

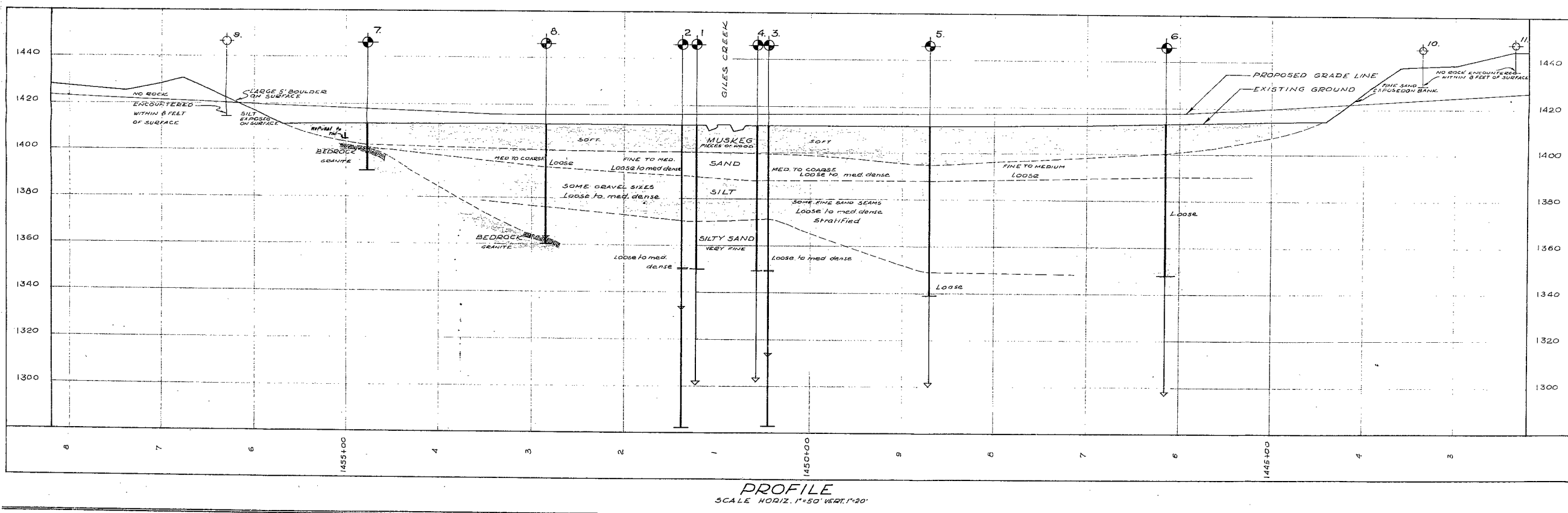
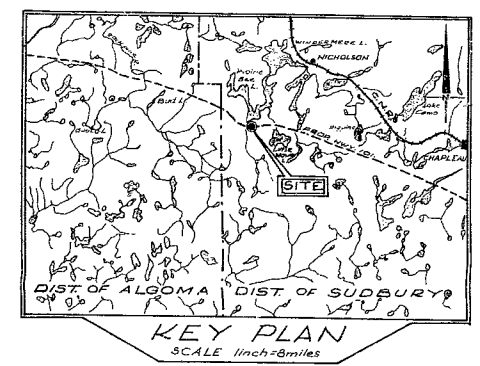
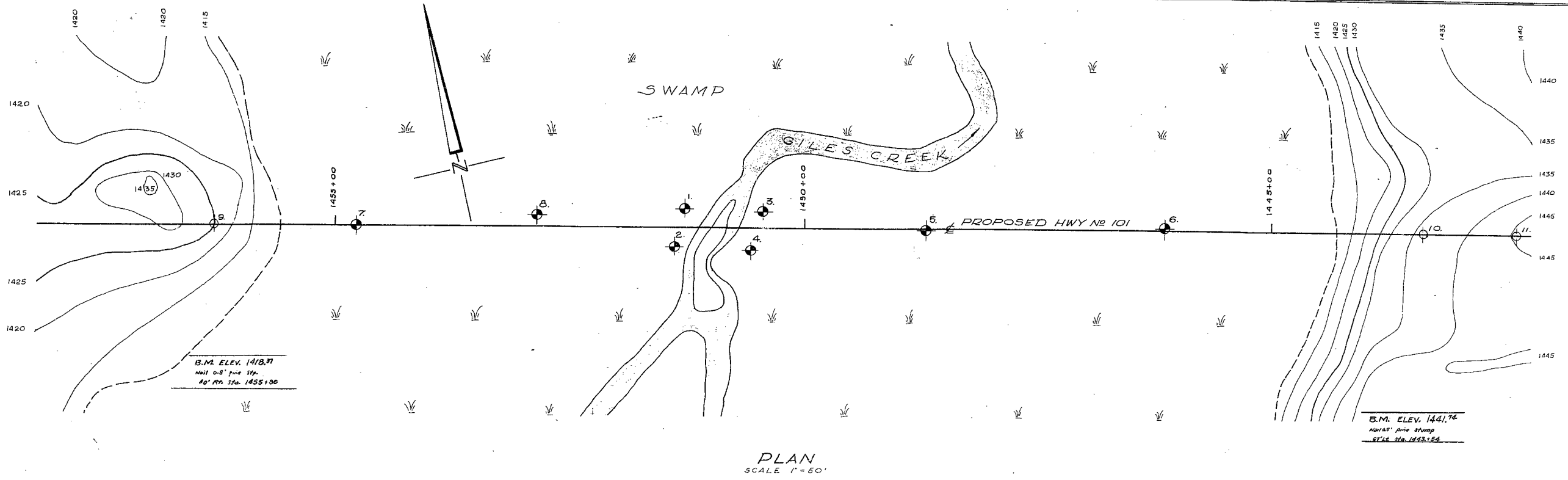
#62-F-240C

