

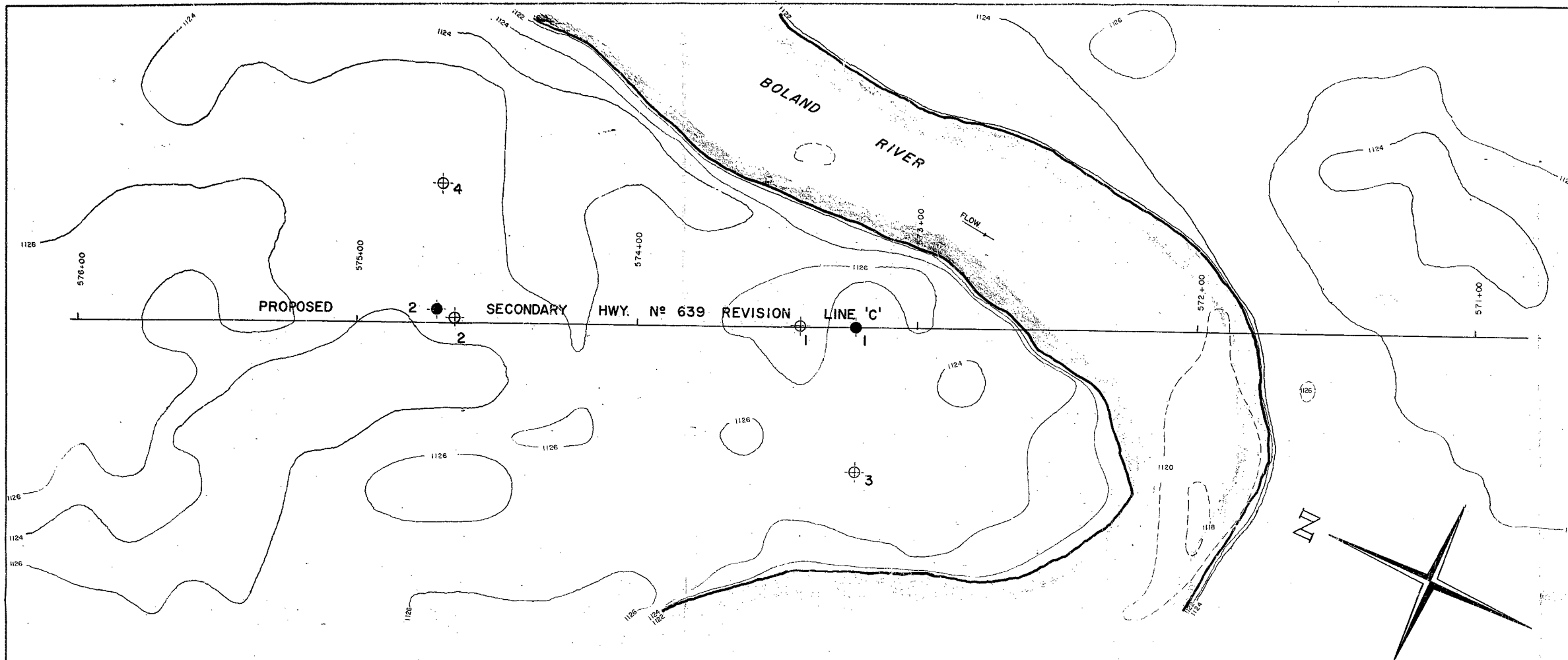
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W.P. 196-63

