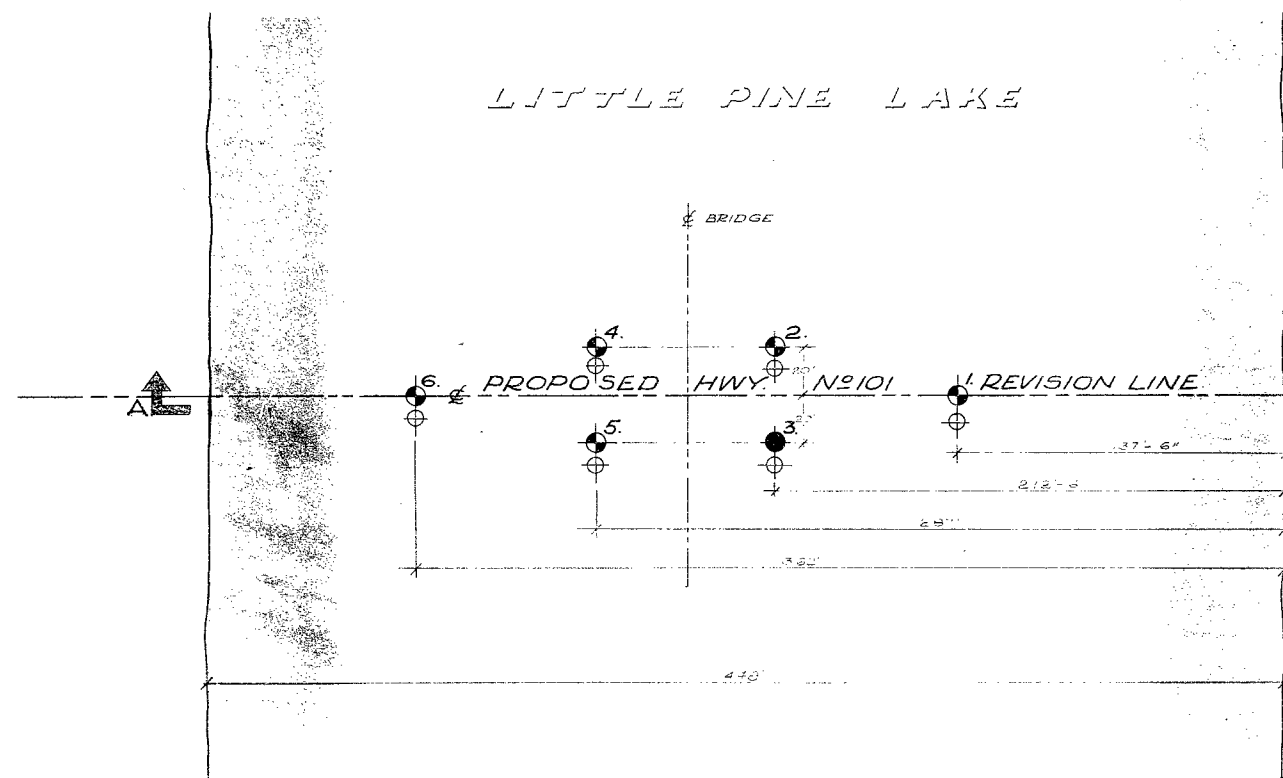
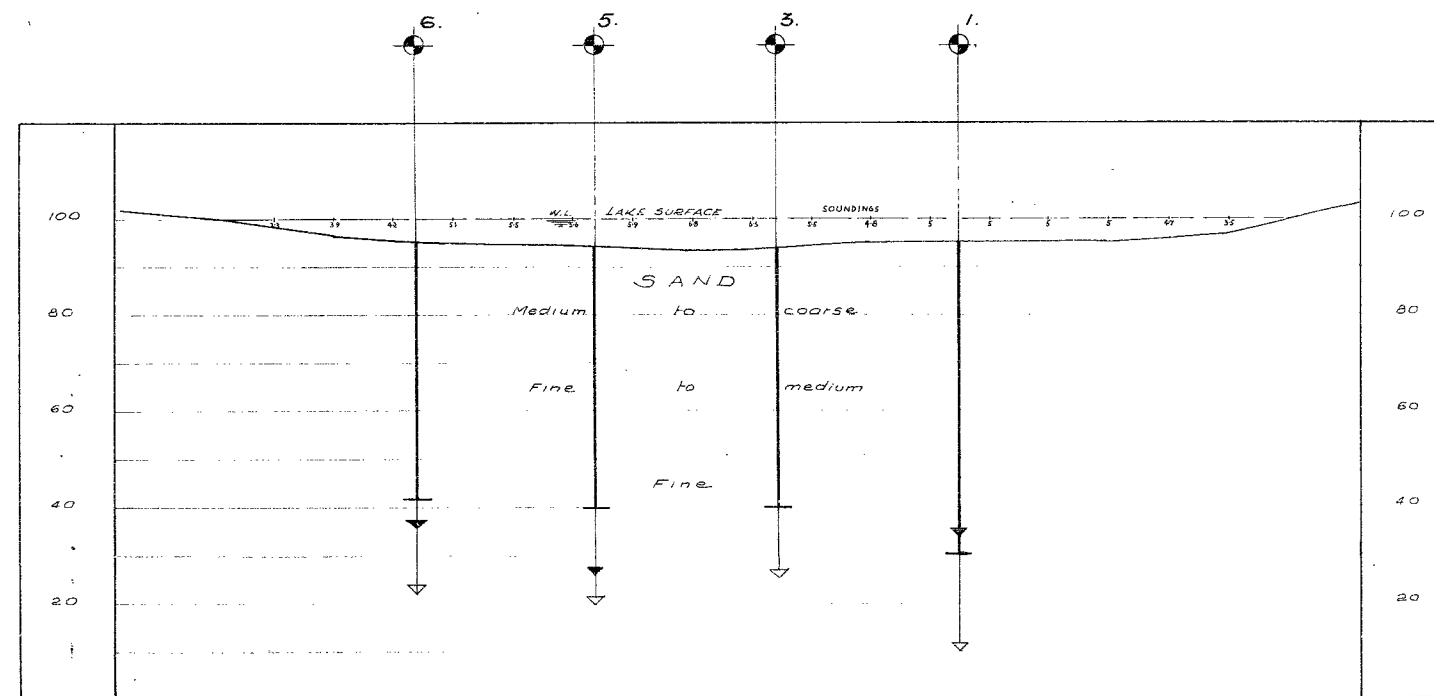
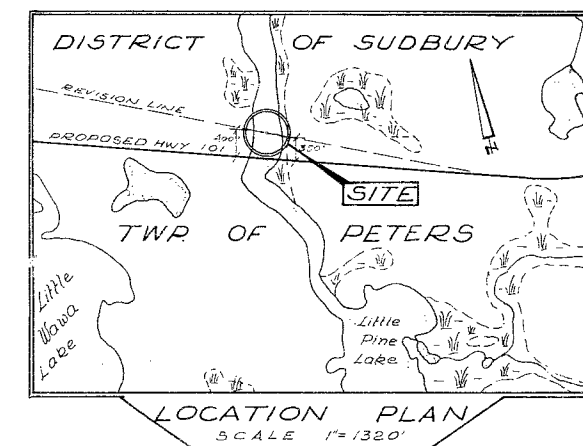
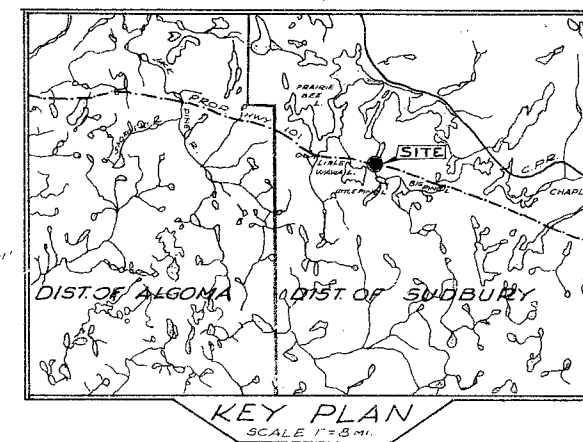


# 62-F-216-C  
W.P. # 148-62  
PROP. CROSSING  
HWY. # 101,  
LITTLE PINE  
LAKE



BORE HOLE LOCATION PLAN  
SCALE 1" = 40'

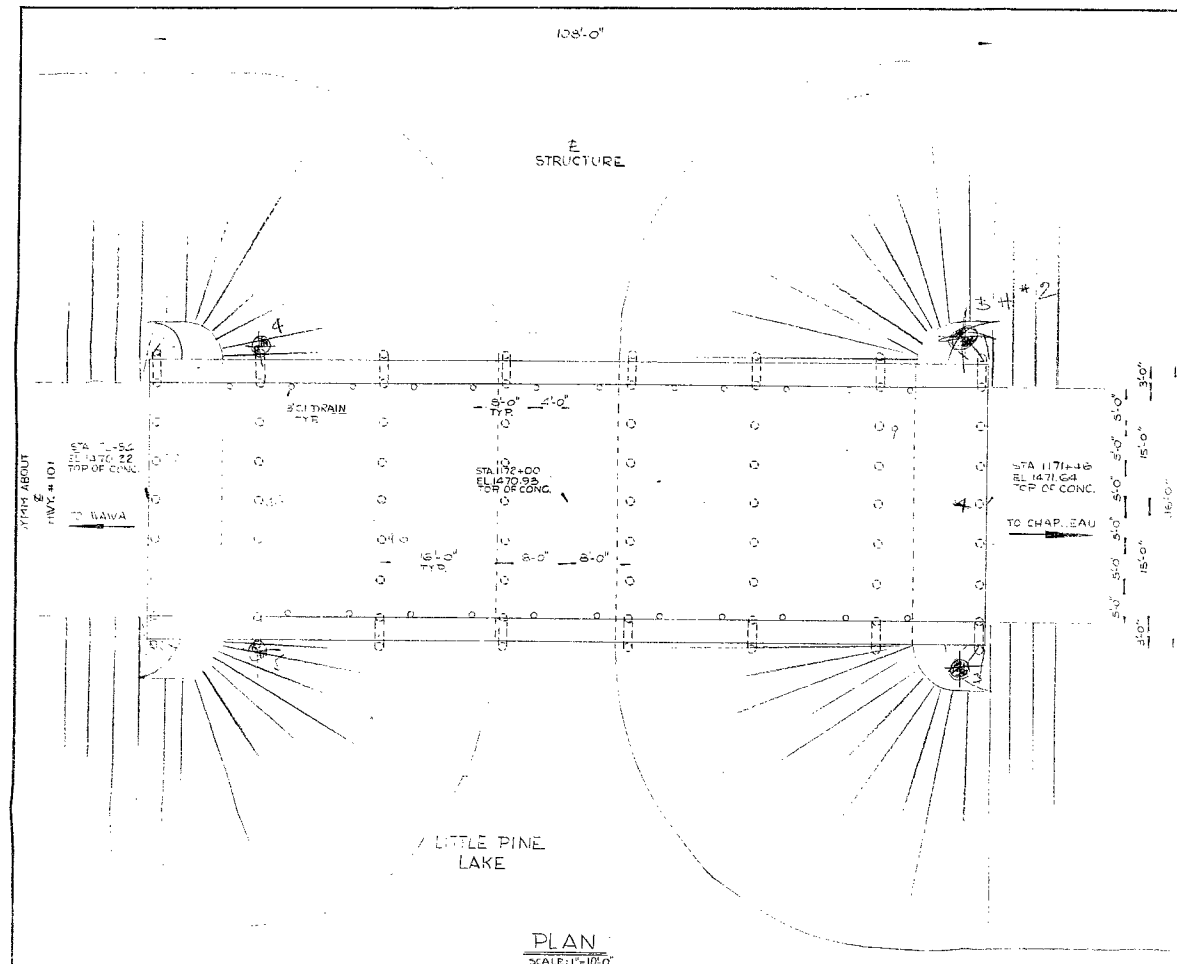


SECTION A-A

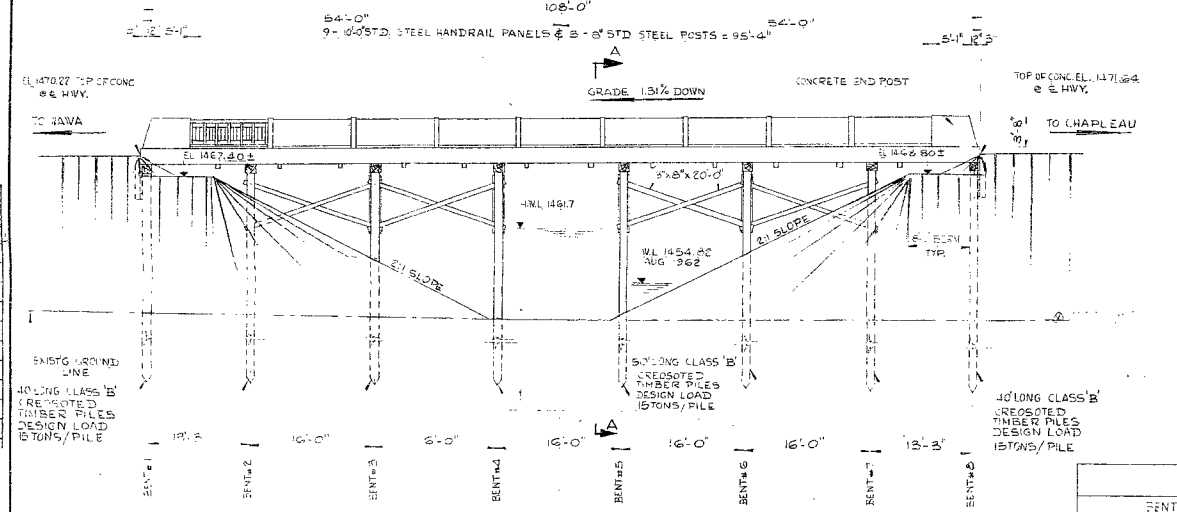
LEGEND	
	BORE HOLE
	BORE HOLE & PENETRATION TEST
	PENETRATION TEST
	END OF BORE HOLE
	END OF PENETRATION TEST IN BORE HOLE
	END OF PENETRATION TEST LOCATED 10' SOUTH OF BORE HOLE

W. A. TROW & ASSOC. LTD.  
FOUNDATION INVESTIGATION  
**LITTLE PINE LAKE & PROPOSED  
HWY. NO. 101 REV. LINE CROSSING**

PROJECT NO. J886 W.P. NO. 187-60 DATE JUNE 1962 DWG. NO. 1.



