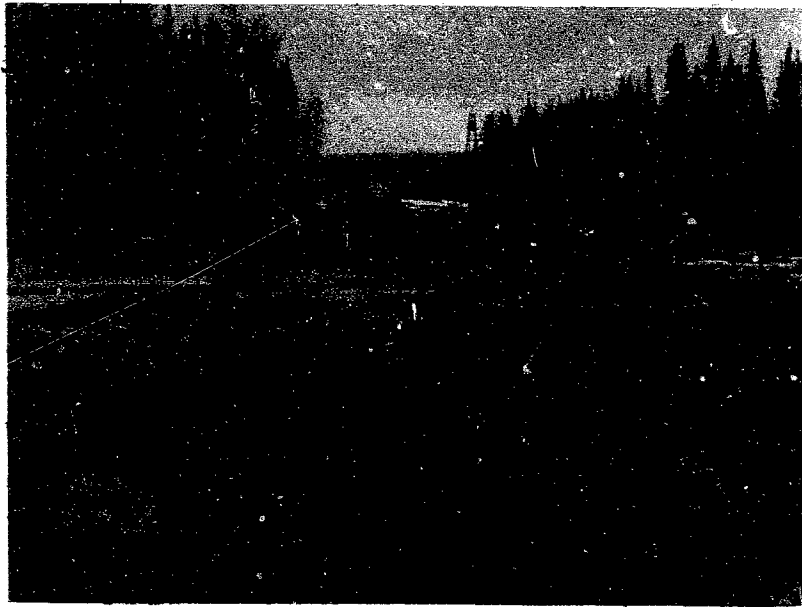
### FIELD INVESTIGATION PROCEDURES

The field work on this project was performed with the aid of two standard diamond drilling machines which were equipped for soil sampling. Standard NX casing was driven to the required sampling depth and then washed clean. A sample of the undisturbed soil below the casing was obtained with a 2 inch O.D. split sampler or a 2 inch I.D. Shelby tube as required. The samplers were either pressed into the ground or driven in conformance with the specifications for the Standard Penetration Test. After the sample was recovered, the casing was driven to the next sampling interval. This procedure was repeated until the desired boring depth was reached or until refusal to driving the casing. Standard BX casing was drilled from the refusal depth if the boring had to go deeper.

A dynamic cone penetration test was performed next to each of the borings. In this test, a 2 inch diameter 60° apex angle cone was driven from the ground surface. The number of blows per foot of penetration of the cone was recorded.

Specimens of rock encountered by the boring were recovered with AX coring equipment.

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Looking North, Drills on BH 1 (Left), BH 2 (Right)



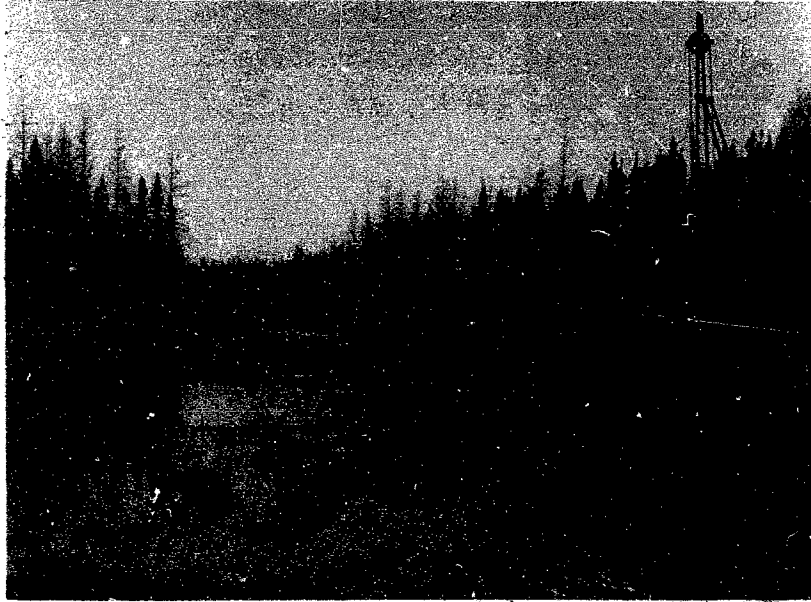
Looking South, Drills on BH 2 (Left), BH 1 (Right)