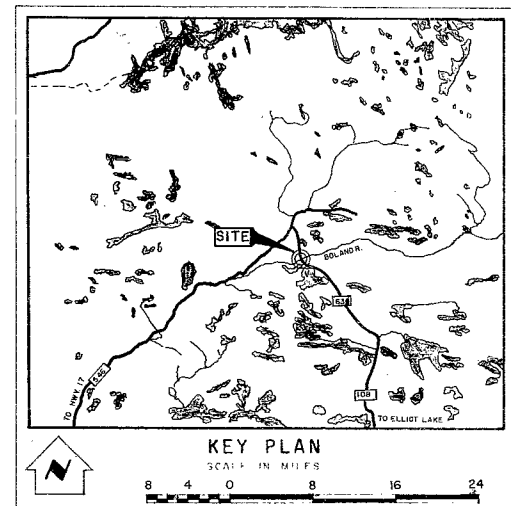
Hwy #639

BOLAND RIVER

BRIDGE

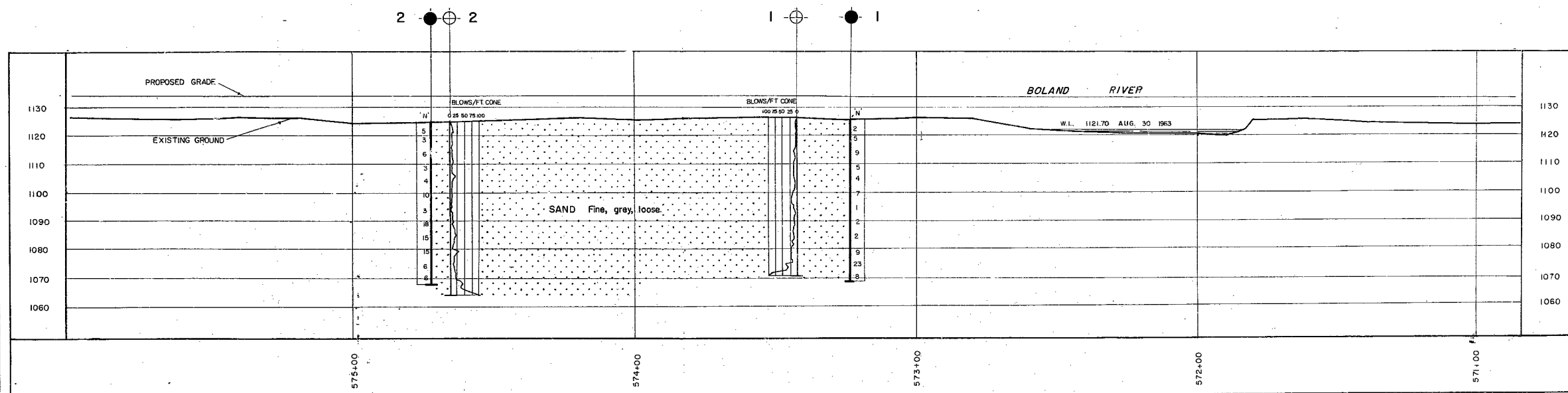


PLAN & PROFILE  
SCALE IN FEET  
20 10 0 20 40



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation		
NO.	ELEVATION	STATION	OFFSET
1	1125-2	573+22	0
2	1124-3	574+72	5' RT.

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



PRINT RECORD		
NO.	FOR	DATE

REVISIONS	DATE	BY	DESCRIPTION

**WILLIAM A. TROW AND ASSOCIATES LIMITED, J 1244**

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

**BOLAND RIVER**

KING'S HIGHWAY NO. 639 LINE 'C' REVISION DIST. NO. 17  
DIST. ALGOMA  
TWP. N. 157 LOT CON.

SUBM'D	CHECKED	W.P. NO. 196-63	M.B.R. DRAWING NO.
DRAWN E.F.K.	CHECKED	JOB NO.	
DATE	SITE NO.	BRIDGE DRAWING NO.	
APPROVED	CONT. NO.		

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

Attention: Mr. S. McCoshie

Mr. A. G. Sternac,  
Principal Foundation Engr.,  
Foundation Section,  
Materials & Research Division.

November 15, 1963

**FOUNDATION INVESTIGATION REPORT BY  
WILLIAM A. TROW & ASSOCIATES LTD.**

**Boland River Bridge, Hwy. #639,  
W.P. 196-63, District #17, Sudbury.**

Attached, we are forwarding to you, the above-  
mentioned report submitted by Wm. A. Trow and Associates of  
Toronto.

We have reviewed the report and are of the opinion  
that it contains all the information necessary for you to proceed  
with further design. Should there be any questions that you  
would like to discuss, please feel free to call on our office.

AGS/MdeF  
attach.

cc: Messrs. A. M. Toye (2)  
H. A. Tregaskes  
H. D. McMillan  
H. McArthur  
T. A. Sharpe  
E. H. Saint  
J. Watt

Foundations Office  
Gen. Files

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

Materials and Research Division

November 14, 1963

William A. Trow & Associates, Ltd.,  
1850 Jane Street,  
Weston, Ontario.

Attention: Mr. Wm. A. Trow

Re: W.P. 196-63, Hwy. 639, Boland River Bridge,  
District No. 17, Sudbury, Ontario.

Dear Sir:

Please consider this as confirmation of our earlier verbal agreement, and as your formal authority to have carried out a foundation investigation at the above site.

Charges for the work performed will be in accordance with your Schedule of Rates, dated November 19, 1962, and invoice to be addressed to the attention of the undersigned.

Yours very truly,

RA /MdeF

cc: Messrs. R. D. Smith (2)  
T. Tate (Accounts)  
Foundations Office  
Gen. Files (2)

*A. J. Rutka*  
A. J. Rutka,  
MATERIALS & RESEARCH ENGINEER

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

attention: Mr. S. McVoshie

Mr. A. G. Stermac,  
Principal Foundation Engr.,  
Foundation Section,  
Materials & Research Division.