PLAN  
SCALE: 1"=10'-0"



SOUTH ELEVATION  
SCALE: 1"=10'-0"

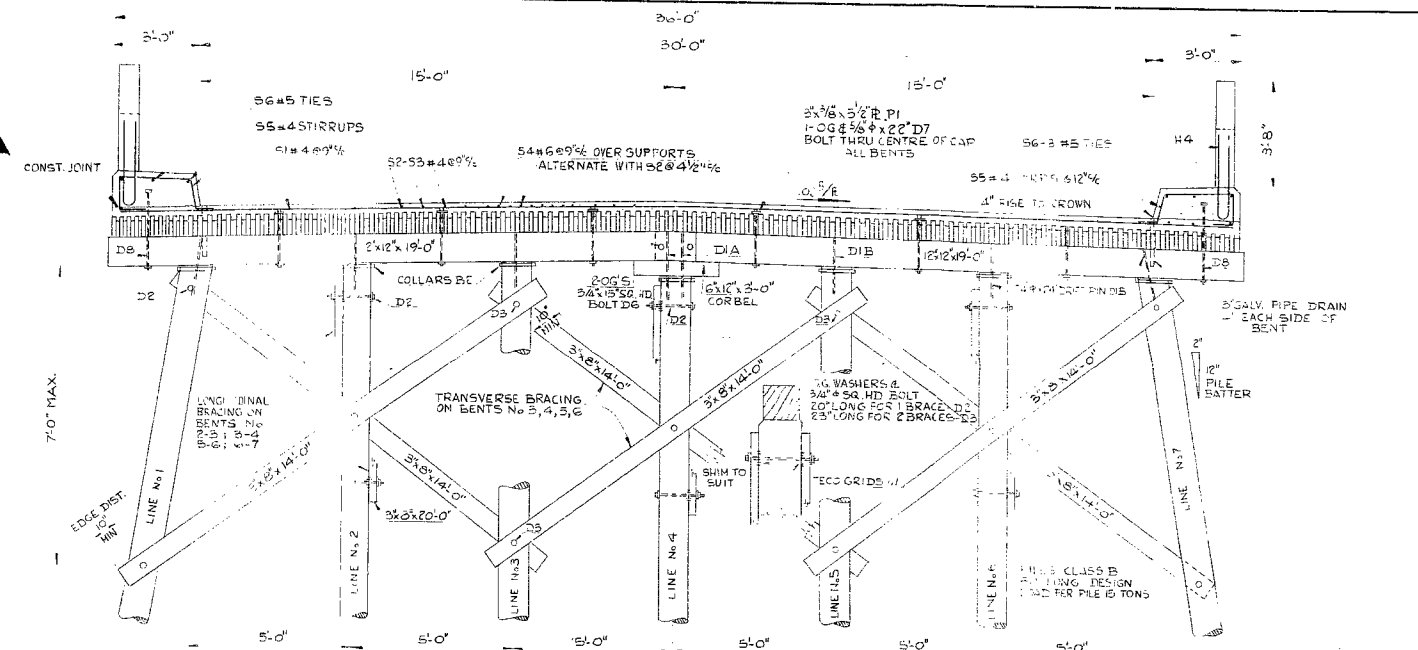
LIST OF DRAWINGS	
D-5167-1	PLAN ELEVATION & SECTION
2	DETAILS
4	BILL OF MATERIALS
5	RAILROAD PANELS & POSTS
6	SOIL DATA

**NOTES**  
TO DISTRICT ENGINEER  
CONCRETE WORK ON THIS STRUCTURE MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE DISTRICT ENGINEER TO CONTRACTOR  
STRUCTURE TO BE BUILT IN ACCORDANCE WITH FORM N-9.9 AND THE SPECIAL PROVISIONS, EXTRA COPIES OF WHICH MAY BE OBTAINED FROM THE DISTRICT ENGINEER.  
ALL TIMBER TO BE CREOSOTED IN ACCORDANCE WITH C.S.A. 080 SPECIFICATIONS WITH A RETENTION OF 3 (EIGHT) LBS. PER CU. FT.  
ALL BENTS & CAPS TO BE IN PLACE BEFORE DECKING IS COMMENCED  
ALL TIMBER TO BE JACK PINE OR EQUAL SUPPLIED IN ACCORDANCE WITH C.S.A. SPECIFICATIONS 086 TABLE 5

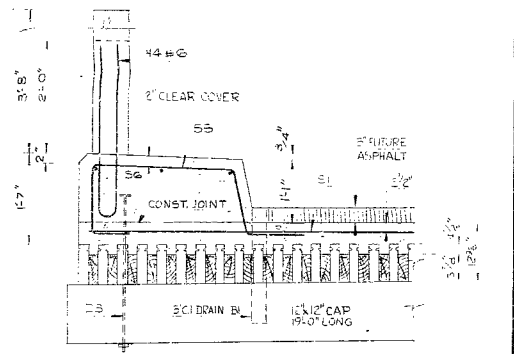
**CONCRETE MIX**  
MIN. STRENGTH OF CONCRETE @ 28 DAYS : 3000 P.S.I.  
MAX. SIZE OF AGGREGATE : 3/4"  
APPROVED ADMIXTURES SUPPLIED BY THE CONTRACTOR WILL BE ADDED TO ALL CONCRETE AS SPECIFIED BY THE ENGINEER.  
**BORING DATA**  
THE COMPLETE SOIL INVESTIGATION REPORT FOR THIS STRUCTURE MAY BE EXAMINED AT THE BRIDGE OFFICE AND FOUNDATION OFFICE, DOWNSVIEW AT ANY REGIONAL OFFICE AND AT THE SAULT STE MARIE DISTRICT OFFICE. THE DEPARTMENT DOES NOT GUARANTEE THE ACCURACY OF THIS REPORT OR THE ABRIDGED VERSION SHOWN ON THESE PLANS.  
**CLEAR COVER ON REINFORCING STEEL**  
AS SHOWN

**CONSTRUCTION NOTES**  
ALL EXPOSED EDGES TO BE CHAMFERED 1/4" EXCEPT AS NOTED.  
ALL CONSTRUCTION JOINTS MUST BE APPROVED BY THE BRIDGE ENGINEER.  
UTMOST CARE MUST BE TAKEN IN HANDLING CREOSOTED TIMBER TO AVOID DEFACEMENT OF SURFACES. NO CHAINS HOOKS OR PEAVES MAY BE USED IN HANDLING SAME.  
FIELD DRILLED HOLES (BOLT, DRIFT PIN, ETC.) MUST BE TREATED WITH HOT CREOSOTE OIL APPLIED WITH BOLT HOLE TREATER. CREOSOTE OIL TO BE SUPPLIED BY D.H.O.  
FRESH WOOD SURFACES EXPOSED BY FIELD CUTTING TO BE TREATED WITH 3 BRUSH COATS OF HOT CREOSOTE OIL.  
THE CONTRACTOR SHALL BE RESPONSIBLE FOR ENSURING THAT THE FINAL DECK ELEVATIONS CONFORM WITH THE ELEVATIONS SHOWN.

TABLE OF PILE HEADS ELEVATIONS									
LINE	BENT#1	BENT#2	BENT#3	BENT#4	BENT#5	BENT#6	BENT#7	BENT#8	
LINE#1	1467.93	1468.10	1468.31	1468.52	1468.72	1468.93	1469.14	1469.31	
LINE#2	1468.03	1468.20	1468.41	1468.62	1468.82	1469.03	1469.24	1469.41	
LINE#3	1468.13	1468.30	1468.51	1468.72	1468.92	1469.13	1469.34	1469.51	
LINE#4	1467.73	1467.90	1468.11	1468.32	1468.52	1468.73	1468.94	1469.11	
LINE#5	1468.13	1468.30	1468.51	1468.72	1468.92	1469.13	1469.34	1469.51	
LINE#6	1468.03	1468.20	1468.41	1468.62	1468.82	1469.03	1469.24	1469.41	
LINE#7	1467.93	1468.10	1468.31	1468.52	1468.72	1468.93	1469.14	1469.31	



SECTION THRU PILE BENT A-A  
N.T.S.



CURB DETAIL  
SCALE: 3/4"=1'-0"

PRINT RECORD		
No.	FOR	DATE
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		

REVISIONS		
DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
LITTLE PINE LAKE TIMBER BRIDGE 12 MILES WEST OF BIG PINE LAKE			
KING'S HIGHWAY No. 101 EPR. DIST. SUDBURY		DIST. No. 18	
TWP. PETERS		CON.	
PLAN, ELEVATION AND SECTION			
APPROVED	BRIDGE ENGINEER	SITE No. 53 N-213	W.P. No. 145-62
DESIGN	ADAPTED	CHECK	CONTRACT
DRAWING	G.P.	CHECK	No.
DATE	NOV 1962	LOADING	450-516
		DRAWING No.	D-5167-1



Mr. A. M. Toye,  
Bridge Engineer.  
Materials & Research Division,  
(Foundation Section)

July 11, 1962.

FOUNDATION INVESTIGATION REPORT  
By: W. A. Trow & Assoc., Ltd.

Attention: Mr. E. McConbie.

Re: Proposed Crossing, Hwy. No. 101,  
Little Pine Lake, District No. 18,  
W.F. 148-62.

Attached, we are forwarding to you, the above-mentioned report submitted by the Consultant, W. A. Trow & Associates. We have reviewed the report and have found the factual information well presented and the recommendations conclusive. We agree with the Consultant's suggestion for a pile loading test. The information that could be obtained from such a test would be very valuable and useful for future work. It is our opinion that a test should be also carried out after all the piles are driven because in this way, information on the degree of soil densification - i.e., increase of bearing capacity due to pile driving, could be obtained.

Should there be any additional questions or problems that you would like to discuss, please feel free to call on our Office.

AGS/HMF  
Attach.

*Afternoon*  
A. G. Sterner,  
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. A. M. Toye (2)  
H. A. Tregaskes  
H. D. McMillan  
H. McArthur  
D. P. Collins  
T. F. Saint  
T. J. Kovich  
J. Roy  
J. E. Grusnier  
F. Norman  
A. Watt  
Foundations Office  
Gen. Files. ✓

Project: J886

July 5, 1962

Mr. A. Rutka,  
Materials and Research Engineer,  
Department of Highways of Ontario,  
Parliament Buildings,  
Toronto, Ontario

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

Re:

Foundation Conditions  
Little Pine Lake Crossing  
Hwy. 101 - WP ~~187-60~~

148-62

Dear Sirs:

The enclosed report describes the results of a foundation investigation completed at this site early in June of this year.

Sand, grading from coarse to fine with depth, was encountered in all borings down at least to 88 feet below the water surface. Because the drilling equipment was relatively light, - a requirement dictated by the need to fly to the various bridge sites in this area, - it was not possible to reach or to confirm bedrock. However, it is concluded that friction piles, either of timber or cylindrical steel are the logical means of foundation support for the proposed bridge structure. Subject to the results of a confirming load test, the estimated safe capacity of a Class B timber pile driven to a depth of 30 feet below the river bed is 20 tons. The working capacity for a 12 inch cylindrical steel pile is 50 tons at a depth of 40 feet. Refusal to driving will not be encountered at these depths.

The water in this lake reaches a maximum depth of approximately 6 $\frac{1}{2}$  feet: this is believed to be a high water level condition. It is anticipated that relatively large waves will build up under the action of strong north winds. As a protection against erosion of the sides of the earth causeway, relatively flat slopes in the order of 5 to 1 should be used. Ample granular fill appears to be available on each shore for this purpose.

We shall be pleased to discuss any queries you may have after you have had an opportunity to review the enclosed information.

Yours very truly,

*W. Trow*

William A. Trow, P.Eng.

WAT/gc  
Encls.

DEPARTMENT OF HIGHWAYS OF ONTARIO  
MATERIALS AND RESEARCH DEPARTMENT  
PARLIAMENT BUILDINGS, TORONTO.

FOUNDATION INVESTIGATION  
LITTLE PINE LAKE CROSSING  
HWY. 101 - W.P. 187 -60

Project: J886

July 1962

William A. Trow & Associates Ltd.



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Recommendations	5

ENCLOSURES

Photographs of Site	
Estimated Safe Capacities of Friction Pile	Table 1
Borehole Location Plan and Estimated Stratigraphy	Dwg. 1
Borehole Logs, Holes 1 to 6	2 - 7
Results of Static Penetration Tests	8
Grain Size Distribution Curves	9

FOUNDATION INVESTIGATION  
LITTLE PINE LAKE CROSSING  
HWY. 101 - WP 187-60

Site and Project

This proposed crossing of the Little Pine Lake is situated about 5 miles to the west of the present construction limit of Hwy. 101 and about 21 air miles west of Chapleau. A recent revision in the route of Hwy. 101 has resulted in the movement of the crossing point approximately 400 feet north of the original line. The survey cutting through the bush for this old line is quite distinct and it served as a reference for locating this revised crossing.

The lake is 448 feet wide at this crossing location. A heavy growth of jack pine extends down to the shoreline on each side of the water. The shores of the lake rise gradually to a height of 5 to 6 feet above the water on the east shore and 3 to 4 feet in height on the west bank. Sand and gravel are exposed on the shore but no larger stones were observed. Some large boulders were noted on the east bank about 800 feet north of the crossing and bedrock outcrops from the west bank about 1200 feet to the south.

The depth of water at this crossing location ranges in the order of 5 to 6 feet. The bed of the lake contains very little organic material and the poles barely penetrated into the soil as the raft was pushed between borehole locations. It is understood that the water was at its highest level at the time of the investigation.

It is assumed that the Highway crossing of this water will take the form of a central bridge connected to the shoreline by earth fill causeways. The bridge presumably will be located adjacent to the deepest water in order to allow for future navigation.

Field Work

A total of 6 borings were made at representative locations across this stretch of water. Four of them were located near the centre of the river where the water was deepest. The locations of the holes are shown on Dwg. 1. All work was performed from a raft which was made from empty drums and from adjacent trees.

Samples were taken generally at 5 foot intervals and, since the soil consisted entirely of fine to coarse sand, they were recovered in the disturbed state using the conventional 2 inch O.D. split spoon. In many instances the sample was lost on withdrawal of the spoon from the BX casing and it was necessary to recover the soil using a side sampler.

The borings ranged in depth from 60 to 70 feet. This was the maximum depth that the casing could be driven and still be recovered after sampling was complete. The friction on the casing was sufficiently large that it was very difficult to recover it, even when driven to these depths. Attempts were made to probe deeper by driving a cone from the bottom of the casing. The maximum penetration depth with this scheme was about 90 feet. A cone penetration test was also performed adjacent to each hole before the boring was made. A static penetration test was carried out in hole 4 using an A drill rod.

The levels of all holes coincided with the water surface. The level of the water was referred to a nail in a tree at the location shown on Dwg. 1.

#### Subsoil and Discussion of Friction

The results of the six borings for this project are recorded on Dwgs. 2 to 7 of this report and on the estimated stratigraphical profile of Dwg. 1. It is seen that the subsoil is remarkably uniform in lateral directions; it consists of sand which grades from coarse to fine with depth. A very thin veneer of mud covers the lake bottom. Refusal was encountered in hole 1, only, where the cone bounced at a depth of 88.5 feet below the water surface. Typical grading curves for the sand are indicated in Dwg. 9.

According to the penetration resistance or N values determined in the sampling program, the sand, below the lake bed, has a relative density ranging from loose to medium dense only. A similar situation was encountered for other bridge site investigations in this general area. Despite this apparent loose condition, however, considerable friction resistance was experienced, both in the driving and withdrawal of casing and cone rods. It was felt, therefore, that the soil is somewhat more competent than is inferred from these empirical indicators.

In order to obtain an approximate indication of the friction resistance generated in the soil, a test was devised which involved the measurement of the force required to push an A drill rod into the soil below casing level. This test had been performed previously for the Giles Creek crossing about 3 miles to the west.

The results of this testing, which was carried out between 18 and 49 feet in hole 4, are presented on Dwg. 8. The graphs, shown in this drawing, illustrate the measured increase in resistance to the A rod with increasing penetration below the bottom of the casing. The slope of this line or increase in resistance with depth is due almost entirely to the friction generated on the side of the A rods. The additional increase in bearing at the tip of the rods is negligible.

An allowance has been made for the increase in end bearing capacity in the computation of the friction coefficient K in Dwg. 8. In all cases it is seen that the value of K is equal to or greater than 2.0. The coefficient K is determined from the expression:

$$K = N \tan \phi R$$

where:        N        is the horizontal earth pressure coefficient acting on the shaft

$\phi$         is the angle of internal friction of the soil

and            R        is a roughness coefficient equal to or less than unity depending on the roughness of the rod

It follows, therefore, that, for  $K = 2$ , the horizontal pressure coefficient, N, on the rod must be greater than 2 unless, of course, the angle of internal friction of the soil is equal to 45 degrees. A value of  $N = 2$  or more is well in excess of the earth pressure at rest condition. A similar result was obtained for static tests on an A rod at the Giles Creek Crossing about 3 miles to the west.

An analysis of a pull-out test on an H pile in the dense sand underlying Ashbridge's Bay in Toronto has indicated a generated value of K equal to 0.2. This is approximately equivalent to an earth pressure at rest condition around the pile. Similarly, published information on pull-out tests on step-tapered Raymond piles in coral sand has shown values of K well in excess of 2 and a horizontal earth pressure coefficient, N, equal to the passive earth pressure state.\* It is concluded from these records, together with the results of the tests on the 1 - 5/8 inch diameter A rod, that the magnitude of friction generated around a pile shaft is dependent upon the degree of displacement of the surrounding soil. With a timber pile or a cylindrical steel pile this displacement will be sufficient to develop the full passive resistance of the soil. It is reasonable to conclude, therefore, that a value of K equal to or in excess of 2 should be obtained. Consequently, any estimates of cylindrical pile capacity, based on an assumed value of  $K = 1$ , should be quite conservative.

### Foundations

In view of the difficulties associated with the installation of footings below the lake bed in loose sand, a pile foundation would seem to be the only reasonable method of support for this bridge. The use of end-bearing piles is feasible, although the refusal level has not been established within the first 88 feet of depth below the water surface. The only reasonable alternative, therefore, is cylindrical friction piles.