W.P. #70-62

Hwy #101

GILES CREEK

CROSSING



LEGEND			
	BORE HOLE & PENETRATION TEST		
	END OF HOLE		
	END OF SAMPLING		
	END OF PENETRATION TEST		
	PROBE		
HOLE NO.	ELEVATION	STATION	DISTANCE FROM E
1.	1411.4	1451+27	20' RT
2.	1411.6	1451+38	21' LT
3.	1411.89	1450+33	17' RT
4.	1411.6	1450+51	24' LT
5.	1411.4	1448+64	1' LT
6.	1412.2	1446+14	2' RT
7.	1411.3	1454+65	2' RT
8.	1410.9	1458+85	11' RT

W.A. TROW & ASSOC. LTD.  
FOUNDATION INVESTIGATION

**PROPOSED HWY. 101 &  
GILES CREEK CROSSING**

W.R. NO 70-62 PROJECT NO J863 DATE JUNE 1962 DWG. NO. 1

Mr. A. M. Teye,

June 15, 1962.

Bridge Engineer.

FOUNDATION INVESTIGATION REPORT

Materials & Research Division,

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

(Foundation Section.)

Attention: Mr. L. McCosbie.

Re: W.P. 70-62 - Giles Creek Crossing,  
Hwy. #101, District No. 13, Sault Ste. Marie.

Attached, we are forwarding to you the above-mentioned report prepared and submitted by the Consultant, W. A. Trow & Associates, Ltd.

We have reviewed the report and find the factual data adequate and well presented. Because of the inherent and unavoidable uncertainties of foundations supported on friction piles, we would strongly support the Consultant's suggestion that the bridge location be altered by moving it to the west side of the swamp. Much excavation will have to be carried out for the road, itself, and some more excavation for the realignment of the creek would not present any difficulties nor excessive expenditure. It is believed that this additional cost would be balanced by the more economical design feasible at the new proposed location.

Should there be any additional information required in this connection, please do not hesitate to contact our office.

A.G./dter

Attach.

cc: Messrs. A. M. Teye (2)  
R. E. Tregaskes  
H. G. Medillan  
L. McArthur  
J. E. Collins  
J. A. Saint  
T. J. Kovich  
J. Roy  
J. L. Graspier  
C. Korman  
A. Watt  
Foundations Office  
Gen. Files.