November 15, 1963

FOUNDATION INVESTIGATION REPORT BY  
WILLIAM A. TROW & ASSOCIATES LTD.  
Boland River Bridge, Hwy. #639,  
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with further design. Should there be any questions that you  
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AG-/MdeF  
attach.

cc: Messrs. A. M. Toye (2)  
H. A. Tregaskes  
B. D. McMillan  
H. Scarthur  
I. A. Charpe  
G. L. Saint  
G. Watt

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

Foundations Office  
Gen. Files

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,  
Principal Foundation Engineer,  
Room 107, Lab. Bldg.,

FROM: J. C. McAllister

DATE: January 28, 1964.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 196-63 - Boland River Bridge  
Sec. Rd. #639 - District #17

Attached please find one print of preliminary plan D-5423-P for the above structure.

The structure has been designed in accordance with the recommendations of the foundation report. The timber piles will have an embedment of 30' with a load of 13 tons/pile. Your recommendation on a pile load test would be appreciated.

*J. C. McAllister*

JCMcA/bm  
c.c. J. Walter

J. C. McAllister,  
for S. McCombie,  
Bridge Planning Engineer.

ADVISED Mr. McALLISTER THAT THE ABOVE  
MENTIONED PROBLEM WAS DISCUSSED WITH  
Mr. W. McFARLANE AND PILES DRIVEN DOWN TO  
ELEV. 1085.0 CAN CARRY LOADS OF 15 TONS.  
FEB. 3 1964

*AES*

## WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS  
LABORATORY TESTING  
SOIL MECHANICS CONSULTATION

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

1850 JANE ST.,  
WESTON, ONT.  
CH. 1-4644

Project: J 1244

November 11, 1963.

Mr. A. Rutka, P. Eng.,  
Materials and Research Engineer,  
Materials and Research Division,  
Dept. of Highways of Ontario,  
Parliament Buildings,  
Toronto, Ont.

Attention: Mr. A. G. Stermac, P. Eng.

Foundation Conditions  
Boland River Bridge, Hwy. 639,  
WP 196 - 63

Dear Sirs:

In conformance with the verbal authorization of Mr. K. Selby, given on October 10th, we have made an investigation of foundation conditions at this river crossing which lies on the proposed route of Hwy. 639, near Flack Lake, in Northern Ontario.

The site work involved in this survey consisted of two borings taken to a depth of 55 feet and 4 cone penetration tests made adjacent to the borings.

As we advised you by telephone soon after field information became available, the soil at this river crossing consists of fine sand at least to the depths of the borings and probably to much greater depths. According to penetration measurements, the sand should be

described as 'loose', although subsequent laboratory test measurements suggest that it is, in fact, 'dense'. By driving a cone in the bottom of the holes, it was found that the same conditions prevailed to 70 feet and 90 feet, in holes 2 and 1, respectively.

In view of the susceptibility of the sand to erosion and the close proximity of the ground to river level, the only reasonable foundation scheme is to support the bridge on timber piles. Although the safe load per pile can be determined accurately only by load tests, approximate field and laboratory tests indicate that a 20 ton load can be developed by a timber pile driven 50 feet into the sand.

The factual information and soil mechanics reasoning which form the basis for these recommendations are considered under the sections that follow.

#### SITE

The site of this crossing lies in a flat heavily wooded area located about 0.5 miles north of Flack Lake and the Laurentian Lodge. Access to the project was made by aircraft from Algoma Mills and by tractor from the lodge. No bedrock or boulders were in evidence in the general area.

The creek meanders through this bush country and at this location it had an average measured flow of 1 foot per second. The water level was about 4 feet below the general ground surface.

Photographs of the site are included in this report.



FIELD WORK AND SUBSOIL

The borings of the investigation were made using light diamond drill equipment, which was flown to the site by a Beaver Aircraft. The holes were cased to 55 feet depth with BX pipe and sampling was terminated at this depth because the light equipment did not have sufficient power to drive or to withdraw pipe taken to greater depth. Conventional split spoon samples were taken in each hole to this depth and a cone was driven below casing level to a maximum depth of 98 feet. Cone penetration tests were also made adjacent to the holes and at the two other locations shown on the site plan. All samples and cone tests were driven using an energy of 350 ft. lbs. per blow. All samples were wrapped in tin foil and in sealed plastic bags to prevent moisture loss.

Static penetration tests were made at 4 levels in hole 2 using a 1.3 inch O.D. E rod 5 feet long as a probe and applying load by levering on a 10 foot A rod. The test arrangement and results of these tests are shown on Dwg. 3. They were made in order to obtain an approximate indication of the end bearing capacity and shaft friction available to piles.

The soil in both holes was found to consist of medium to coarse sand for the first 10 feet of depth and then fine sand below. According to penetration test results, it is in a very loose condition, although this empirical indication of relative density was not confirmed by laboratory tests.