\* "Pulling Tests on Piles in Sand" - H.O. Ireland - 4th Int. Conf. on Soil Mechanics and Foundation Engineering - 1957

In the foregoing section, arguments have been presented in support of a proposal for friction piles in this "loose" sand. It has been concluded that a high friction resistance will be developed and that the use of a friction factor  $K = 1$  will result in conservative design. This is the reasoning presented for other bridge sites in this area.

The estimated safe capacities for timber piles and cylindrical steel piles, driven into this sand deposit, have been indicated in Table 1. The assumptions used in the preparation of these estimates also are shown.

On the basis of these tabulated values, it is recommended that Class B timber piles should be designed for a load of 20 tons, which capacity should be attainable at a depth of 30 feet below the river bed. A design load of 50 tons should be used for 12 inch O.D. steel piles and this capacity should be developed after a penetration of 40 feet into the sand. It is recommended that these estimates be confirmed by a least one load test.

The settlement associated with the application of the foundation loading should be well within tolerable limits and it will occur as soon as load is applied. The settlement under the weight of the causeway fill also will be immediate and of small magnitude. Since the causeway fill must be placed first, it will not cause any movement of the bridge structure.

The subsoil being entirely granular in nature, there will be no embankment stability problem. In addition, since the river bed is clean, fill can be placed directly on it without concern about entrapped organic material.

The causeway approaches to this bridge pass across a long narrow body of water having an unrestricted reach of 2500 feet to the north and a lesser reach of about 900 feet to the south. According to recent studies by the Corps of Engineers, a wave 1 foot high can be developed under a steady 30 m.p.h. wind over a reach of about 2500 feet.\* If the wind maintains a velocity of 60 m.p.h. for 10 minutes, the wave height increases to 2 feet. The recommended rip rap protection against these waves for a 3:1 embankment slope is as follows\*\*:

Maximum size of stone	-	1000 lbs.
25% greater than	-	300 lbs.
45-75% ranging from	-	10 - 300 lbs.
25% less than		10 lbs.
nominal thickness	=	18 inches.

Since large stones probably are in short supply in the immediate area, the alternative protection is to use flatter slopes. An examination of beaches by the Corps of Engineers has indicated that coarse

\* - Slope Protection along Reservoirs: H.L. Drake, U.S. Corps of Engineers  
A.S.C.E. Apr. 11/61

\*\* - Design of Small Dams: U.S. Bureau of Reclamation, Table 17, P.207

gravel surfaces remain stable on slopes of approximately 5:1. The corresponding equilibrium slope for medium to coarse sand is 10:1. Since the water is quite shallow at this crossing, and since ample finer granular material should be available, the use of flatter slopes may provide economic protection for possible heavy wave action. The natural slope in the first 25 feet from each shore at this site is approximately equal to 6:1.

#### Recommendations

Sand, grading from coarse to fine and extending at least to a depth of 88 feet, underlies the river bottom at this site. Although field penetration tests indicate this material to be loose, it is believed to be sufficiently competent to support large displacement friction piles. It is recommended, therefore, that the structure be supported on Class B timber piles, or on cylindrical steel piles. The estimated safe loads for these mediums of support are 20 tons at 30 feet, and 50 tons at 40 feet, respectively. These estimates should be confirmed by one load test. There is an ample supply of timber adjacent to this crossing.

The embankment approaches to the bridge will be exposed to considerable wave action. The use of flat slopes in the order to 5:1 are suggested as protection against erosion.

WAT/lt  
July 1962.



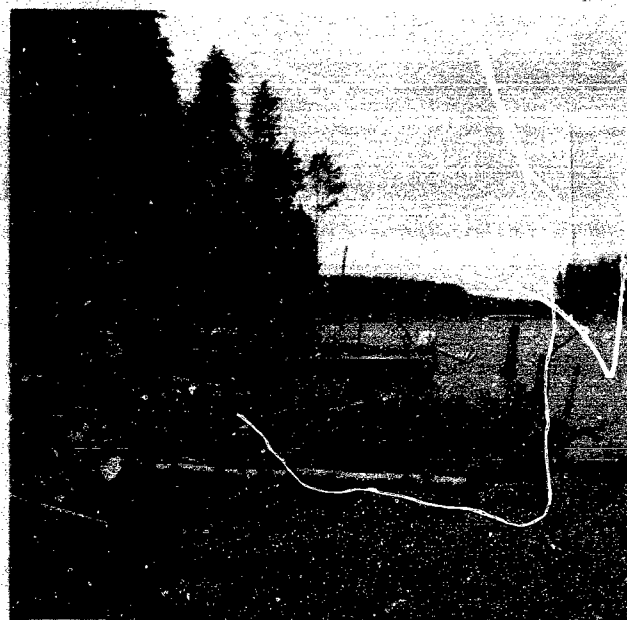
*W. Trow*  
William A. Trow (P. Eng.)



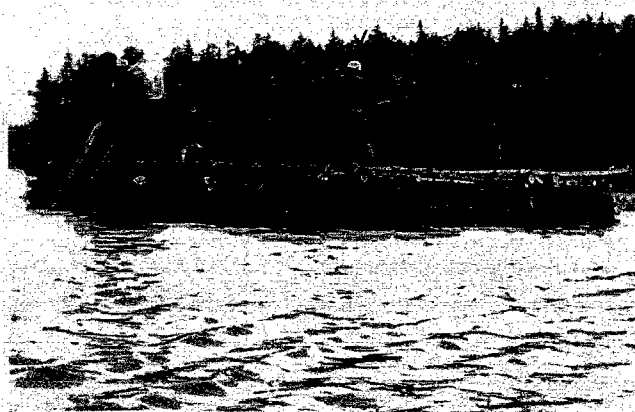
View of Raft



View of West Bank  
From Raft



View of West Bank  
From The South



View of Raft



View of West Bank  
From Raft



View of West Bank  
From The South





From The West  
Bank - Along CL



View From The East Bank  
Looking Along Centre Line



Looking North  
From The Centre Line



From The West  
Bank - Along CL



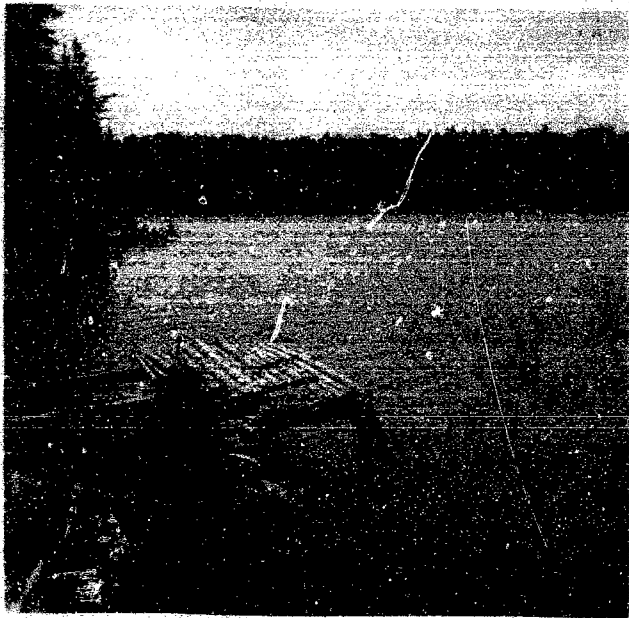
View From The East Bank  
Looking Along Centre Line



Looking North  
From The Centre Line



East Bank  
And Raft Looking South



East Bank  
From The North



East Bank  
From The South



East Bank  
And Raft Looking South



East Bank  
From The North



East Bank  
From The South

TABLE NO. 1

SUMMARY OF ESTIMATED PILE CAPACITIES IN TONS<sup>+</sup> - Little Pine Lake Crossing Hwy. 101

Depth ** Below Stream Bed	Timber Pile*	Cylindrical Steel Pile <sup>++</sup>
30	23.5	33.5
40	39.5	54.5
50	60	80.5
60		111

\*\* - Assume extra capacity due to weight of fill offset by possibility of scour to greater depths.

\* - Tip diameter = 8 ins.; average diameter = 10 ins.

++ - O.D. = 12 inches.

+ - Estimated from expression:

$$Q = \frac{1}{F} ( A (0.3\gamma D N_{\gamma} + \gamma Z N_q) + \frac{1}{2} \gamma Z^2 P K )$$

where: A = area of pile tip in sq.ft.

$\gamma$  = 63 p.c.f., is the submerged weight of the soil.

D = tip diameter of pile in feet

Z = depth of pile below stream bed level

$N_{\gamma}$  &  $N_q$  = are bearing capacity factors - assume = 30

P = average perimeter of pile in feet

K = is the friction coefficient - assume = 1

F = 2, is the factor of safety.

Note: Recommended loadings - subject to load test

Timber pile = 20 tons at Z = 30 feet

Steel pile = 50 tons at Z = 40 feet.

The factor of safety in these recommendations is somewhat greater than 2.

BOREHOLE No. 1  
 PROJECT Hwy. 101 Little Pine Lake, WP 187 - 60  
 LOCATION See Dwg. 1.  
 HOLE LOCATION See Dwg. 1.  
 HOLE ELEVATION 101.3 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 (S) ⊕<sup>+</sup>

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X<sup>LI</sup>

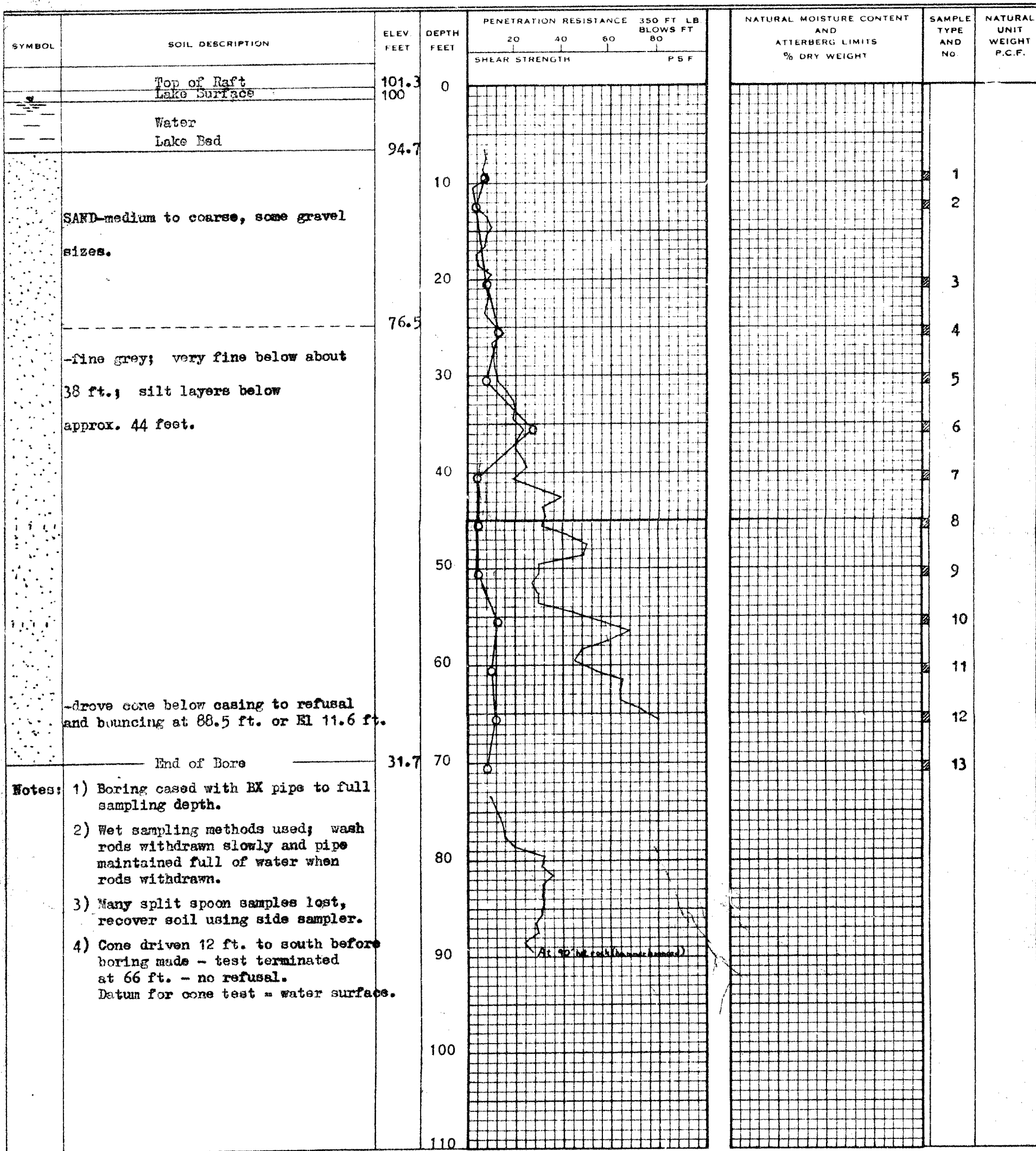
## ATTERBERG LIMITS

LIQUID LIMIT —○—

PLASTIC LIMIT ———

## SAMPLE TYPE

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



## LEGEND

BOREHOLE NO. 2  
PROJECT Hwy. 101 - Little Pine Lake, WP 187 - 60  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.3 ft.  
DATUM See Dwg. 1.