*A. G. Sterner*  
A. G. Sterner,

PRINCIPAL FOUNDATION ENGINEER

21<sup>st</sup> June 1962.

- ① Discrepancy between stake at site & preliminary drawing given to ~~contractor~~ consultant mentioned to J McCombie
- ② Design load used is 11 ton so no pile loading test will be carried out

WJH.

**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.

Project: J863

62F240C

June 14, 1962

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

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

Attention: Mr. A.G. Stermac

Re: Foundation Investigation  
Proposed Crossing Giles Creek  
Highway 101 WP 70-62

Dear Sirs:

The enclosed report contains the results of a foundation investigation recently completed at this swamp crossing.

The observations and conclusions of this submission are essentially as given in our preliminary letter of June 4. The only modification is in the estimated capacity of friction piles. It is now our opinion that the pile loads can be developed at slightly higher levels than was suggested in this letter.

The recommendations of this report briefly are as follows:

- 1) The bridge, if placed adjacent to the existing channel of Giles Creek, must be supported on cylindrical displacement piles. The estimated safe capacities and associated depths are indicated in Table 2. A pile load test, to confirm these estimates, is suggested.

2) If the bridge is moved to the west side of the swamp, it can be supported on end-bearing piles driven to bedrock which comes quite close to the surface in that area.

3) The 10 to 15 feet of muskeg covering this swamp, must be removed along the route of the highway. Since it will probably compress rather than displace under load, it should be removed by drag line.

We shall be pleased to discuss any matter that may occur to you after you have reviewed the contents of this report.

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, ONTARIO

FOUNDATION INVESTIGATION  
PROPOSED CROSSING GILES CREEK  
HIGHWAY 101 WP 70-62

Project: J863

William A. Trow and Associates Ltd.

June, 1962

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ENCLOSURES

Photographs of Site

Table 1 - Summary of Static Penetration Tests  $\frac{1}{2}$  inch Rod

Table 2 - Estimated Pile Capacities

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Static Penetration Test Results Dwg. 10

Drained triaxial Test Result " 11

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FOUNDATION INVESTIGATION  
PROPOSED CROSSING GILES CREEK  
HIGHWAY 101 WP 70-62

Site and Project

This creek crossing lies on a section of the route of proposed Highway 101, which passes about 1 mile to the south of Prairie Bee Lake approximately 24 air miles west of Chapleau, Ontario.

Although the shoreline and islands of this lake are composed of sand beaches and bedrock outcrops, this crossing passes over a treeless swamp which extends for about 2 miles south of the lake.

The swamp is drained by Giles Creek which flows, in a meandering course, in a northerly direction to the lake. At the time of the investigation, in late May, the water was flowing at a rate of approximately 1 foot per second, although the flow varied depending upon the width of the channel; the fine sand in the creek bed was just barely moving with this flow. The depth of the stream bed varied up to a maximum of about four feet in the locations where scouring had occurred.

The swamp is about 1100 feet wide at this crossing location. The westerly and easterly approaches to the swamp consist of well drained, heavily treed terrain. On the steep east bank, fine sand is exposed, while the soil on the west boundary consists of coarse silt. One large boulder about 5 feet across was noted on this latter hillside. It was thought that bedrock lay close to the surface on this bank, although this impression was not confirmed by shallow probings.

According to present plans, the swamp will be crossed by means of a causeway, which will rise to a height of about 6 feet above the surrounding ground. The type of structure, used to pass over the creek, will be determined by foundation conditions. The fill for the causeway will be obtained from the hillsides which bound the swamp.

Scope of Investigation

A total of eight borings were made across this swamp at the locations shown on Dwg. 1. Four of these holes were concentrated adjacent to the existing creek; the remainder were spread out to the east and west along the centreline of Highway 101. The purpose of these latter borings was to determine the depth and condition of the



surface mantle of muskeg, and, as well, to obtain information at alternative bridge locations in the event that a re-routing of the river is economically feasible.

All borings were made using wet sampling methods. The holes were cased with BX pipe to the maximum sampled depth of 70 feet; sampling to greater depths was precluded because of the extremely high friction resistance acting on the casing. Information about soil conditions below this level was obtained, either by washing or by driving a 2 inch cone below the casing. A cone penetration was also made beside each boring before sampling was begun. Bedrock was cored in holes 7 and 8 which were located to the west of the creek.