Moisture content tests were made on each sample in order to obtain an indirect indication of in place density. This determination is made from the expression:

$$\gamma_{dry} = \frac{62.4S}{1+WS}$$

where:

$\gamma_d$	is the dry unit weight of the sand
S	is its specific gravity, determined in one test to be 2.67
W	is the moisture content expressed as a decimal.

This equation is valid for a saturated soil. Although it is possible that some moisture was lost in the samples of coarser sand, most of the measurements are considered to be valid.

Relative density measurements were made on two composite samples of sand from hole 2, 20 to 25 feet and 45 to 50 feet. The results of these tests are shown in computation sheet No. 1. It is seen that the relative density of most of the samples lies within the dense to very dense range, as described by the Bureau of Reclamation.\*

Typical gradings of the sand from hole 2 are indicated in Dwg. 4.

#### FOUNDATIONS

According to the laboratory determinations of relative density, the sand should be in a much more compact state than the empirical field penetration tests suggest. Therefore it could be argued that the support of the bridge on simple footings should be possible, particularly if the footing base first were to be stabilized by compaction with vibrating roller equipment. However, in view of the close proximity of the water table, - which would make compaction difficult, - and the vulnerability of the sand to erosion, it is recommended that the structure be supported on timber piles.

The safe working capacity of a friction pile can be determined with accuracy only by the performance of a pile load test. However, in order to provide some guidance for preliminary planning, some simple crude static

\* "Research on Determining the Density of Sands by Spoon Penetration Testing" Holtz & Gibbs - Fourth Int. Conf. on Soil Mechanics & Foundation Engineering, 1957.

penetration and pull out tests were made on an E drill rod at four levels in hole 2. The results of these tests are shown on Dwg. 3, and a consideration or examination of this information is given in the appendix. As a result of this approximate analysis, values of bearing capacity factor  $N = 80$  and shaft friction factor  $k = 0.22$  have been recommended. The value of  $k$  is considered to be low for a timber pile, since it was determined on a steel shaft of small displacement.

The ultimate capacities of timber piles having 8 inch tips and an average diameter of 10 inches, have been indicated in Table 1 for various depths of penetration. It is seen that a working load of 20 tons can be developed with a factor of safety just above 3 for a penetration of 50 feet into the sand. It is not expected that refusal will be encountered at this depth or that the pile will be damaged by driving.

The upper portions of the timber piles should be protected from abrasion during flood periods by well graded rip rap or by sheet piling.

No embankment stability problem exists at this site, since the soil is granular to great depth. No other foundation problems are anticipated.

If you have any queries after you have reviewed the contents of this report we shall be pleased to discuss them with you.

Yours very truly,

*W. A. Trow*

William A. Trow, P.Eng.

WAT/gc  
Encls.



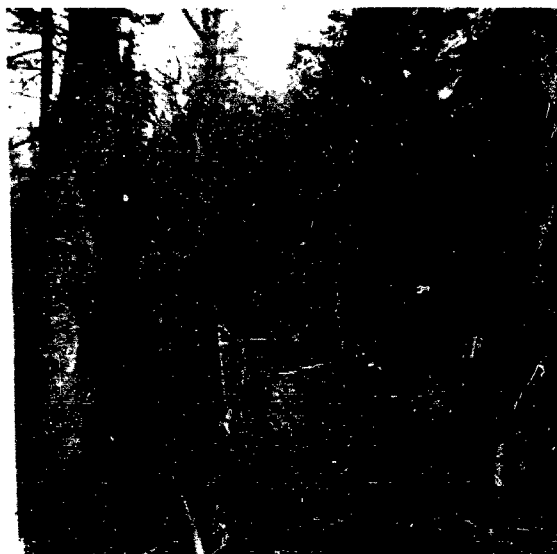
View from the West  
Drill on Hole 2



View from the North.  
Drill on Hole 2.



View looking Upstream  
Drill on Hwy. Centreline.



View looking north along centreline.  
Drill on Hole 2.



View from the west  
looking south. Hole in



View from the North  
looking south. Hole in



View looking eastward  
at hole. Centre line



View looking north along centre line.  
Hole in Hole in

APPENDIXEXAMINATION OF STATIC PENETRATION TEST RESULTS

Drawing 3 consists of a plot of ultimate capacity of a 1.3 inch diameter E rod at various depths of penetration below casing level. Also indicated is the pull-out resistance for three of the tests. By subtracting this pull-out or friction resistance the maximum load in end bearing at the particular final penetration depth is determined.