## 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  $(S_u) +^s$

## NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

LI  
X

## ATTERBERG LIMITS

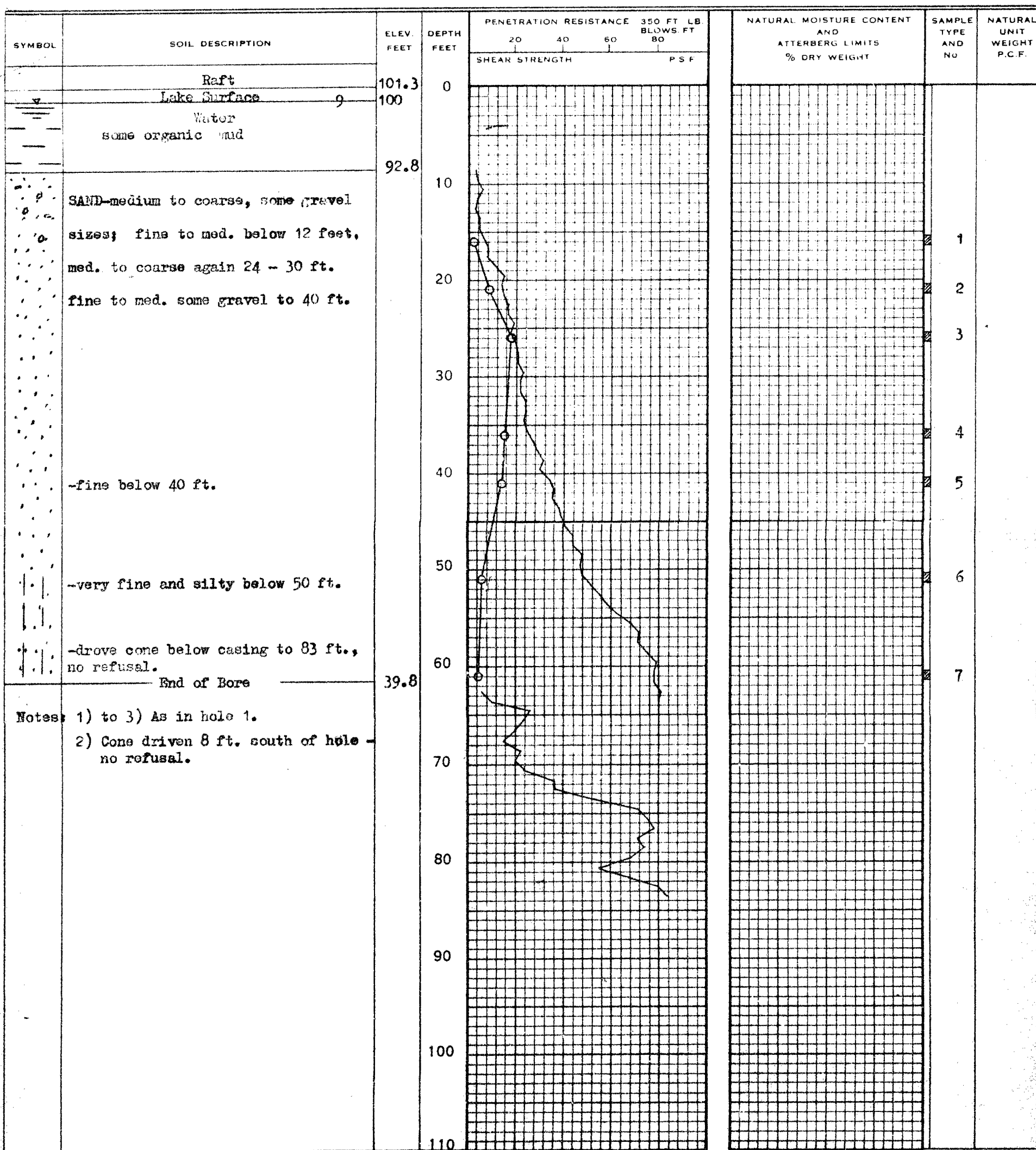
LIQUID LIMIT —○—

PLASTIC LIMIT ———

## SAMPLE TYPE

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

□  
■  
■





## 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 ⊗  
VA E TEST AND SENSITIVITY (S<sub>v</sub>) ⊕<sup>s</sup>

## NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

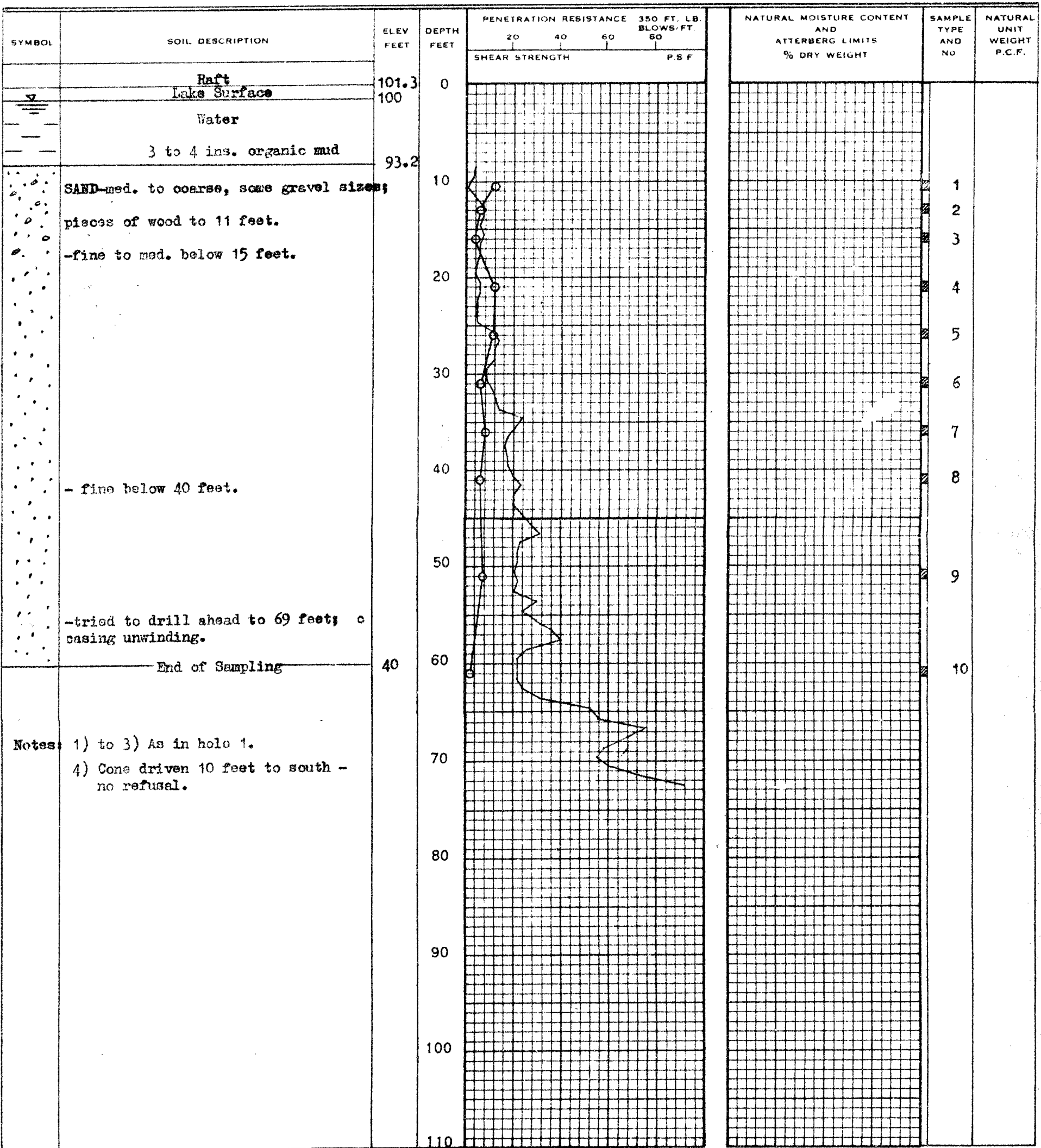
X<sup>LI</sup>

## 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. 3  
PROJECT Hwy. 101 - Little Pine Lake, WP 187 - 60  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.3 ft.  
DATUM See Dwg. 1.



## LEGEND

BOREHOLE NO. 4  
PROJECT Hwy. 101 - Little Pine Lake, WP 187-60  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.3 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 (S) +<sup>s</sup>

## NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

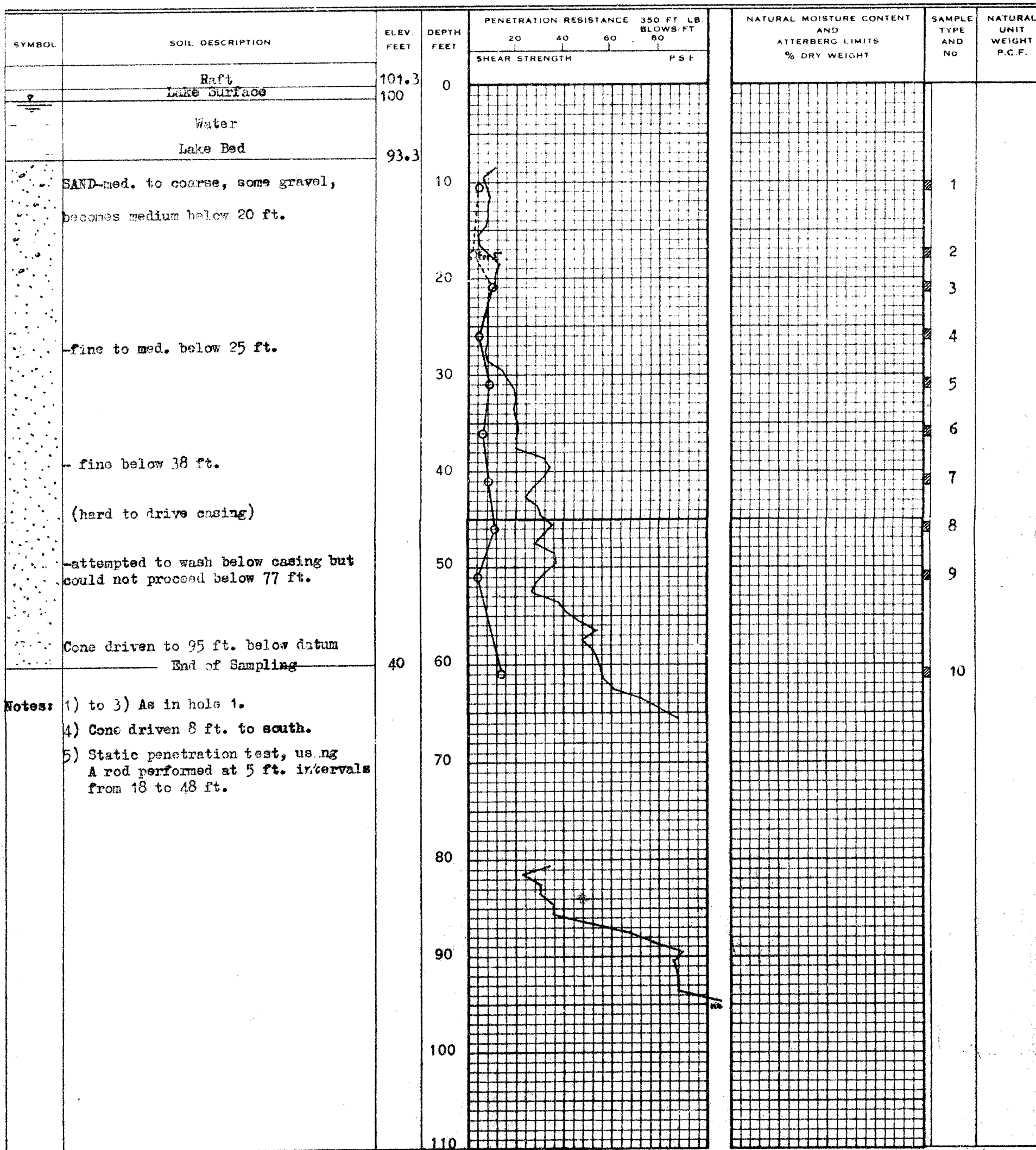
## ATTERBERG LIMITS

LIQUID LIMIT —○—

PLASTIC LIMIT ———

## SAMPLE TYPE

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

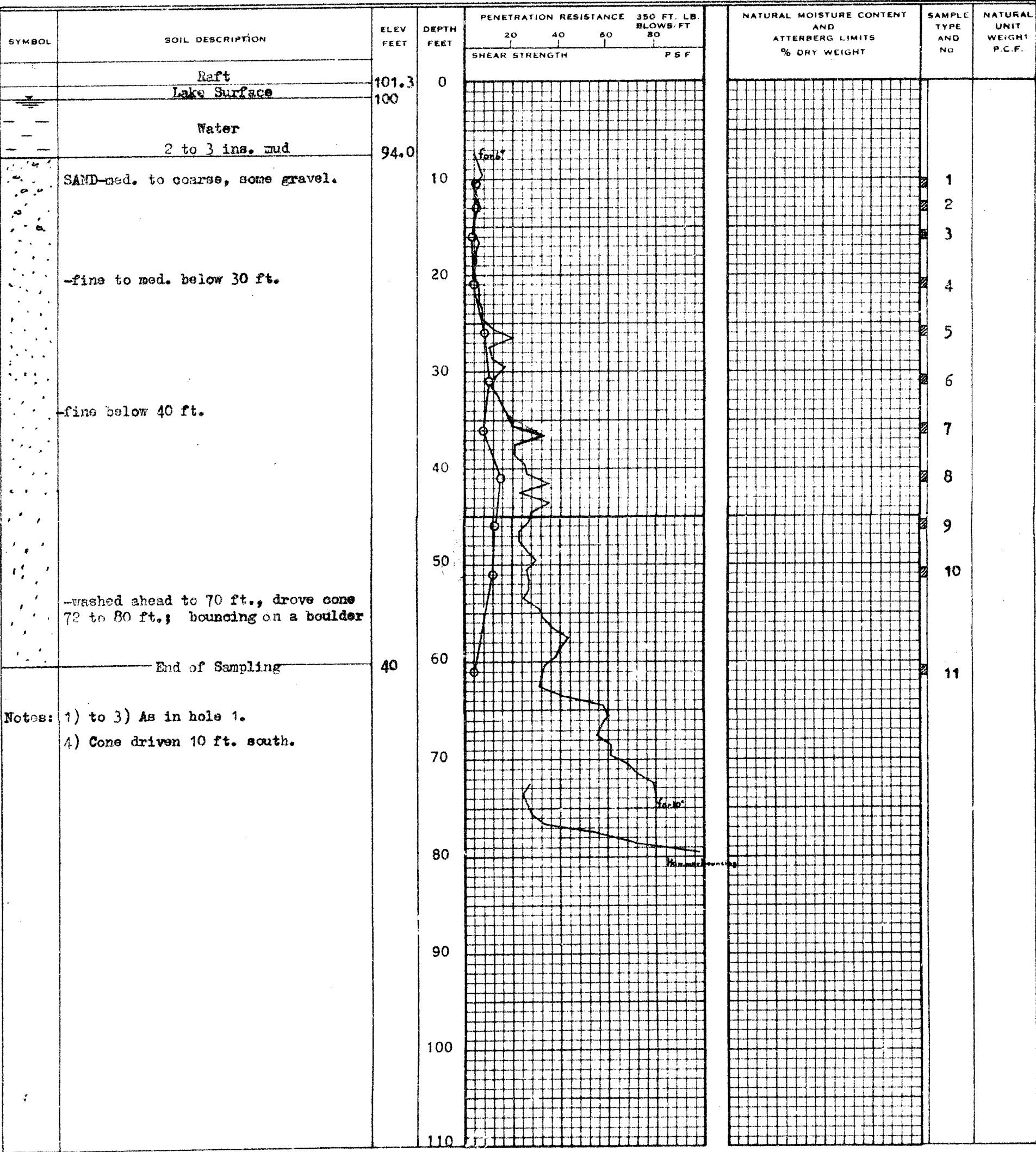


LEGEND

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 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. 5  
PROJECT Hwy. 101 - Little Pine Lake, WP 187 - 60  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.3 ft.  
DATUM See Dwg. 1.



BOREHOLE No. 6  
 PROJECT Hwy. 101 - Little Pine Lake, WP 187 - 60  
 LOCATION See Dwg. 1.  
 HOLE LOCATION See Dwg. 1.  
 HOLE ELEVATION 101.3 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 (S) ⊕<sup>s</sup>

## NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

X<sup>LI</sup>

## ATTERBERG LIMITS

LIQUID LIMIT —○—

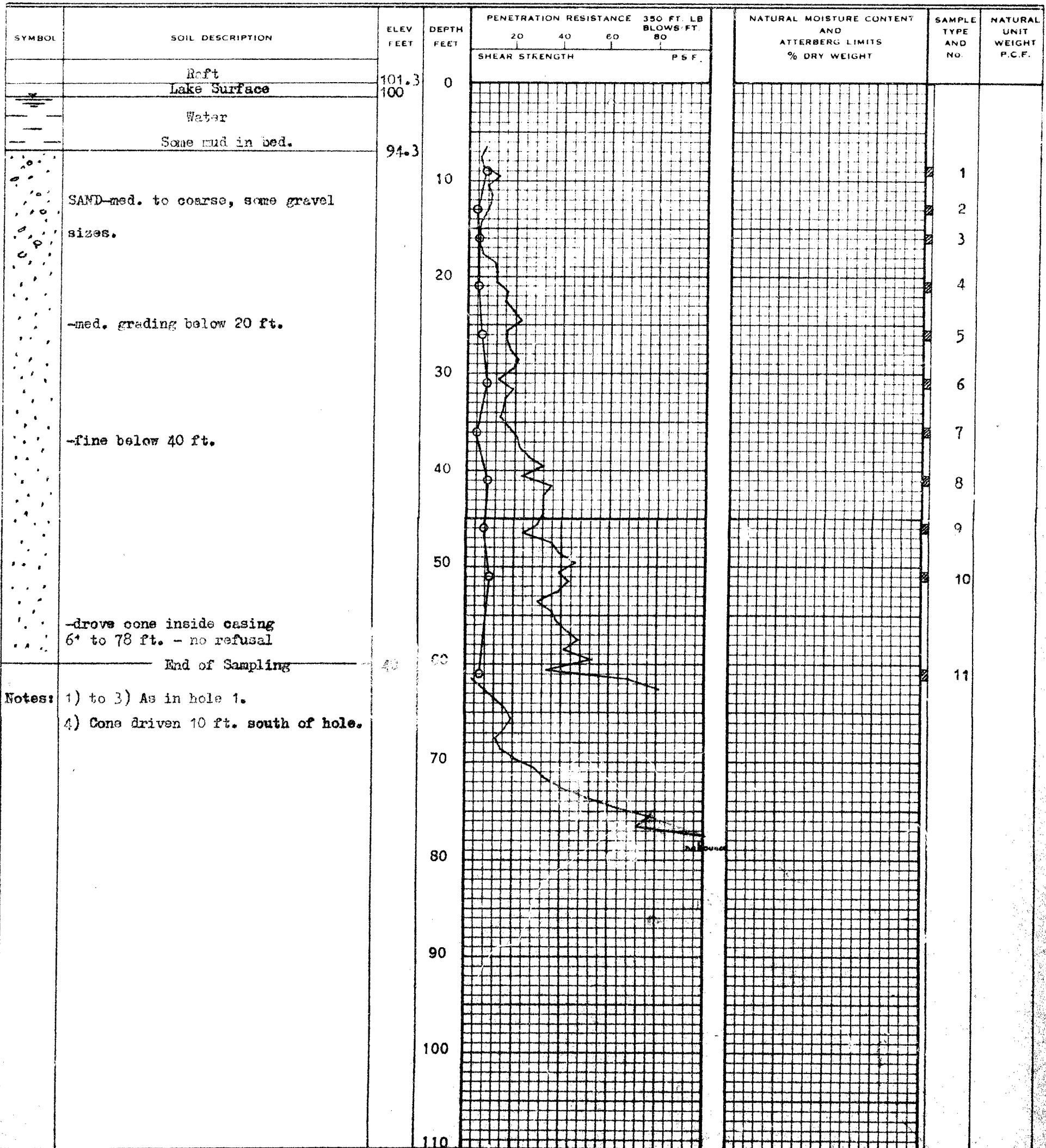
PLASTIC LIMIT ———

## SAMPLE TYPE

2" O.D. SPLIT TUBE —■—

2" I.D. SHELBY TUBE —■—

3" O.D. SHELBY TUBE —■—



Depth Below Casing Inches

Increase = 165 lbs./in.  
 = 4660 psf  
 YZ = 715 psf

Increase = 117 lbs./in.  
 = 3300 psf  
 YZ = 1040 psf

Test at 18½ ft.  
 below datum.