In most instances, samples were recovered in the disturbed state using the conventional 2 inch O.D. split spoon which was driven into the soil under an energy of 350 ft.lbs. per blow. Generally the penetration resistance was much lower than the results of the adjacent cone tests, even though care was taken to avoid "quick" conditions in the sand by keeping the casing full of water and by withdrawing the wash rods slowly. When samples were lost in the split spoon, the soil was recovered using a side sampler.

Some two inch I.D. Shelby tube samples also were recovered of the upper muskeg and of the silt deposit found below 20 feet. The purpose of this sampling was to obtain some indication of the unit weight of the soil and to permit a closer examination of stratification and general composition.

Three shallow probings were made on the approaches to the swamp, in order to determine if bedrock would be encountered during excavation of these slopes. Initially, this work was done using a pack sack drill, but, when the water supply became short, the probing was continued by driving a bar with a sledge hammer.

Although the soil appeared loose in the sampled borings, it was quite difficult to withdraw the casing or the drill rods after the cones had been driven. It was concluded that this high friction resistance must also develop when piles are driven into the sand. Consequently, a simple field test was devised in an attempt to provide some indication of this resistance. This test was done in a special boring made 10 feet to the south of hole 1. In brief, the test consisted of measuring the force required to push a 1-5/8 inch diameter A drill rod into the soil below the bottom of the casing. The force was applied by levering with a 10 foot drill rod and by adding other known loads available on the site. A maximum penetration of about 2 feet below casing level was obtained and the load for each inch of penetration test was recorded.

In addition, similar tests were made in other borings using a  $\frac{1}{2}$  inch diameter smooth steel rod. The results of these tests are recorded on Dwg. 10 and on Table 1. Although this method of testing can be considered as crude, it is felt that the results are useful for providing some indication of the friction resistance that can be generated around piles. The results and interpretation of these tests are discussed in a following section.

The elevations of the borings were related to the bench marks indicated on Dwg. 1. Although a hand level was used to obtain this information, the traverse closed within 0.01 feet; this agreement may be fortuitous. The holes were related to a recently installed stake on centre line which was recorded as Station 1450+70. This is exactly 100 feet less than the stations and bench marks indicated on old stakes and on the drawings provided for this project. The west bench mark was found to be 40 feet right of Station 1455+90, rather than 40 feet left.

All equipment for this investigation was flown in from Dalton, Ontario. Access to the swamp was obtained by a raft made out of two powered canoes. With this arrangement several days of man hours were saved in the movement of the equipment between the landing strip and the site.

#### Subsoil

The soil underlying this site consists of a surface mantel of muskeg which is underlain by strata of sand, stratified silt and very fine sand. Since the arrangement and description of these soil layers are given in the borehole logs and in the stratigraphical profile of Dwg. 1, no particular purpose is served in a repetition of this description.

The upper layer of muskeg was composed, for the most part, of organic material including twigs and other pieces of timber. However, it also contained some layers of sand, particularly at lower levels. Field vane tests were made in this material, although the applicability of this test method to this fibrous deposit could be questioned. Undrained shear strengths ranging from 300 to 900 psf and with an arithmetic average of 530 psf were recorded. The attempts to obtain density measurements of the peat were not very successful because of the sand and the fibrous nature of the deposit; the organic material either was displaced before entering the tube or it was compressed. In three tests the unit weight measurements ranged from 62 to 96 pcf. The higher results reflect the presence of sand seams in the muskeg.

Some field vane tests also were made in the deposit of silt since it appeared to be slightly cohesive in the field. Undrained strengths ranging from 450 psf to 1200 psf were obtained. Again, after closer examination of shelby tube samples and of grain size distribution curves, the acceptability of these results is questionable.

The density of the silt was determined both by measurements of the weight and volume of material recovered in a shelly tube, by similar observations on a cylindrical triaxial sample, and by indirect measurements of moisture content and specific gravity. Reasonably good agreement was obtained and an average natural unit weight in the order of 126 pcf was estimated to be representative of in-situ conditions. The results of these measurements are shown in the bore-hole logs.