This maximum load is given approximately and conservatively by the expression:

$$Q = A D N *$$

where:

A	is the end bearing area of the rod in square feet = .0092 sq. ft.
D	is the surcharge weight of soil D feet below the ground surface
N	is a bearing capacity factor

At the lower limit of penetration, which is at least 8 diameters below the casing, the depth D can be taken from the ground surface. Above this lower limit, the proximity of the casing must have increasing effect since the change in ultimate load noted in the tests is too great to be accounted for by bearing capacity factors or friction or a combination of these resistances. The rate of increase of load in the tests at and below 32 feet is equal to or greater than 1500 lbs. per foot, which is equivalent to a bearing capacity factor  $N = 2700$ . This is much higher than is indicated by Meyerhof. In all of the tests below 32 feet, the maximum loading was limited to approximately 2300 lbs., since the machine reaction tended to tip over at this loading. Using this limiting load and subtracting the uplift force to obtain a value for Q in the above expression, the corresponding values for N at depths of 34, 44 and 53 feet were determined to be

\*"Some Recent Research on the Bearing Capacity of Foundations", - Meyerhof, Can. Geotech. Journal

95, 71 and 55 respectively. Since it is not reasonable to expect a variation in  $N$  for this uniform soil condition, it is concluded that all of the tests, and particularly those at 44 and 53 feet were influenced to some extent by the proximity of the casing. If the rod had been able to penetrate to the same depth in all tests, a much higher net capacity would have been measured at these greater depths and consequently a higher value of  $N$  would have been determined. For purposes of the preliminary estimation of pile capacities a value of  $N = 95$  will be assumed.

The magnitude of friction force acting on the rod will vary depending on the proximity to the casing. The friction in the upper levels should be low because of the shielding effect of the casing. An approximate estimate of the unit friction value,  $k$ , was made by projecting the lower straight line portion of the curve back to the zero load ordinate and by assuming that all of the friction,  $F$ , was developed on the length of rod  $l$ , below this line.

The unit friction coefficient  $k$  is determined from the expression:

$$F = A k l D$$

where:  $A$  is the area of the shaft within the length  $l$   
and  $S$  and  $D$  have values as indicated above.

Solving in this expression, the values of  $k$  for the tests at 32, 42 and 52 feet are .213, .255 and .365 respectively.

For purposes of this examination of pile capacity, a friction factor  $k = 0.22$  will be assumed. This lies in the lower range of friction coefficients suggested by Meyerhof\* and therefore it is conservative, particularly when applied to larger displacement timber piles.

\* "The Ultimate Bearing Capacity of Foundations", G.G. Meyerhof, Geotechnique, 1951

TABLE 1

ESTIMATED CAPACITY OF TIMBER PILE AT VARIOUS DEPTHS BELOW SURFACE  
(To be Confirmed by Load Test)

<u>Depth</u>	<u>Safe Capacity (F.S.=3)</u>
20	8.4
30	13.6
40	19.4
50	25.8

Assumptions:

Ultimate capacity Q given by expression:

$$Q = A(0.3 \gamma_{BN} + D N) + P k \frac{1}{2} \gamma D^2$$

where:

A = 0.35 sq.ft. is the end bearing area for a pile of tip diameter B = 0.67 ft.

$\gamma$  = 130 pcf above the water table and 65 pcf below

D is the depth of penetration into the soil in feet

N = 95, the estimated bearing capacity factor from appendix.

P = 2.62 ft. is the perimeter of a timber pile of average shaft diameter = 10 ins.

k = 0.22 is the estimated shaft friction coefficient from the appendix



COMPUTATION SHEET NO. 1Relative Density DeterminationsComposite Sample, Hole 2, 20 and 25 feet

Maximum density determined by vibrating dry sand under pressure:

Two tests = 101.3 and 101.7 p.c.f.

Use 101.7 p.c.f. =  $\gamma_{max}$ .

Minimum density by pouring sand into 1000 cc. flask filled with water:

One test = 82.2 p.c.f. =  $\gamma_{min}$ .

In place density from Table below = 101.8 =  $\gamma$

$$\text{Relative density} = \frac{\gamma_{max} (\gamma - \gamma_{min.})}{\gamma (\gamma_{max} - \gamma_{min.})} = 100\%$$

Composite Sample, Hole 2, 45 and 50 feet

Maximum density, two tests as above, = 100.5 and 101.7 p.c.f.

Use 101.7 p.c.f. =  $\gamma_{max}$ .

Minimum density, one test, poured dry into beaker = 87 p.c.f.

one test, poured into water, as above = 82.2 p.c.f.

Use  $\gamma_{min.}$  = 82.2 p.c.f.

Relative density for  $\gamma$  = 102.5 p.c.f. at 45 and 50 ft. = 100%

Note: for  $\gamma_{dry}$  = 94 p.c.f.: Relative density = 65%.