Test at 23½ ft.  
 below datum

Increase = 155 lbs./in.  
 = 4370 psf  
 YZ = 1365 psf

Increase = 275 lbs./in.  
 = 7760 psf  
 YZ = 1690 psf

Test at 28½ ft.

Test at 33½ ft.

Increase = 500 lbs./in.  
 = 14,100 psf

Test at 38½ ft.

YZ = 2015 psf

Increase = 195 lbs./in.  
 = 5500 psf  
 YZ = 2340 psf

Test at 43½ ft.

Ultimate Capacity at any Depth Z  
 $Q = A(0.30DN_Y + YZN_q) + YZlpK$

Lake bed  
 surface

Increase = 205 lbs./in.  
 = 5800 psf  
 YZ = 2665 psf

A rod  
 D = .1354"  
 A = 0.0144  
 sq.ft.

P = .425'  
 Bottom of  
 EX casing

Lake bed = 7½ ft. below datum  
 Submerged weight of soil  
 YZ = 65 psf  
 Take  $N_Y = N_Y = N$   
 For  $\phi \sim 35^\circ$  to  $40^\circ$   
 $\beta \sim 40^\circ$  to  $100$

K = friction coefficient

Values of K

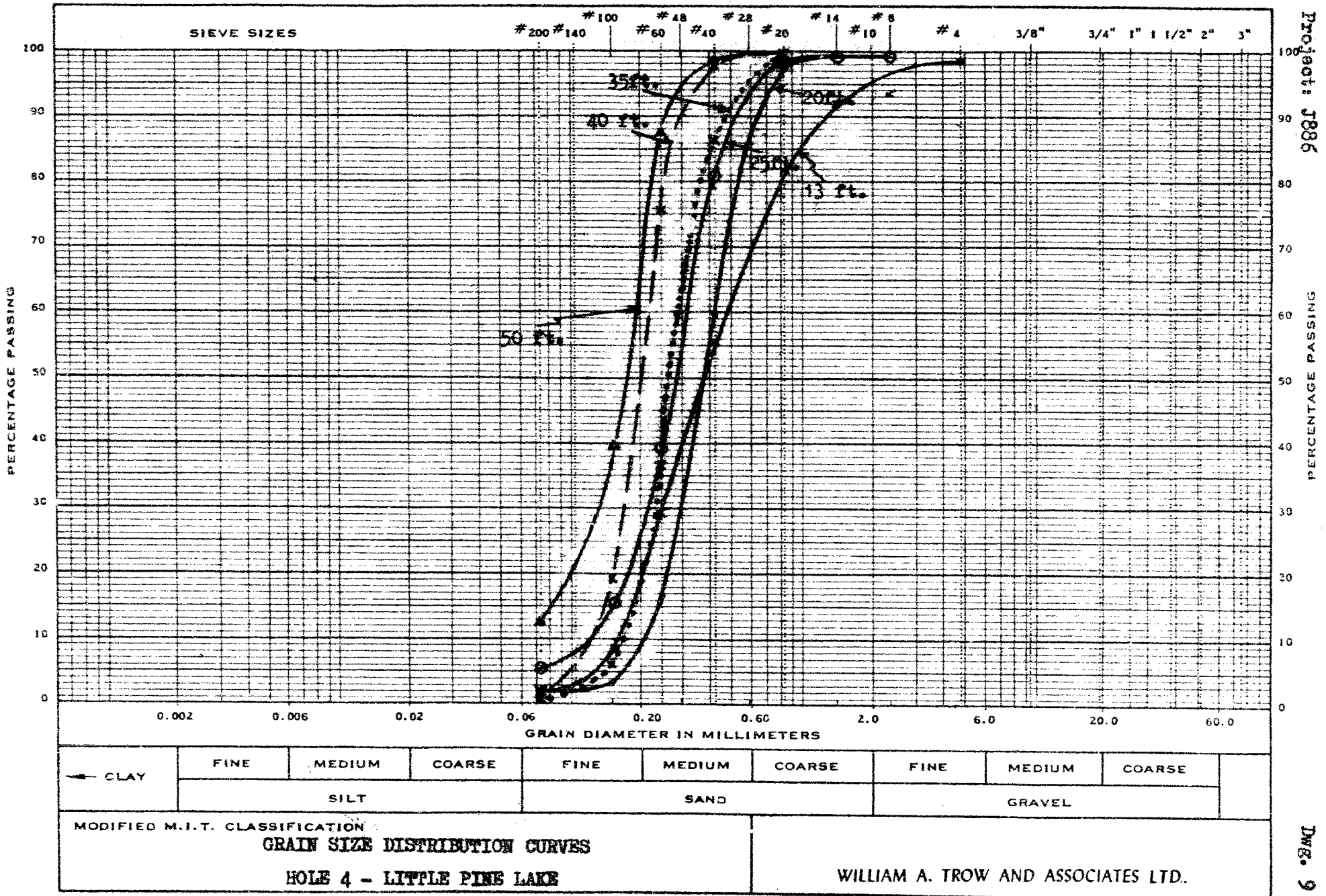
(Z+7½)	N=40	100
18½	6.4	6.2
23½	3.09	2.95
28½	3.1	3.0
33½	4.53	4.45
38½	6.94	6.87
43½	2.31	2.25
48½	2.14	2.08

Test at 48½ ft.

RESULTS OF STATIC CONE TESTS  
 ON A DRILL ROD, HOLE 4 - LITTLE PINE LAKE

WILLIAM A. TROW & ASSOCS. LTD.

# MECHANICAL ANALYSIS





Materials and Research Division

June 8, 1962.

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

Attention: Mr. W. A. Trow.

<sup>148-62</sup>  
Re: W.F. 187-60, Hwy. 101, Little Pine Lake,  
District #18, Sault Ste. Marie.

Dear Sir:-

Please consider this your authority to carry out a foundation investigation at the above site.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Fourteen copies of the completed foundation report should be submitted to the Foundation Section as soon as possible. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Charges for the work performed will be in accordance with your schedule of Rates, dated May 24, 1959, and invoice to be addressed to the attention of the undersigned.

Note:- As North Bay is the nearest recognized mobilization point, payment for mobilization will be from there, as discussed with your representative.

NLS/ndef

Yours very truly,

cc: Messrs. J. McCombie  
H. Mearthur  
D. P. Collins  
E. R. Saint  
N. D. Smith (2)

Mrs. T. Tate  
Foundations Office ✓  
Gen. Files (2)

*G. Luck*  
A. Rutka,  
MATERIALS AND RESEARCH BRCH.

Mr. A. H. Teye,

Bridge Engineer.

Materials & Research Division,  
(Foundation Section)

July 11, 1962.

FOUNDATION INVESTIGATION REPORT

By: W. A. Trow & Assoc., Ltd.

Attention: Mr. J. McLaughlin.

Re: Proposed Crossing, Hwy. No. 101,  
Little Pine Lake, District No. 18,  
S.F. 140-62.

Attached, we are forwarding to you, the above-mentioned report submitted by the Consultant, W. A. Trow & Associates. We have reviewed the report and have found the factual information well presented and the recommendations conclusive. We agree with the Consultant's suggestion for a pile loading test. The information that could be obtained from such a test would be very valuable and useful for future work. It is our opinion that a test should be also carried out after all the piles are driven because in this way, information on the degree of soil densification - i.e., increase of bearing capacity due to pile driving, could be obtained.

Should there be any additional questions or problems that you would like to discuss, please feel free to call on our office.

G. /ndw  
attach.

*Afternoon*  
A. G. Sternes,  
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. A. H. Teye (2)  
B. A. Tregaskes  
B. E. McMillan  
B. McArthur  
E. R. Collins  
V. F. Saint  
T. J. Ravich  
J. Roy  
J. E. Gruspler  
F. Herman  
A. Watt  
Foundations Office  
Gen. Files.

## MEMORANDUM

*File with report  
also*

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

FROM: J.C. McAllister

Bridge Division,  
DATE: November 28, 1962.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. #148-62,  
Little Pine Lake Bridge,  
Hwy #101, District #18.

Attached please find one print of plan D-5167-1 for the above crossing. The structure has been designed in accordance with the recommendations contained in the Foundation Report submitted by Mr. W.A. Trow & Associates.

The pile test recommended should not be necessary as the design load in this structure is about 11 tons/pile.

*J.C. McAllister*

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

JCMcA/dm



Bridge Division,  
May 30, 1962

W. Trow and Associates,  
1850 Jane Street,  
WILSON, Ontario.

Re: Foundation Investigation  
Hawk Jet, to Chapleau

Attention: Mr. W. Trow

Dear Sir:

The following structures have been removed  
from your drilling program as outlined by Mr. Starnac.

1. W.P. 79-62, Algoma Central Railway
2. W.P. 82-62 Kinniwabi River
3. W.P. 83-62 Kinniwabi River
4. W.P. 84-62 Kinniwabi River
5. W.P. 86-62 Jack Pine River

It has been decided to defer construction of a  
grade separation at the Algoma Central Railway  
indefinitely.

The three crossings of the Kinniwabi River were  
considered to be too inaccessible for foundation inves-  
tigation to be carried out at this time. Your aerial  
inspection of the Jack Pine River indicated that this  
crossing was solid rock and therefore the foundation  
investigation is not required.

Yours very truly,

S/S/C/m  
c.c. A. Starnac  
J.C. McAllister

B. McCosbie, P. Eng.  
Bridge Planning Engineer

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

SB-08-62

ACTION SLIP

61-6236

DATE Nov 30, 1962

TO Henry J. Symantec

FROM A. G. STERMAC

- |  |   |
|--|---|
| <input type="checkbox"/> NOTE AND FILE             | <input type="checkbox"/> PREPARE REPLY FOR MY SIGNATURE |
| <input type="checkbox"/> NOTE AND RETURN TO ME     | <input type="checkbox"/> TAKE APPROPRIATE ACTION        |
| <input type="checkbox"/> RETURN WITH MORE DETAILS  | <input type="checkbox"/> PER YOUR REQUEST               |
| <input type="checkbox"/> NOTE AND SEE ME           | <input type="checkbox"/> FOR YOUR SIGNATURE             |
| <input type="checkbox"/> PLEASE ANSWER             | <input type="checkbox"/> FOR YOUR INFORMATION           |
| <input type="checkbox"/> FOR YOUR APPROVAL         | <input type="checkbox"/> INVESTIGATE AND REPORT         |
| <input type="checkbox"/> RETURN WITH YOUR COMMENTS | <input type="checkbox"/> _____                          |

COMMENTS \_\_\_\_\_

Please file this in the report  
by Trans W. R. 198-62

MEMORANDUM

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

FROM: J.C. McAllister

Bridge Division,  
DATE: November 28, 1962.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. #86-62,  
Jackpine River Bridge,  
Hwy #101, District #18.

Attached please find one print of plan D-5175-1 for  
the above structure for your information.

A foundation investigation was not carried out here, but  
exposed bedrock was reported at the crossing by Mr. W.A. Trow.

*J.C. McAllister*

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

JCMcA/dm

Mr. A. E. Argue,  
Director,  
Design Services Branch,  
Admin. Bldg.

Foundation Section,  
Design Services Branch,  
Room 107, Lab. Bldg.

June 30, 1971

Comments on Claim - Contract 65-33

In 1962 a number of foundation investigations along the proposed line of Hwy. 101 were carried out for the Department by the Consultant, W. A. Trow and Associates Ltd. Some of the sites were hardly accessible, while others were not accessible at all, unless highly expensive mobilization measures were undertaken.

The Consulting Engineer flew over the proposed crossing on the Jack Pine River and observed bedrock to be exposed on both banks. He therefore suggested that an investigation at that time be dispensed with, and the verification of his observations be carried out at a later date when the site would be reasonably accessible. There is no written record of this recommendation, but it seems reasonable to assume that such a statement was made. A subsequent request for a foundation investigation to verify the Consultant's aerial observation was never received and, consequently, at this particular crossing, an investigation was never carried out.

Based on the Consultant's aerial inspection, the bridge for the Jack Pine River crossing was designed on spread footings bearing on rock. The drawings were reviewed and approved, all decisions being based on the aerial inspection report.

As discovered during construction, there was bedrock at the east abutment location, although at greater depth than shown on the drawings, while at the west abutment location, bedrock was not encountered even after the footing excavation was deepened.

It is quite possible that the Consultant, while flying over Jack Pine Creek, did not precisely identify the crossing location, and reported on the general conditions on the river banks - the bedrock surface can be rather erratic in this part of the country.

AGS/MdeF

A. G. Stermac  
Principal Foundation Engineer

23-63-136  
62-F-216e

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: J886

July 5, 1962

Mr. A. Rutka,  
Materials and Research Engineer,  
Department of Highways of Ontario,  
Parliament Buildings,  
Toronto, Ontario

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

Re: Foundation Conditions  
Little Pine Lake Crossing  
Hwy. 101 - WP ~~148-62~~  
148-62

Dear Sirs:

The enclosed report describes the results of a foundation investigation completed at this site early in June of this year.

Sand, grading from coarse to fine with depth, was encountered in all borings down at least to 88 feet below the water surface. Because the drilling equipment was relatively light, - a requirement dictated by the need to fly to the various bridge sites in this area, - it was not possible to reach or to confirm bedrock. However, it is concluded that friction piles, either of timber or cylindrical steel are the logical means of foundation support for the proposed bridge structure. Subject to the results of a confirming load test, the estimated safe capacity of a Class B timber pile driven to a depth of 30 feet below the river bed is 20 tons. The working capacity for a 12 inch cylindrical steel pile is 50 tons at a depth of 40 feet. Refusal to driving will not be encountered at these depths.

The water in this lake reaches a maximum depth of approximately 6½ feet; this is believed to be a high water level condition. It is anticipated that relatively large waves will build up under the action of strong north winds. As a protection against erosion of the sides of the earth causeway, relatively flat slopes in the order of 5 to 1 should be used. Ample granular fill appears to be available on each shore for this purpose.

We shall be pleased to discuss any queries you may have after you have had an opportunity to review the enclosed information.

Yours very truly,

*W. Trow*

William A. Trow, P.Eng.

WAT/gc  
Encls.

WILLIAM A. TROW AND ASSOCIATES LTD.

DEPARTMENT OF HIGHWAYS OF ONTARIO  
MATERIALS AND RESEARCH DEPARTMENT  
PARLIAMENT BUILDINGS, TORONTO.

FOUNDATION INVESTIGATION  
LITTLE PINE LAKE CROSSING  
HWY. 101 - W.P. ~~187-60~~  
148-60

Project: J886

July 1962

William A. Trow & Associates Ltd.