One drained triaxial test was performed on a sample of silt from hole 4. This test was performed in order to obtain some measure of the angle of internal friction of the sand which is one of the factors influencing the skin friction acting on piles driven into this soil. In view of the approximate nature of the examination of friction resistance at this site, one test was considered sufficient; it was assumed that no effective cohesion is available. An effective angle of internal friction equal to  $\approx 40$  degrees was determined, as shown on Dwg. 11.

Typical grading curves for the various strata at this site are shown on Dwg. 12 and 13.

#### Foundation Capacity

According to the results of the borings in the vicinity of and to the east of Giles Creek, relatively loose soil underlies this site to a depth of at least 130 feet. No refusal situation for end-bearing piles was encountered. In view of the small size of the crossing, the continuation of the borings to greater depths did not seem to be warranted, even if penetration and sampling could have been accomplished without difficulty.

However, even though it was not possible to establish a refusal condition, it was felt, - on the basis of the high resistance experienced during withdrawal of A rods and BX casing, - that skin friction of considerable magnitude would develop around displacement piles. Consequently, a crude but indicative test procedure was devised in the field to measure the magnitude of this force. As indicated in the foregoing pages, this test consisted of forcing a length of A rod into the soil below the casing at various levels between 12 and 47 feet.

The records of static resistance versus depth of penetration into the sand below the casing are shown on Dwg. 10 of this report. The ultimate capacity,  $Q$ , of this shaft at any depth  $Z$  below the ground surface and below the localized zone of influence of the casing, is given by the Terzaghi expression:

$$Q = A(0.3\gamma D N_{\gamma} + \gamma Z N_q) + \gamma Z hPK$$

where:  $A$ ,  $P$  and  $D$  are the cross section area, perimeter and diameter of the drill rod in feet units respectively

$\gamma$  is the submerged unit weight of the soil

$N_\gamma$  &  $N_q$  are bearing capacity factors

$h$  is the embedded length of the rod in the sand

$K$  is a friction coefficient which is a function of the angle of internal friction of the soil and the horizontal earth pressure coefficient with allowance for reduction in sliding resistance between sand and steel.

By applying this expression to the failure loads versus penetration charts of Dwg. 10, it can be shown that the first or end-bearing term of the formula contributes only a very small amount of the total increase in resistance over any given increment of depth. The actual increase over 1 foot of depth in a sand having a submerged weight equal to 64 pcf is about 37 lbs. In the graphical result for the test at the 22 foot level the average total increase in capacity over 12 inches of depth is 700 lbs. Friction on the sides of the rod therefore accounts for 95 percent of the total resistance.

It is concluded from the graphical results and the foregoing reasoning that the slope of these load-penetration test relationships, with slight modification, provides a measure of the friction resistance generated along the shaft of the drill rod. Reducing this to a unit resistance basis and relating the result to the second term of the bearing capacity equation, a value for  $K$ , the friction coefficient, can be determined.

For the test at 22 feet, the average value of  $K$  is computed to be equal to 1.3. This result is much higher than is recorded by Meyerhof\* and is greater than is determined assuming that the horizontal earth pressure against the shaft is equal to the at rest or  $K_0$  condition. For  $K_0 = 0.5$  and the tangent of the angle of internal friction =  $0.84 (\phi = 40^\circ)$ , - neglecting any reduction to  $\tan \phi$  at the sand-steel interface, -  $K$  is computed to be equal to 0.42. Since the measurement of  $\phi$  is believed to be reasonable for this silt, the value of  $K = 1.3$  can only be explained by a mobilized earth pressure approaching the passive condition.

This reasoning is supported to some extent by the results of the penetration tests performed using a  $\frac{1}{2}$  inch diameter rod. The value of  $K$  is generally lower in these tests and it is submitted that this is the result of a smaller displacement of the surrounding sand.

\* "The Ultimate Bearing Capacity of Foundations" Fig. 22  
G.G. Meyerhof, Geotechnique, 1951

The sharp changes in slope of the test curves of Dwg. 10 are believed to be indicative of the passage of the rod into material of different composition. Where the resistance increases sharply, the rod must have reached a thin layer of relatively more dense or coarser soil. In those instances where the resistance fell off or remained constant, a parting or bed of soft clayey silt must have been encountered.