Looking North, Drills on BH 1 (Left), BH 2 (Right)



Looking South, Drills on BH 2 (Left), BH 1 (Right)



Looking West (Upstream)  
Creek in Flood, Drills on BH-1 (Left), BH 5 (Right)



Looking East (Downstream)  
Drills on Boreholes 1 & 2



Looking West (Upstream)  
Creek in Flood, Drills on BH 1 (Left), BH 5 (Right)



Looking East (Downstream)  
Drills on Boreholes 1 & 2

BOREHOLE NO. 1  
PROJECT Proposed Bridge  
LOCATION Hornby, Ontario  
HOLE LOCATION See Site Plan Dwg.  
HOLE ELEVATION 1053.4 ft.  
DATUM Shown on site plan Dwg.

## PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—  
2" I.D. SHELBY TUBE \* \* \* \* \*  
2" DIA. CONE ————

## SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕  
UNCONFINED COMPRESSION ⊗  
VANE TEST AND SENSITIVITY (S) +

NATURAL MOISTURE CONTENT  
AND LIQUIDITY INDEX

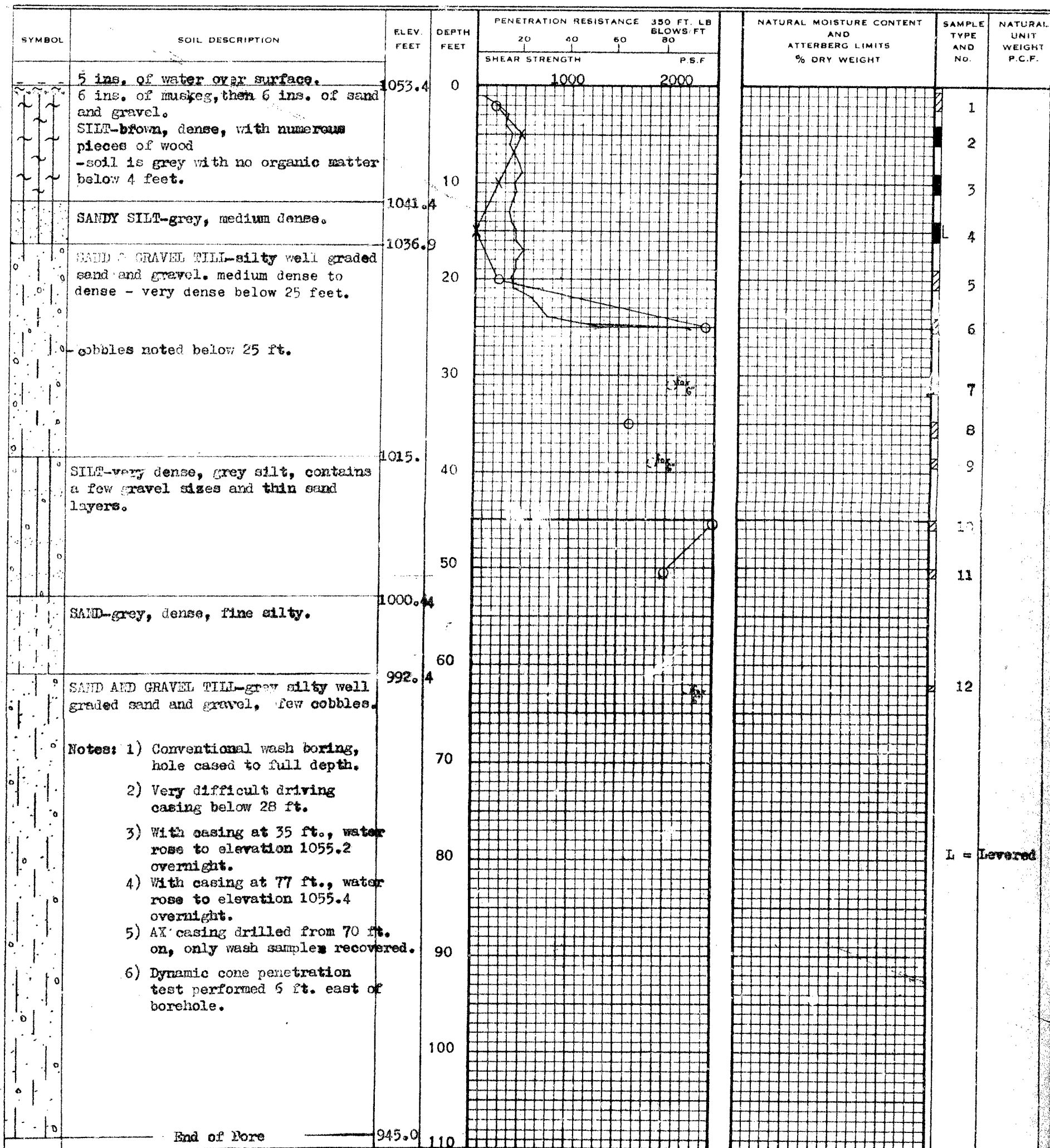
LI  
X

## ATTERBERG LIMITS

LIQUID LIMIT —○—  
PLASTIC LIMIT ———

## SAMPLE TYPE

2" O.D. SPLIT TUBE —■—  
2" I.D. SHELBY TUBE —■—  
3" O.D. SHELBY TUBE —■—



L = Levered

## LEGEND

## PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—  
 2" I.D. SHELBY TUBE —\*—\*—\*—\*—  
 2" DIA. CONE ————

## SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕  
 UNCONFINED COMPRESSION ⊗  
 VANE TEST AND SENSITIVITY (S) ⊕<sup>s</sup>

## NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

LI  
X

## ATTERBERG LIMITS

LIQUID LIMIT —○—  
 PLASTIC LIMIT ————

## SAMPLE TYPE

2" O.D. SPLIT TUBE ————  
 2" I.D. SHELBY TUBE ————  
 3" O.D. SHELBY TUBE ————

BOREHOLE No. 2.

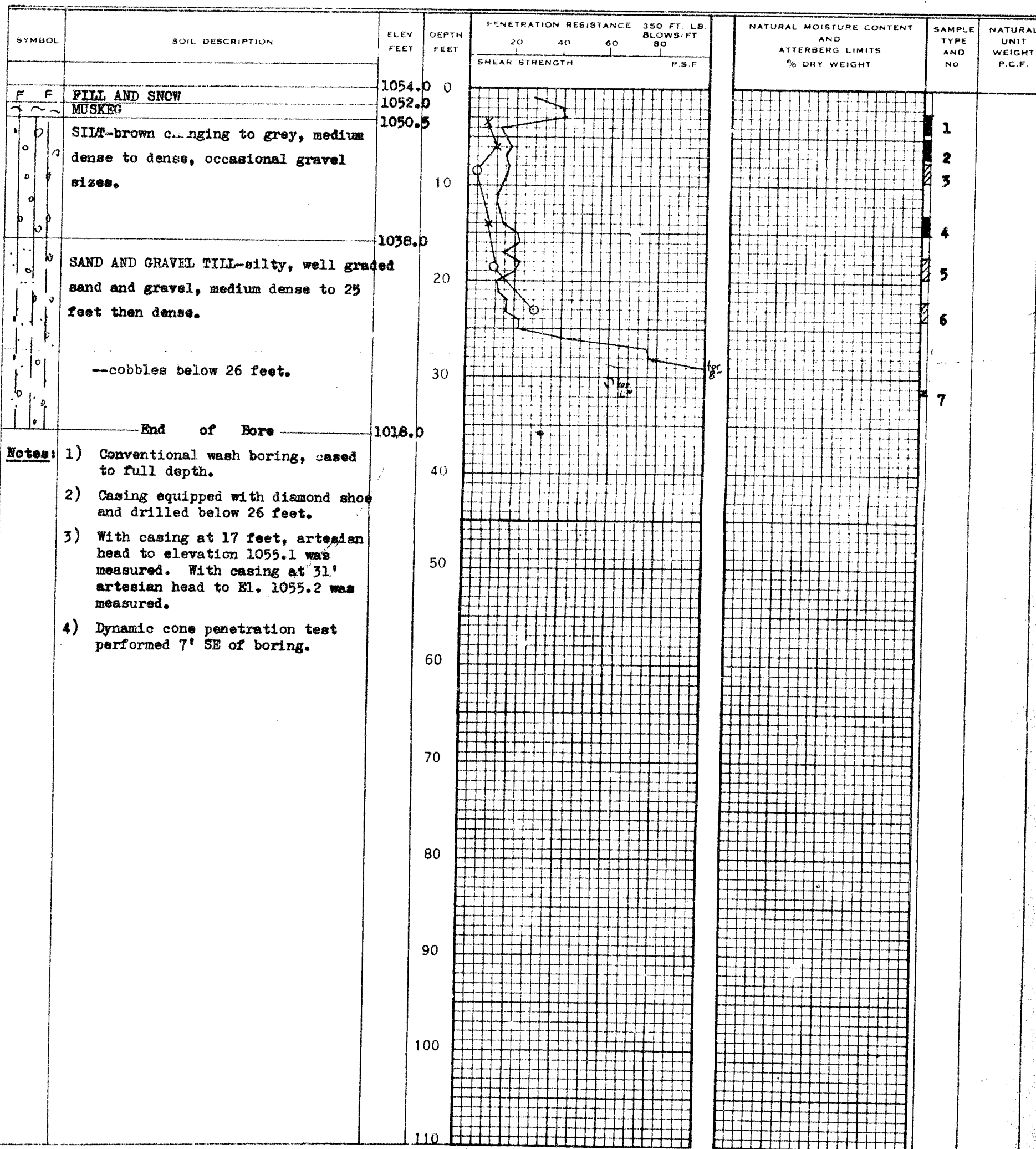
PROJECT Proposed Bridge.