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Borehole Location Plan and Estimated Stratigraphy	Dwg. 1
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FOUNDATION INVESTIGATION  
LITTLE PINE LAKE CROSSING  
HWY. 101 - WP 187-60

Site and Project

This proposed crossing of the Little Pine Lake is situated about 5 miles to the west of the present construction limit of Hwy. 101 and about 21 air miles west of Chapleau. A recent revision in the route of Hwy. 101 has resulted in the movement of the crossing point approximately 400 feet north of the original line. The survey cutting through the bush for this old line is quite distinct and it served as a reference for locating this revised crossing.

The lake is 448 feet wide at this crossing location. A heavy growth of ~~jack~~ pine extends down to the shoreline on each side of the water. The shores of the lake rise gradually to a height of 5 to 6 feet above the water on the east shore and 3 to 4 feet in height on the west bank. Sand and gravel are exposed on the shore but no larger stones were observed. Some large boulders were noted on the east bank about 800 feet north of the crossing and bedrock outcrops from the west bank about 1200 feet to the south.

The depth of water at this crossing location ranges in the order of 5 to 6 feet. The bed of the lake contains very little organic material and the poles barely penetrated into the soil as the raft was pushed between borehole locations. It is understood that the water was at its highest level at the time of the investigation.

It is assumed that the Highway crossing of this water will take the form of a central bridge connected to the shoreline by earth fill causeways. The bridge presumably will be located adjacent to the deepest water in order to allow for future navigation.

Field Work

A total of 6 borings were made at representative locations across this stretch of water. Four of them were located near the centre of the river where the water was deepest. The locations of the holes are shown on Dwg. 1. All work was performed from a raft which was made from empty drums and from adjacent trees.

Samples were taken generally at 5 foot intervals and, since the soil consisted entirely of fine to coarse sand, they were recovered in the disturbed state using the conventional 2 inch O.D. split spoon. In many instances the sample was lost on withdrawal of the spoon from the BX casing and it was necessary to recover the soil using a side sampler.

The borings ranged in depth from 60 to 70 feet. This was the maximum depth that the casing could be driven and still be recovered after sampling was complete. The friction on the casing was sufficiently large that it was very difficult to recover it, even when driven to these depths. Attempts were made to probe deeper by driving a cone from the bottom of the casing. The maximum penetration depth with this scheme was about 90 feet. A cone penetration test was also performed adjacent to each hole before the boring was made. A static penetration test was carried out in hole 4 using an A drill rod.

The levels of all holes coincided with the water surface. The level of the water was referred to a nail in a tree at the location shown on Dwg. 1.

#### Subsoil and Discussion of Friction

The results of the six borings for this project are recorded on Dwgs. 2 to 7 of this report and on the estimated stratigraphical profile of Dwg. 1. It is seen that the subsoil is remarkably uniform in lateral directions; it consists of sand which grades from coarse to fine with depth. A very thin veneer of mud covers the lake bottom. Refusal was encountered in hole 1, only, where the cone bounced at a depth of 88.5 feet below the water surface. Typical grading curves for the sand are indicated in Dwg. 9.

According to the penetration resistance or N values determined in the sampling program, the sand, below the lake bed, has a relative density ranging from loose to medium dense only. A similar situation was encountered for other bridge site investigations in this general area. Despite this apparent loose condition, however, considerable friction resistance was experienced, both in the driving and withdrawal of casing and cone rods. It was felt, therefore, that the soil is somewhat more competent than is inferred from these empirical indicators.

In order to obtain an approximate indication of the friction resistance generated in the soil, a test was devised which involved the measurement of the force required to push an A drill rod into the soil below casing level. This test had been performed previously for the Giles Creek crossing about 3 miles to the west.

The results of this testing, which was carried out between 18 and 49 feet in hole 4, are presented on Dwg. 8. The graphs, shown in this drawing, illustrate the measured increase in resistance to the A rod with increasing penetration below the bottom of the casing. The slope of this line or increase in resistance with depth is due almost entirely to the friction generated on the side of the A rods. The additional increase in bearing at the tip of the rods is negligible.

An allowance has been made for the increase in end bearing capacity in the computation of the friction coefficient K in Dwg. 8. In all cases it is seen that the value of K is equal to or greater than 2.0. The coefficient K is determined from the expression:

$$K = N \tan \phi R$$

where:        N        is the horizontal earth pressure coefficient acting on the shaft

$\phi$         is the angle of internal friction of the soil

and            R        is a roughness coefficient equal to or less than unity depending on the roughness of the rod

It follows, therefore, that, for  $K = 2$ , the horizontal pressure coefficient, N, on the rod must be greater than 2 unless, of course, the angle of internal friction of the soil is equal to 45 degrees. A value of  $N = 2$  or more is well in excess of the earth pressure at rest condition. A similar result was obtained for static tests on an A rod at the Giles Creek Crossing about 3 miles to the west.

An analysis of a pull-out test on an H pile in the dense sand underlying Ashbridge's Bay in Toronto has indicated a generated value of K equal to 0.2. This is approximately equivalent to an earth pressure at rest condition around the pile. Similarly, published information on pull-out tests on step-tapered Raymond piles in coral sand has shown values of K well in excess of 2 and a horizontal earth pressure coefficient, N, equal to the passive earth pressure state.\* It is concluded from these records, together with the results of the tests on the 1 - 5/8 inch diameter A rod, that the magnitude of friction generated around a pile shaft is dependent upon the degree of displacement of the surrounding soil. With a timber pile or a cylindrical steel pile this displacement will be sufficient to develop the full passive resistance of the soil. It is reasonable to conclude, therefore, that a value of K equal to or in excess of 2 should be obtained. Consequently, any estimates of cylindrical pile capacity, based on an assumed value of  $K = 1$ , should be quite conservative.

#### Foundations

In view of the difficulties associated with the installation of footings below the lake bed in loose sand, a pile foundation would seem to be the only reasonable method of support for this bridge. The use of end-bearing piles is feasible, although the refusal level has not been established within the first 88 feet of depth below the water surface. The only reasonable alternative, therefore, is cylindrical friction piles.

\* "Pulling Tests on Piles in Sand" - H.O. Ireland - 4th Int. Conf. on Soil Mechanics and Foundation Engineering - 1957

In the foregoing section, arguments have been presented in support of a proposal for friction piles in this "loose" sand. It has been concluded that a high friction resistance will be developed and that the use of a friction factor  $K = 1$  will result in conservative design. This is the reasoning presented for other bridge sites in this area.

The estimated safe capacities for timber piles and cylindrical steel piles, driven into this sand deposit, have been indicated in Table 1. The assumptions used in the preparation of these estimates also are shown.

On the basis of these tabulated values, it is recommended that Class B timber piles should be designed for a load of 20 tons, which capacity should be attainable at a depth of 30 feet below the river bed. A design load of 50 tons should be used for 12 inch O.D. steel piles and this capacity should be developed after a penetration of 40 feet into the sand. It is recommended that these estimates be confirmed by a least one load test.

The settlement associated with the application of the foundation loading should be well within tolerable limits and it will occur as soon as load is applied. The settlement under the weight of the causeway fill also will be immediate and of small magnitude. Since the causeway fill must be placed first, it will not cause any movement of the bridge structure.

The subsoil being entirely granular in nature, there will be no embankment stability problem. In addition, since the river bed is clean, fill can be placed directly on it without concern about entrapped organic material.

The causeway approaches to this bridge pass across a long narrow body of water having an unrestricted reach of 2500 feet to the north and a lesser reach of about 900 feet to the south. According to recent studies by the Corps of Engineers, a wave 1 foot high can be developed under a steady 30 m.p.h. wind over a reach of about 2500 feet.\* If the wind maintains a velocity of 60 m.p.h. for 10 minutes, the wave height increases to 2 feet. The recommended rip rap protection against these waves for a 3:1 embankment slope is as follows\*\*:

Maximum size of stone	-	1000 lbs.
25% greater than	-	300 lbs.
45-75% ranging from	-	10 - 300 lbs.
25% less than		10 lbs.
nominal thickness	=	18 inches.

Since large stones probably are in short supply in the immediate area, the alternative protection is to use flatter slopes. An examination of beaches by the Corps of Engineers has indicated that coarse

\* - Slope Protection along Reservoirs: H.L. Drake, U.S. Corps of Engineers  
A.S.C.E. Apr. 11/61

\*\* - Design of Small Dams: U.S. Bureau of Reclamation, Table 17, P.207

gravel surfaces remain stable on slopes of approximately 5:1. The corresponding equilibrium slope for medium to coarse sand is 10:1. Since the water is quite shallow at this crossing, and since ample finer granular material should be available, the use of flatter slopes may provide economic protection for possible heavy wave action. The natural slope in the first 25 feet from each shore at this site is approximately equal to 6:1.

#### Recommendations

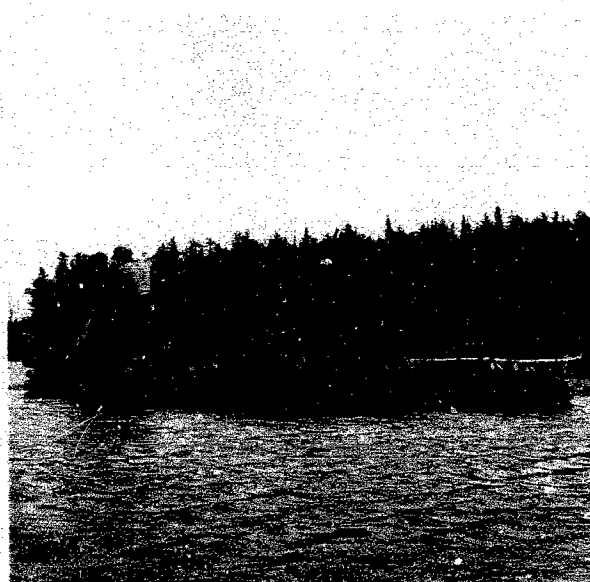
Sand, grading from coarse to fine and extending at least to a depth of 88 feet, underlies the river bottom at this site. Although field penetration tests indicate this material to be loose, it is believed to be sufficiently competent to support large displacement friction piles. It is recommended, therefore, that the structure be supported on Class B timber piles, or on cylindrical steel piles. The estimated safe loads for these mediums of support are 20 tons at 30 feet, and 50 tons at 40 feet, respectively. These estimates should be confirmed by one load test. There is an ample supply of timber adjacent to this crossing.

The embankment approaches to the bridge will be exposed to considerable wave action. The use of flat slopes in the order to 5:1 are suggested as protection against erosion.

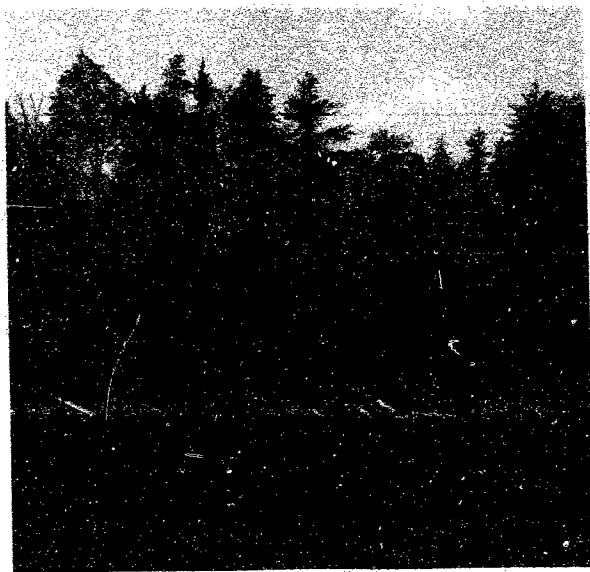
WAT/lt  
July 1962.



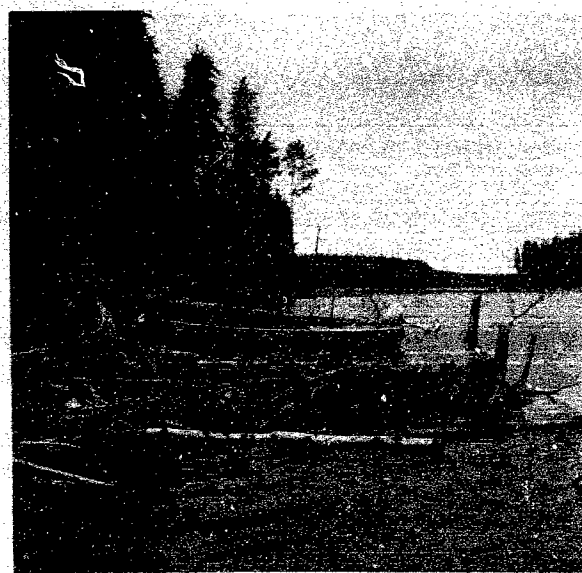
*W. Trow*  
William A. Trow (P. Eng.)



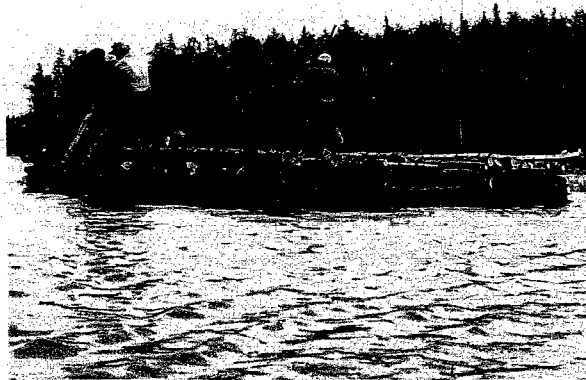
View of Raft



View of West Bank  
From Raft



View of West Bank  
From The South



View of Raft



View of West Bank  
From Raft



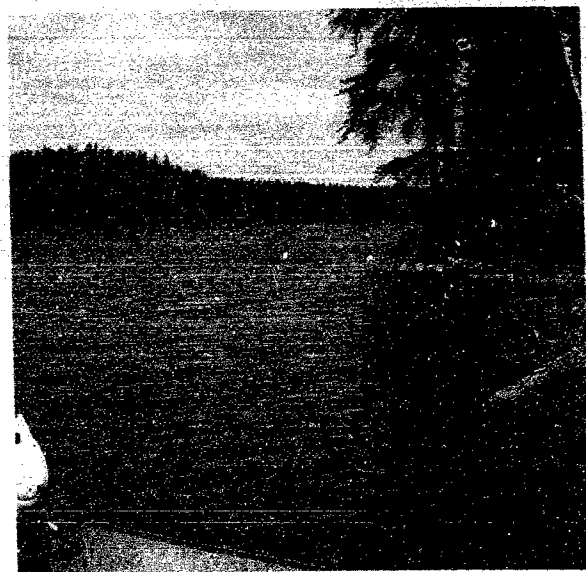
View of West Bank  
From The South



East Bank  
And Raft Looking South



East Bank  
From The North



East Bank  
From The South





East Bank  
And Raft Looking South



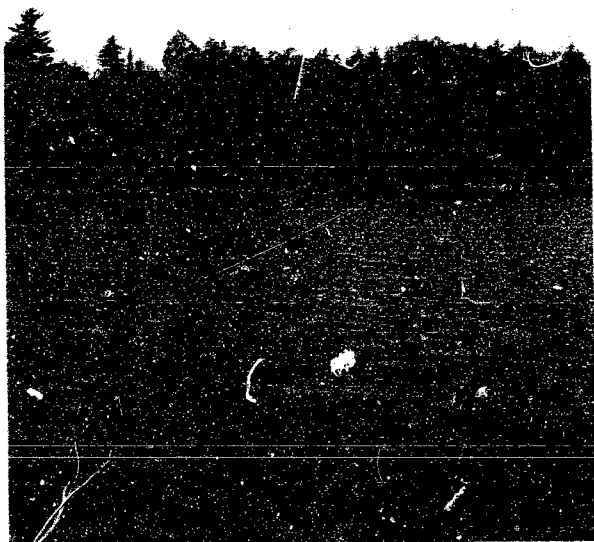
East Bank  
From The North



East Bank  
From The South



From The West  
Bank - Along CL



View From The East Bank  
Looking Along Centre Line



Looking North  
From The Centre Line



From The West  
Bank - Along CL



View From The East Bank  
Looking Along Centre Line



Looking North  
From The Centre Line

TABLE NO. 1SUMMARY OF ESTIMATED PILE CAPACITIES IN TONS<sup>+</sup> - Little Pine Lake Crossing Hwy. 101

Depth ** Below Stream Bed	Timber Pile*	Cylindrical Steel Pile <sup>++</sup>
30	23.5	33.5
40	39.5	54.5
50	60	80.5
60		111

\*\* - Assume extra capacity due to weight of fill offset by possibility of scour to greater depths.