Although it may be considered that undue importance has been placed on the results of these crude tests, the marked increase in shaft friction with depth cannot be disputed. This same observation was made under full scale conditions for a series of pull out tests on Raymond step-tapered piles driven in coral sand\*. The computed horizontal earth pressure coefficients in these tests were found to be equal to the fully mobilized passive earth pressure state. The conclusion of that paper was that a large displacement timber or steel pile develops friction coefficients well in excess of unity when driven in granular soils. This on-site and recorded experience has been used in the determination of the estimated capacities for timber and cylindrical steel piles shown in Table 2. A friction coefficient  $K$  equal to unity has been conservatively assumed. A factor of safety of 2 has been applied to the computations.

These capacities are somewhat higher than were indicated in a preliminary report of June 4. It is recommended that at least one load test be made in order to confirm these estimates. A few hours should be permitted to elapse after driving before performing the test. It should not be expected that the piles will necessarily encounter refusal at any of the test levels indicated, although the resistance to driving will increase gradually with depth. The pile capacities noted in Table 2 are based on an effective penetration below stream bed level. It is assumed that the muskeg below has been replaced by sand fill.

No estimate has been made of the capacity of an H pile, since this type of support is not particularly suited to this soil condition. On the basis of the reasoning expressed in the foregoing paragraphs, the friction developed on the shaft should be of relatively low magnitude since very little displacement of soil particles will occur.

The preceding comments have been concerned with the method of support for a structure located at the existing creek crossing. Although dredging of a new channel would be required, it may be economic to move the crossing location to the west side of the swamp where bedrock lies within easy reach of end-bearing piles. At the hole 7 location dense granite bedrock was found at a depth of 11 feet below the surface.

About 10 feet of muskeg covers the route of Highway 101 across this swamp. The positive method of removing this material is to excavate it using a drag line. Although the width of the excavation will have to be somewhat greater than the roadway, it is anticipated that the sides of

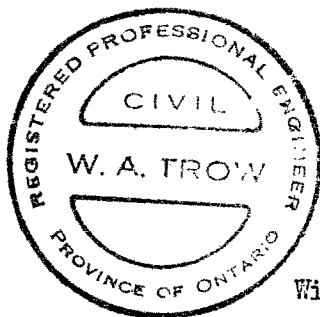
\* "Pulling Tests on Piles in Sand" - H.O. Ireland - 4th Int. Conf. on Soil Mechanics and Foundation Engineering, 1957

the cutting will remain relatively stable and erect during this removal of soil. The earth pressure exerted by this light-weight material, particularly in the submerged state, will be very small. It is sufficiently cohesive and fibrous to support itself under the slightly unbalanced condition created by the excavation. Boreholes driven through the muskeg tended to remain open after the casing was withdrawn. The backfill for the embankments across the swamp will consist of fine sand obtained from the higher ground to the east and west.

#### Recommendations

The following recommendations are submitted as a result of this investigation:

- 1) The structure, if placed at the existing stream location, can be founded on timber piles or on cylindrical steel displacement piles. The estimated capacities for various depths of penetration are given in Table 2. At least one pile load test should be performed in order to confirm these estimates.
- 2) If the structure is relocated to the west end of the swamp, it can be supported on end-bearing piles which will bear on bedrock at shallow depths below the surface. A trestle-type bridge should be feasible at this location. A new creek channel must be dug for this scheme.
- 3) About 10 feet of muskeg must be removed in the swampy approaches to the bridge. In some instances the depth of organic material will extend to 15 feet. Removal using drag line equipment is recommended.