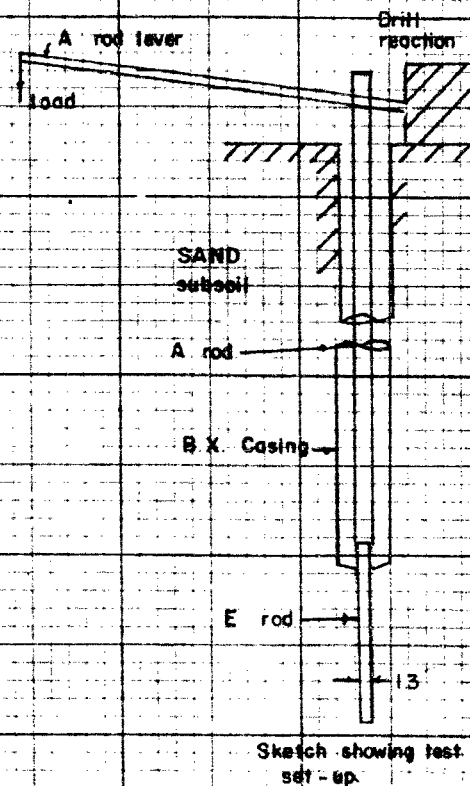
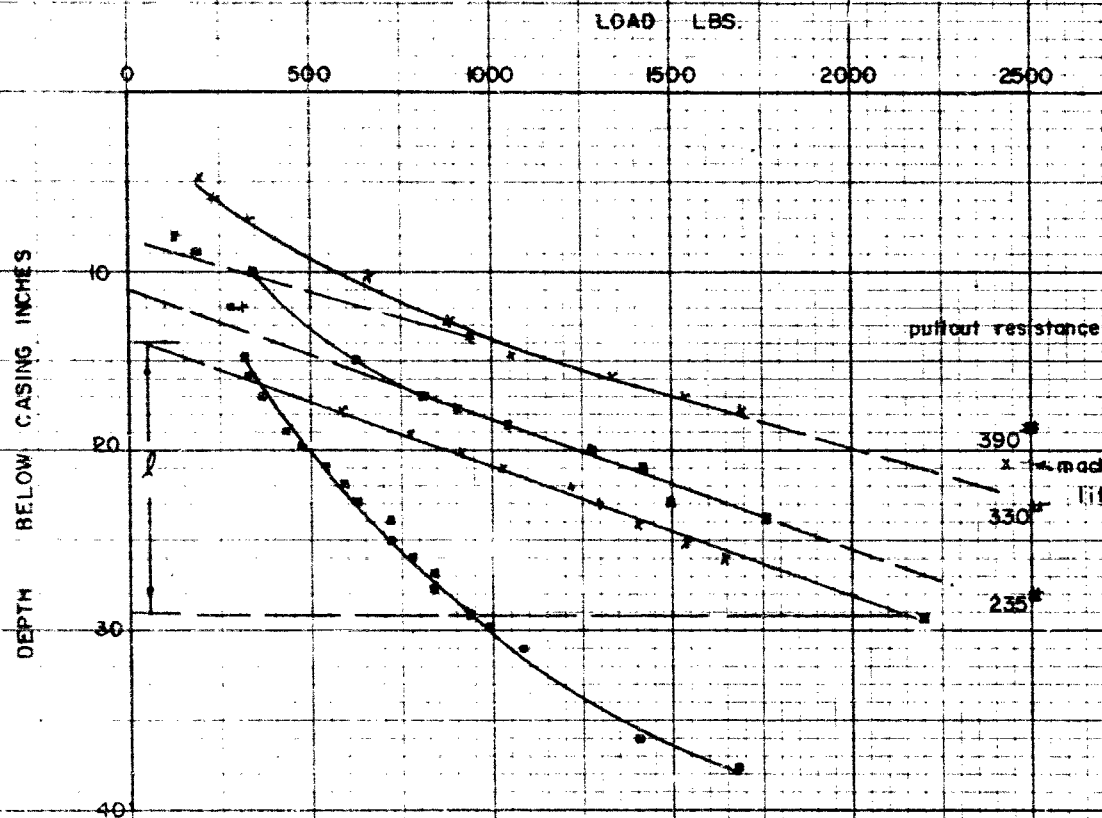
According to Bureau of Reclamation\*, the term dense applies for relative densities from 65 to 85%, and very dense applies for relative densities greater than 85%. Almost all of the densities indicated in the following table fall within these density limits.

Unit Weight Determinations from Moisture Meas.