\* - Tip diameter = 8 ins.; average diameter = 10 ins.

++ - O.D. = 12 inches.

+ - Estimated from expression:

$$Q = \frac{1}{F} ( A (0.3\gamma DN_{\gamma} + \gamma ZN_q) + \frac{1}{2}\gamma Z^2 PK )$$

where: A = area of pile tip in sq.ft.

$\gamma$  = 63 p.c.f., is the submerged weight of the soil.

D = tip diameter of pile in feet

Z = depth of pile below stream bed level

$N_{\gamma}$  &  $N_q$  = are bearing capacity factors - assume = 30

P = average perimeter of pile in feet

K = is the friction coefficient - assume = 1

F = 2, is the factor of safety.



Note: Recommended loadings - subject to load test

Timber pile = 20 tons at Z = 30 feet

Steel pile = 50 tons at Z = 40 feet.

The factor of safety in these recommendations is somewhat greater than 2.

### LEGEND

2" O.D. SPLIT TUBE        
2" I.D. SHELBY TUBE        
2" DIA. CONE              

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

SAMPLE TYPE

2" O.D. SPLIT TUBE \_\_\_\_\_

2" I.D. SHELBY TUBE \_\_\_\_\_

3" O.D. SHELBY TUBE \_\_\_\_\_

BOREHOLE NO. 1  
PROJECT Hwy. 101 Little Pine Lake, WP 187 - 60  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.3 ft.  
DATUM See Dwg. 1.

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.	
				SHEAR STRENGTH P.S.F.							
				10	20	30	40				
	Top of Raft Lake Surface	101.3	0								
	Water	100									
	Lake Bed	94.7									
	SAND-medium to coarse, some gravel sizes.		10						1		
									2		
			20						3		
			76.5							4	
			30						5		
									6		
			40						7		
									8		
			50						9		
									10		
			60						11		
									12		
			31.7	70						13	
			80								
			90								
			100								
			110								

-fine grey; very fine below about 38 ft.; silt layers below approx. 44 feet.

-drove cone below casing to refusal and bouncing at 88.5 ft. or El 11.6 ft.

End of Bore

Notes: 1) Boring cased with BX pipe to full sampling depth.

2) Wet sampling methods used; wash rods withdrawn slowly and pipe maintained full of water when rods withdrawn.

3) Many split spoon samples lost, recover soil using side sampler.

4) Cone driven 12 ft. to south before boring and - test terminated at 66 ft. - no refusal.

Datum for cone test = water surface.

At 90 ft. test (bore terminated)

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)<sub>v</sub>

NATURAL MOISTURE CONTENT  
AND LIQUIDITY INDEX

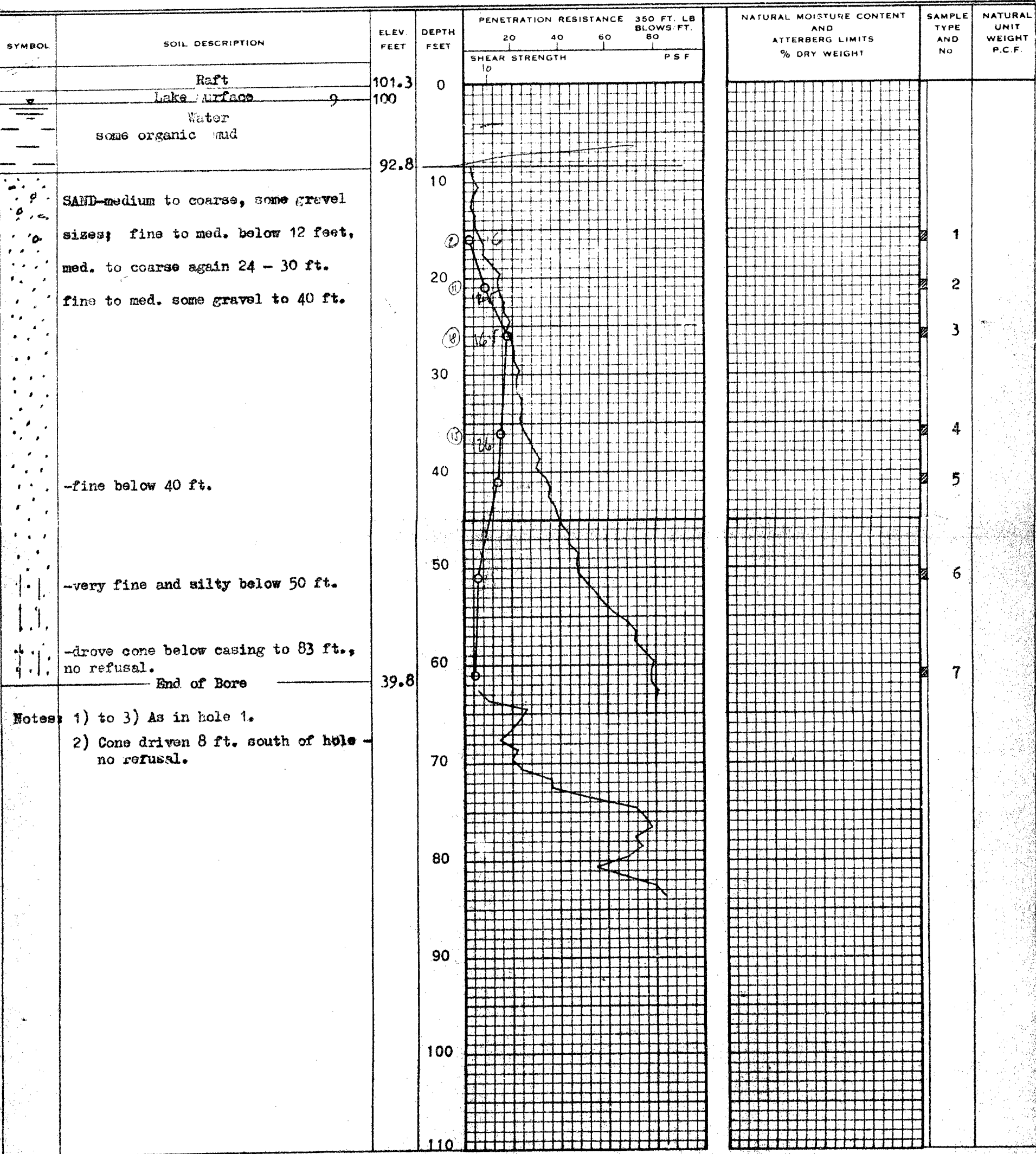
ATTERBERG LIMITS

LIQUID LIMIT  
PLASTIC LIMIT

SAMPLE TYPE

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

BO: HOLE No. 2  
PROJECT Hwy. 101 - Little Pine Lake, WP 187 - 60  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.3 ft.  
DATUM See Dwg. 1.





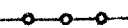
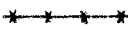

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION




DRAWING No. 4  
PROJECT No. J886

## 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) 

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX 

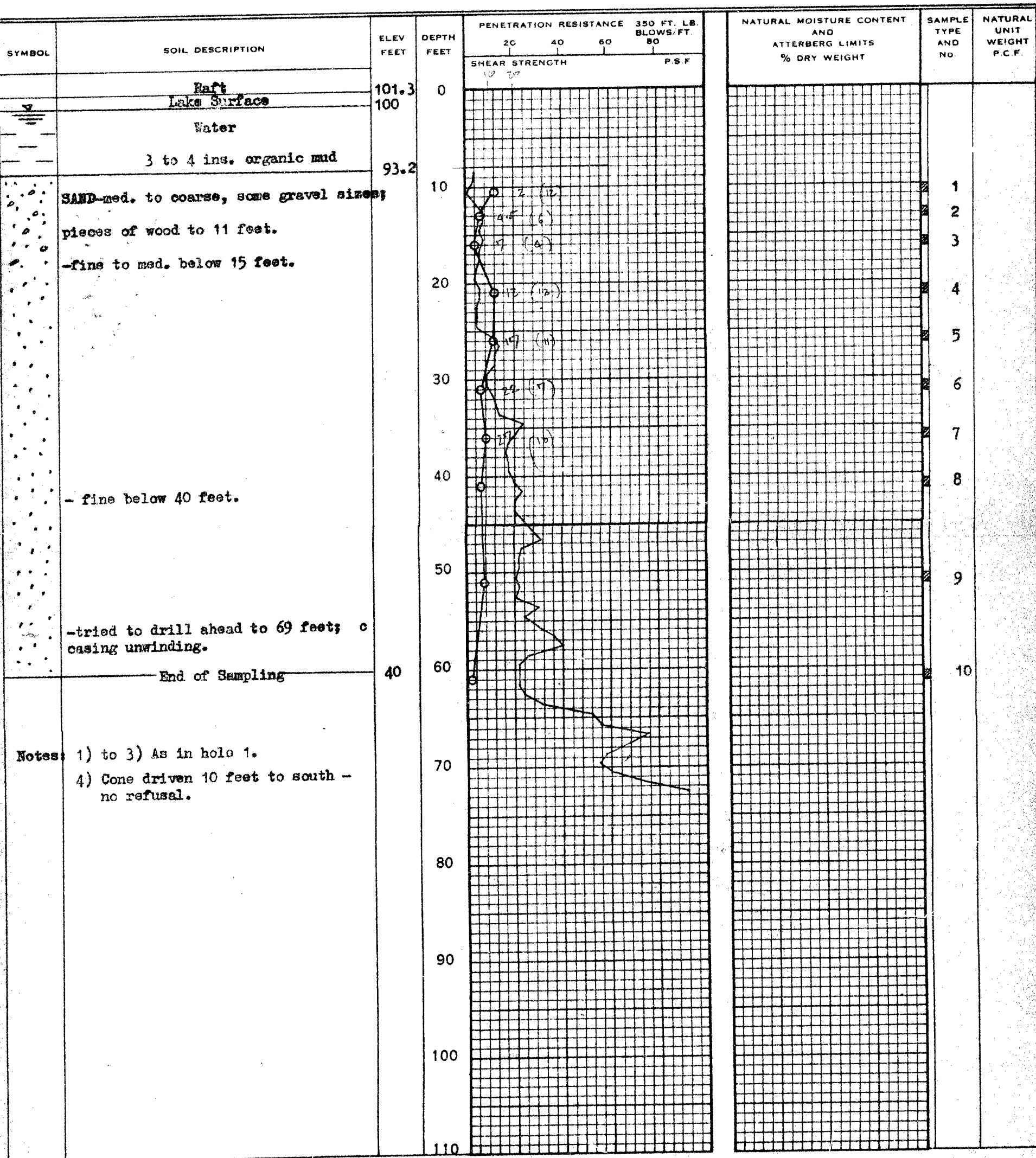
### ATTERBERG LIMITS

LIQUID LIMIT   
PLASTIC LIMIT 

### SAMPLE TYPE

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

BOREHOLE No. 3  
PROJECT Hwy. 101 - Little Pine Lake, WP 187 - 60  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.3 ft.  
DATUM See Dwg. 1.



## LEGEND

BOREHOLE NO. 4  
PROJECT Hwy. 101 - Little Pine Lake, WP 187-60  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.3 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 (S) ⊕

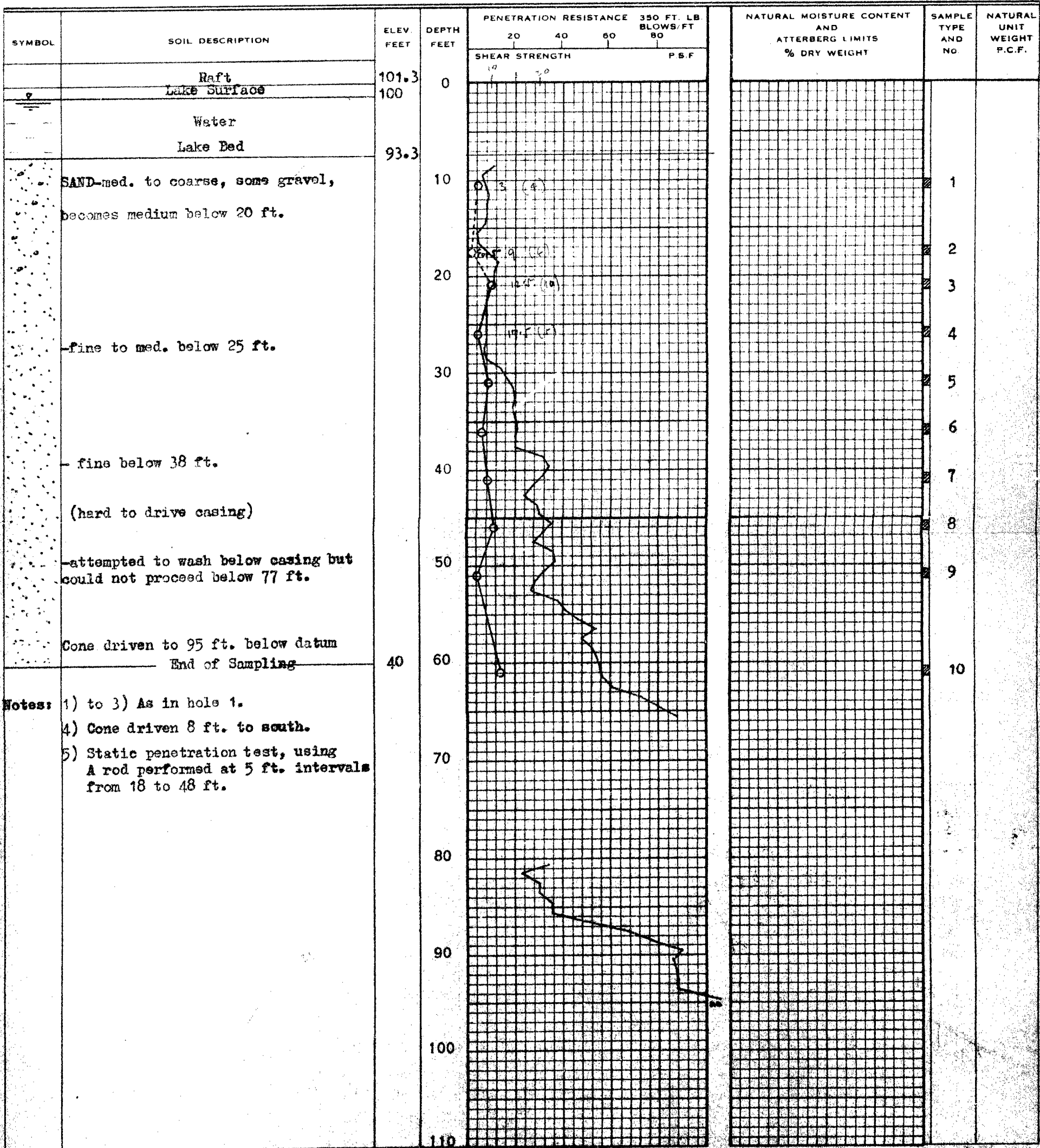
## NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