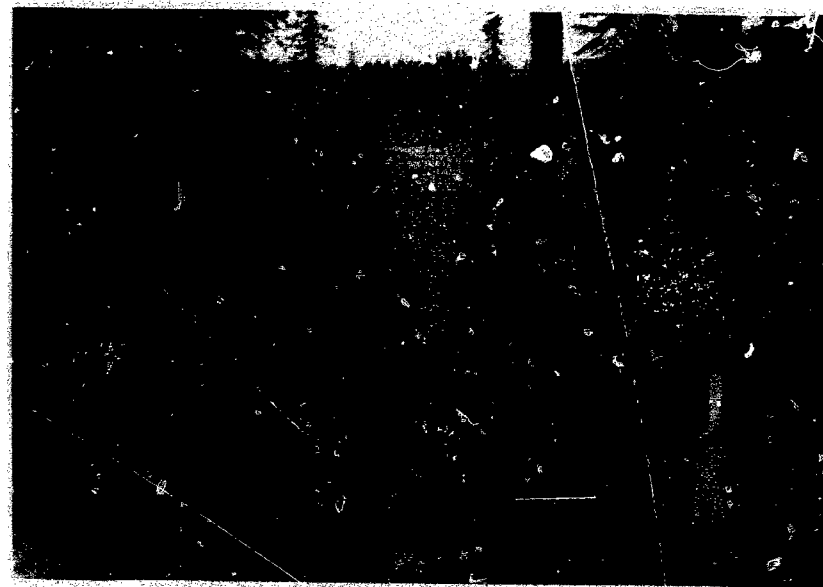
*W. Trow*

William A. Trow, P.Eng.