Depth	<u>Borehole No. 1</u>		<u>Borehole No. 2</u>	
	Moist.W%	Dry pcf	Moist.W%	Dry pcf
2	14.8	119.4	26.2	98.0
5	20.1	108.3	18.5	111.5
10	22.4	104.3	11.8	126.8
15	23.6	102.2	30.8	91.5
20	27.0	96.8	22.5	104.0
25	30.8	91.4	23.9	101.8
30	29.6	93.0	21.2	106.4
35	25.8	98.8	23.0	103.1
40	23.0	103.2	23.2	102.9
45	22.9	103.3	23.4	102.5
50	28.8	94.2	23.2	102.9
55	26.1	98.1	22.5	104.0

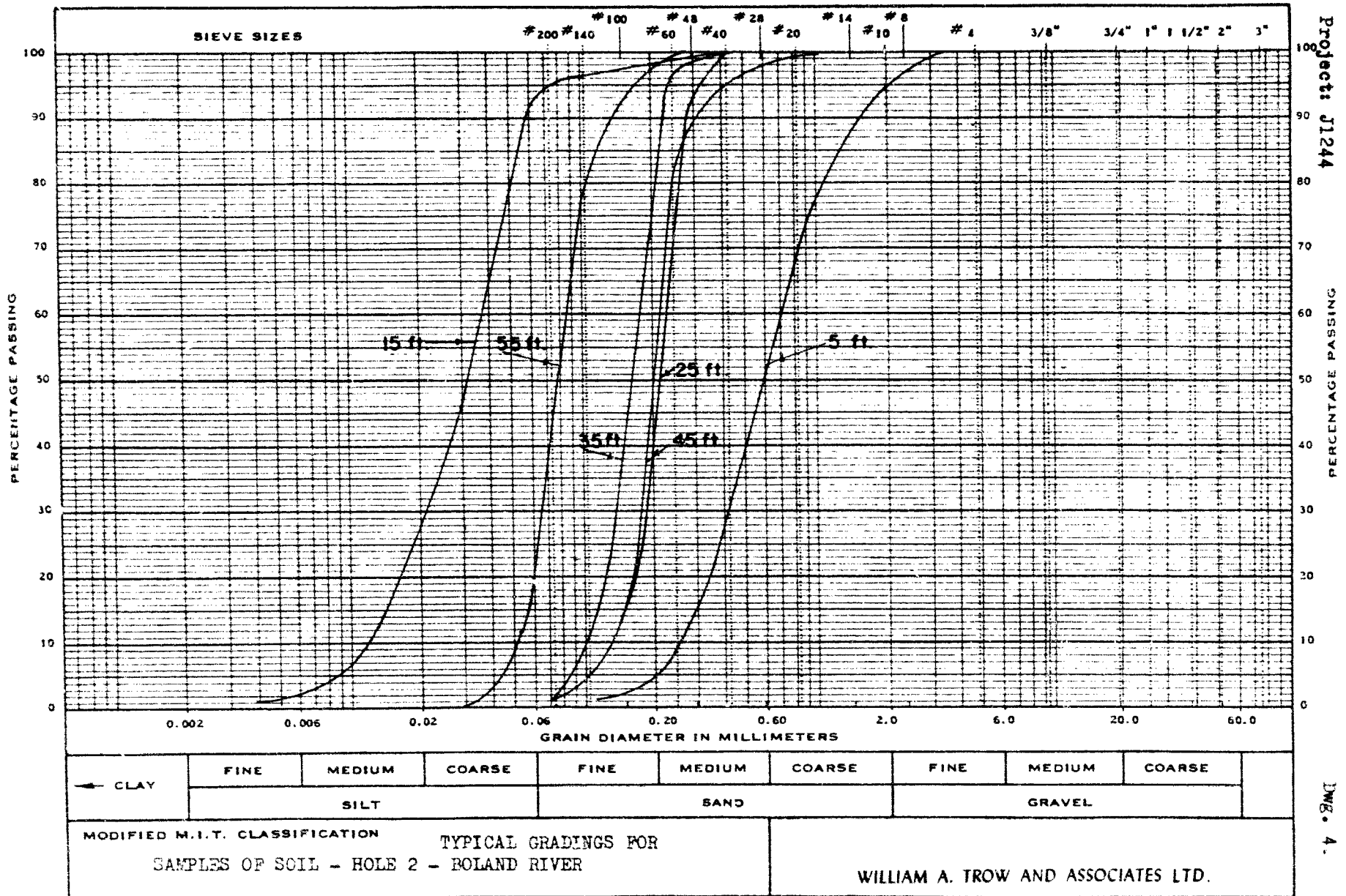
Specific Gravity, S, Hole 2, SS 8 = 2.67.

\* For saturated soil  $\gamma_{dry} = \frac{62.4 s}{1 + W s}$  (W = moist.cont. expressed as decimal)




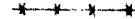
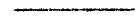
RESULTS OF STATIC PENETRATION TESTS ON E ROD

# MECHANICAL ANALYSIS







BOREHOLE NO 1  
PROJECT Holand River Crossing - Hwy. 639  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 1125.2 ft.  
DATUM See Dwg. 1.