## ATTERBERG LIMITS

LIQUID LIMIT —○—  
PLASTIC LIMIT ————

## SAMPLE TYPE

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





## 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 ———

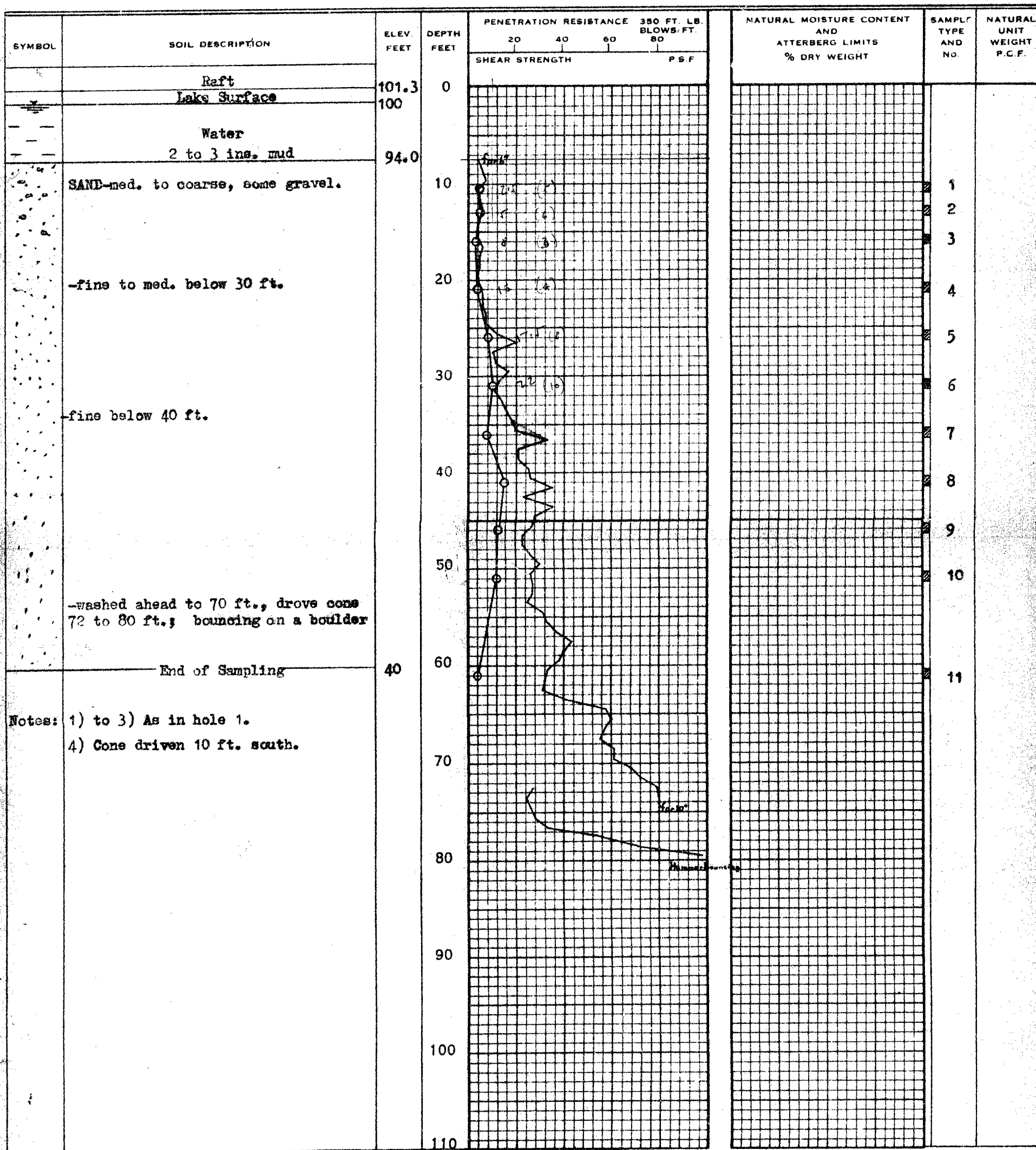
## SAMPLE TYPE

2" O.D. SPLIT TUBE —■—

2" I.D. SHELBY TUBE —■—

3" O.D. SHELBY TUBE —■—

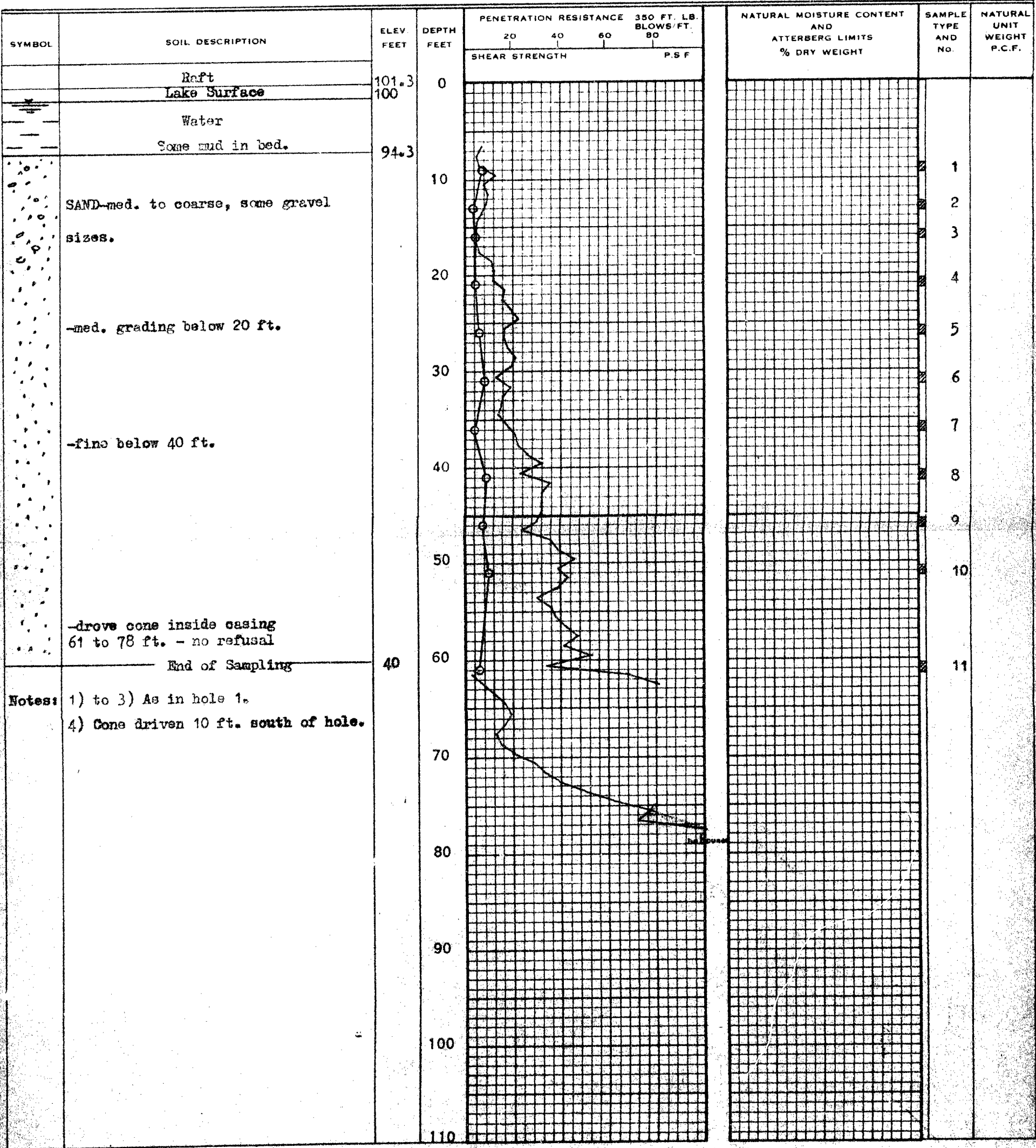
BOREHOLE No. 5  
 PROJECT Hwy. 101 - Little Pine Lake, WP 187 - 60  
 LOCATION See Dwg. 1.  
 HOLE LOCATION See Dwg. 1.  
 HOLE ELEVATION 101.3 ft.  
 DATUM See Dwg. 1.

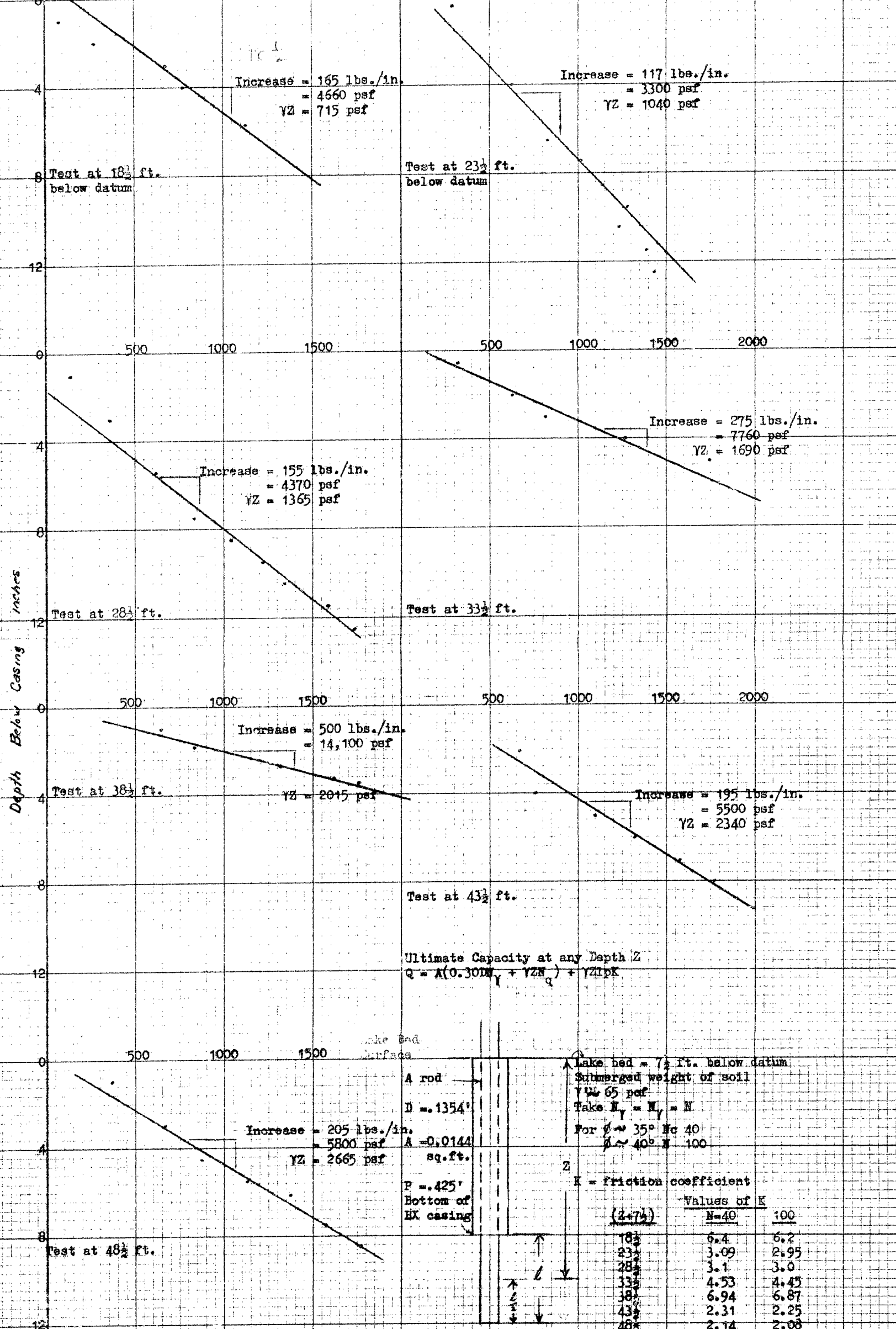


BOREHOLE No. 6  
PROJECT Hwy. 101 - Little Pine Lake, WP 187 - 60  
LOCATION See Dwg. 1.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.3 ft.  
DATUM See Dwg. 1.

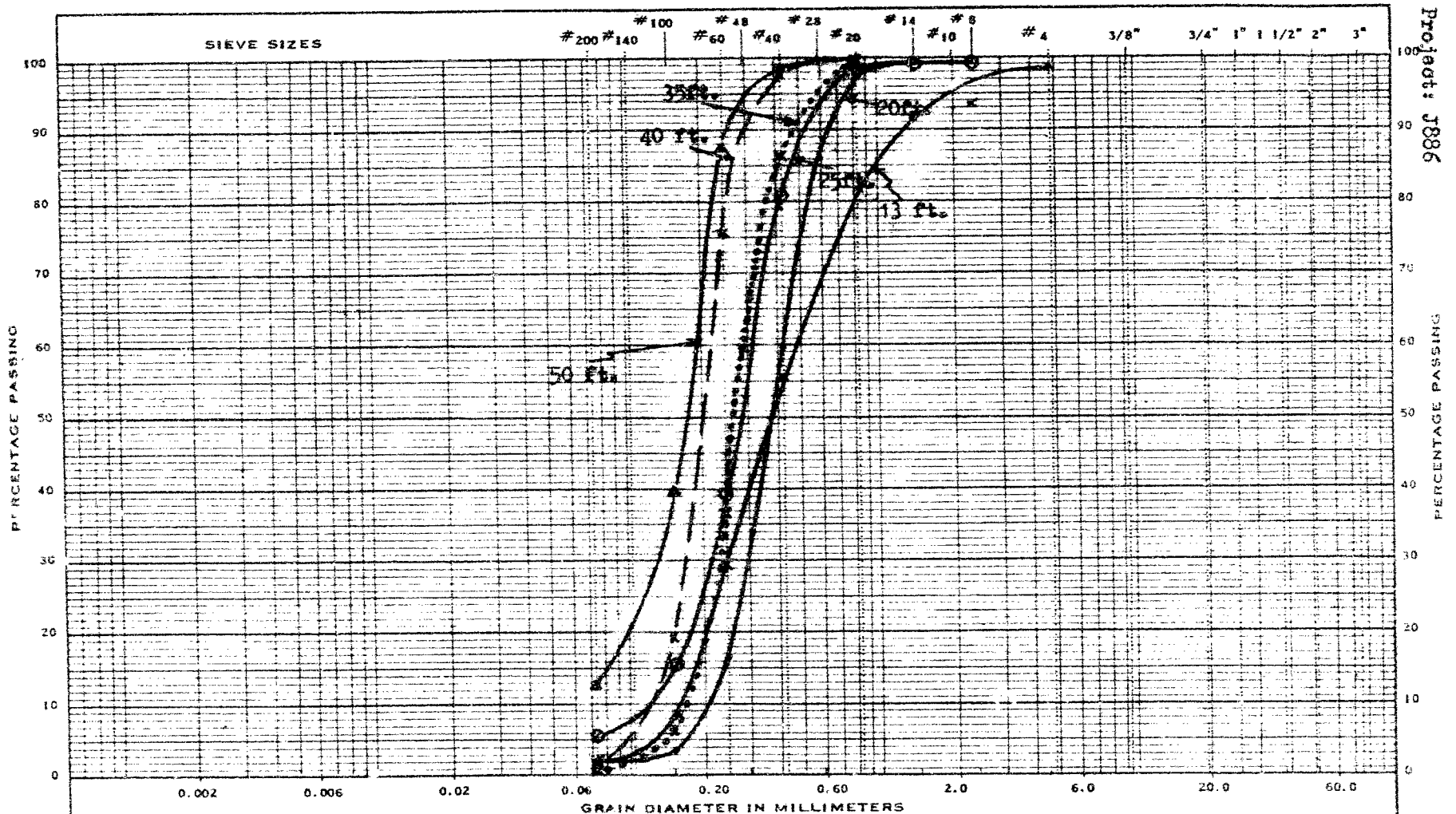
PENETRATION RESISTANCE  
2" O.D. SPLIT TUBE —○—○—○—  
2" I.D. SHELBY TUBE \* \* \* \* \*  
2" DIA. CONE ————  
SHEAR STRENGTH  
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕  
UNCONFINED COMPRESSION ⊗  
VANE TEST AND SENSITIVITY (S)<sub>v</sub> †

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 —■—





# MECHANICAL ANALYSIS



MODIFIED M.I.T. CLASSIFICATION  
**GRAIN SIZE DISTRIBUTION CURVES**  
**HOLE 4 - LITTLE PINE LAKE**

WILLIAM A. TROW AND ASSOCIATES LTD.

Project: J886

PERCENTAGE PASSING

DWG. 9

Department of Highways

COPY

For the Information of

Mr. A. Stermac  
Principal Foundations Eng.  
Room 107 Lab. Bldg.

Bridge Division,  
May 30, 1962.

RM 110  
File Ags

410-2
GEOCRES No.

W. Trow and Associates,  
1850 Jane Street,  
WESTON, Ontario.

Attn: Mr. W. Trow

Re: W.P. 148-62  
Little Pine Lake  
Hwy. #101, District #18

Gentlemen:

Confirming our conversation of this morning (May 28) the new alignment crosses Little Pine Lake at a distance of 350' north of the existing line on the east bank and 400' north on the west bank. Your men were going to sound a new alignment and investigate for a bridge approximately 75' long crossing the deepest portion of the river.

We are considering the use of a timber trestle at this location. If you find that the length of timber piles required is excessive you will determine the bearing permissible on steel tube piles.

Yours truly,

SMCC/m  
c.c. A. Stermac  
J.C. McAllister

SMCC  
S. McCombie, P. Eng.  
Bridge Planning Engineer