LOCATION Hornepayne, Ontario.

HOLE LOCATION See Site Plan Dwg.

HOLE ELEVATION 1054.0 ft.

DATUM Shown on Site Plan.



## SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

PROJECT No. J1409

### PENETRATION RESISTANCE

## SHEAR STRENGTH

UNDRAINED TRIAXIAL  
AT OVERBURDEN PRESSURE 

### UNCONFINED COMPRESSION

VANE TEST AND SENSITIVITY (S) +

 $x^L$ 

## LIQUID LIMIT

PLASTIC LIMIT

**SAMPLE TYPE**

2" O.D. SPLIT TUBE \_\_\_\_\_  
2" I.D. SHELBY TUBE \_\_\_\_\_  
3" O.D. SHELBY TUBE \_\_\_\_\_

2" I.D. SHELBY TUBE

3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT.		NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.		
				20	40				60	80
				SHEAR STRENGTH P.S.F.						
				1000	2000					
=====	WATER	1053.5	0							
FILL	silty sand and gravel with a large quantity of wood and other organic material.	1052.5								
SILT	grey, medium dense, contains few gravel sizes.	1046.3	10							
SAND AND GRAVEL	TILL-grey, silty, well graded sand and gravel, medium dense.	1039.5	20							
End of Bore		1028.5	40							
Notes:	1) Conventional wash boring, cased to full depth. 2) Difficult driving casing below about 22 ft. 3) Dynamic cone penetration test performed 5 feet south of borehole.			I = Levered						



# WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

DRAWING No. 4.  
PROJECT No. J1409

## LEGEND

### PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—  
2" I.D. SHELBY TUBE —x—x—x—x—  
2" DIA. CONE —————

### SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕  
UNCONFINED COMPRESSION ⊗  
VANE TEST AND SENSITIVITY (S) +




NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

X<sup>LI</sup>

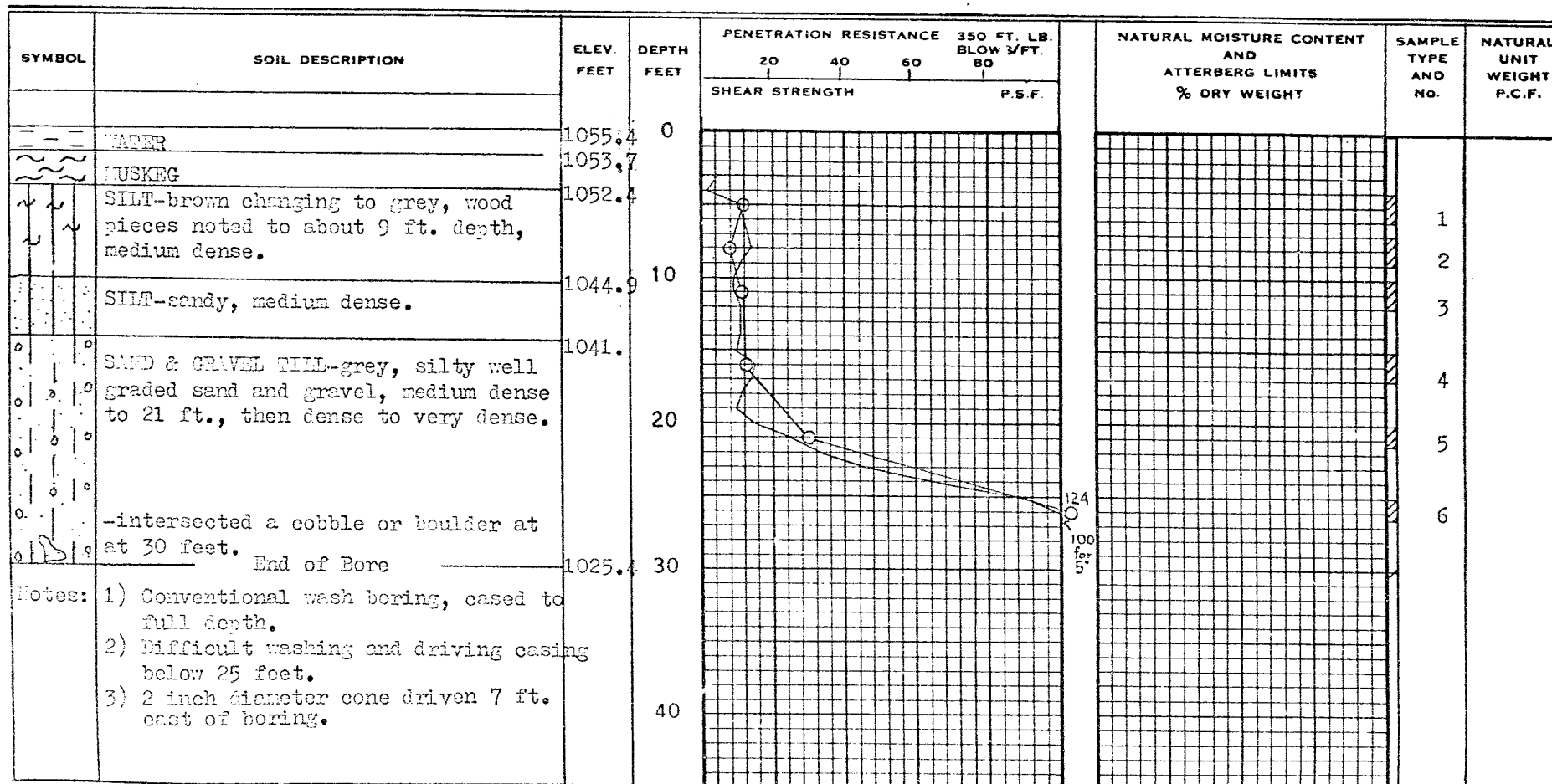
### ATTERBERG LIMITS

LIQUID LIMIT —○—  
PLASTIC LIMIT ———

### SAMPLE TYPE

2" O.D. SPLIT TUBE   
2" I.D. SHELBY TUBE   
3" O.D. SHELBY TUBE 

BOREHOLE No. 4  
PROJECT Proposed Bridge  
LOCATION Hornepayne, Ontario  
HOLE LOCATION See Site Plan Dwg.  
HOLE ELEVATION 1055.4 ft.  
DATUM Shown on site plan Dwg.



## LEGEND

BOREHOLE NO. 5.  
PROJECT Proposed Bridge.  
LOCATION Hornepayne, Ontario.  
HOLE LOCATION See Site Plan Dwg.  
HOLE ELEVATION 1054.2 ft.  
DATUM Shown on Site Plan Dwg.

## PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—  
2" I.D. SHELBY TUBE —x—x—x—x—  
2" DIA. CONE ————

## SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊗  
UNCONFINED COMPRESSION ⊗  
VANE TEST AND SENSITIVITY (S) ⊕

## NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

## ATTERBERG LIMITS

LIQUID LIMIT —○—  
PLASTIC LIMIT ———

## SAMPLE TYPE

2" O.D. SPLIT TUBE ⊗  
2" I.D. SHELBY TUBE ⊗  
3" O.D. SHELBY TUBE ⊗