## 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  $\pm$  

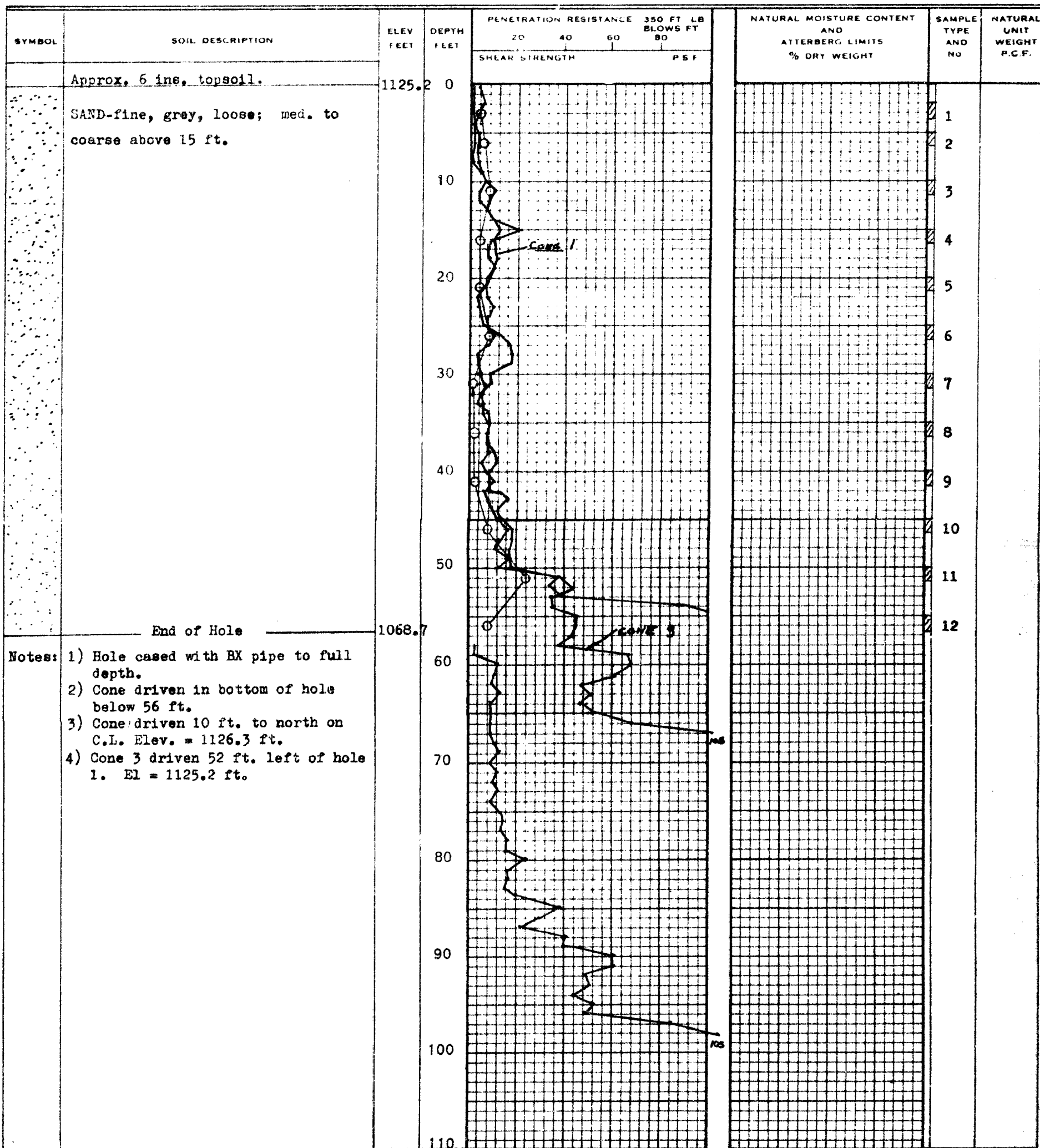
NATURAL MOISTURE CONTENT  
AND LIQUIDITY INDEX 

## ATTERBERG LIMITS

LIQUID LIMIT   
PLASTIC LIMIT 

## SAMPLE TYPE

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



BOREHOLE No. 2  
PROJECT Boland River Crossing, Hwy. 639  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 1124.3 ft.  
DATUM

## 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

## ATTERBERG LIMITS

LIQUID LIMIT

PLASTIC LIMIT

## SAMPLE TYPE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

3" O.D. SHELBY TUBE

X<sup>L1</sup>

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