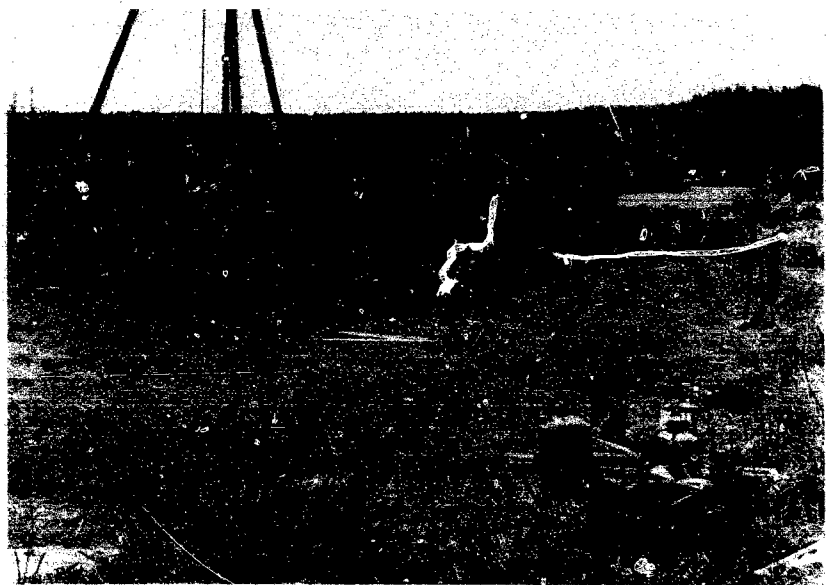
WAT/gc  
J863  
June 1962



Looking East - Drill on Hole 6



Site From East Bank Along Hwy. CL



From East Bank Looking To The Southwest



West Approach To Swamp



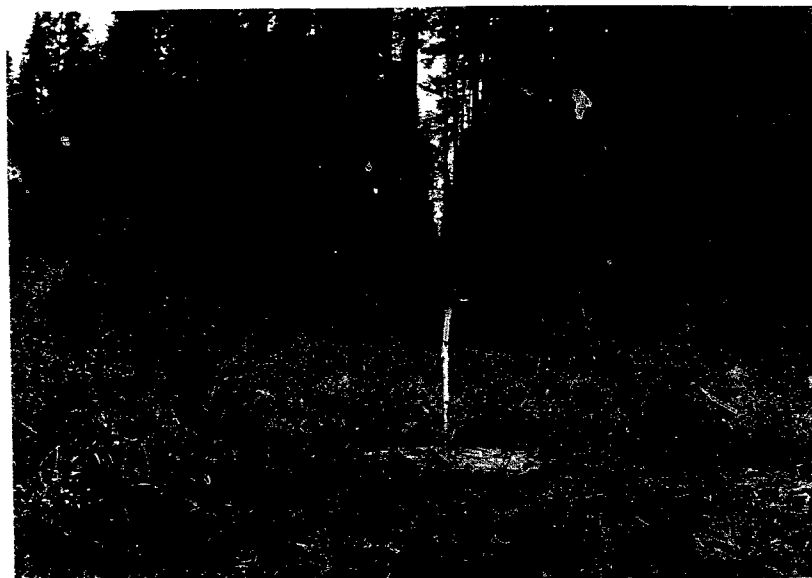
Looking East - Drill on Hole 6



Site From East Bank Along Hwy. 61



From East Bank Looking To The Southwest

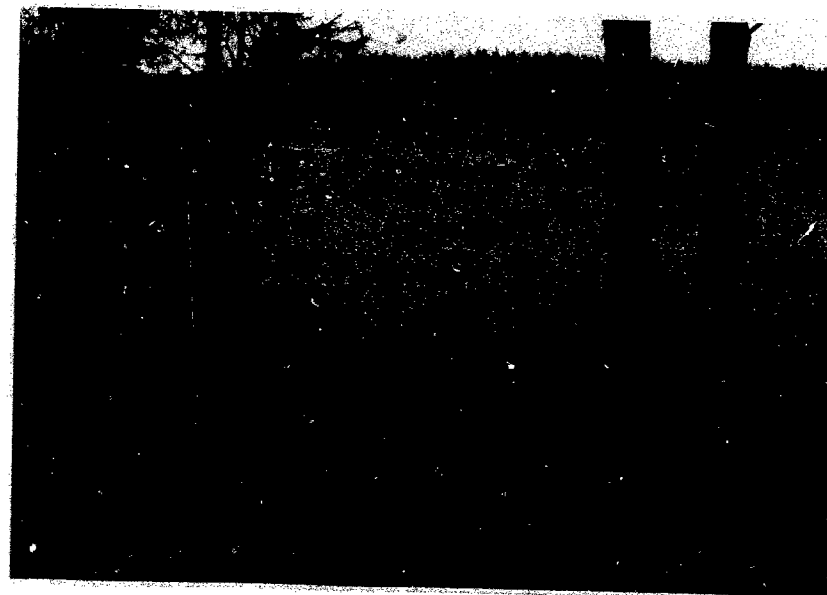


West Approach To Swamp





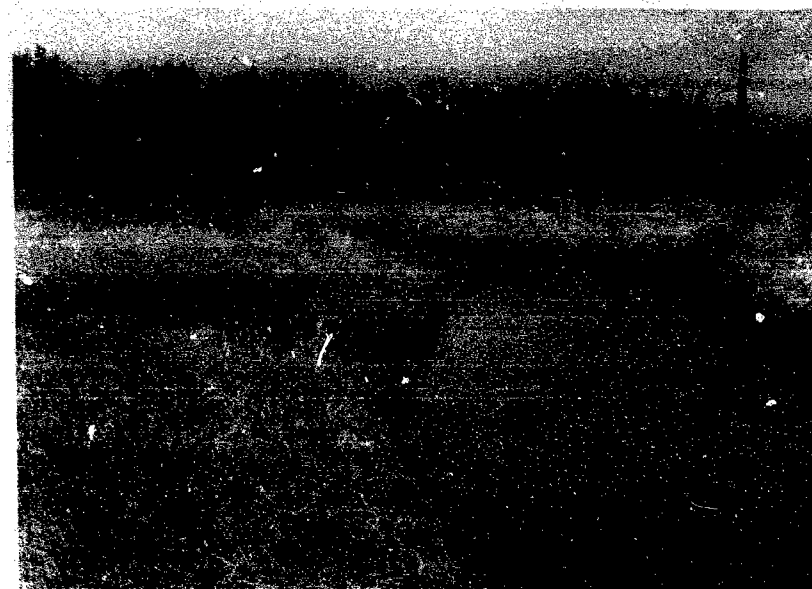
Looking West Along CL Sta 1450+70 Shown In Foreground



Site From West Approach Along CL



Giles Creek From East Bank Looking To Southwest



Downstream Of Site Looking West



Looking West Along CL Sta 1450+70 Shown In Foreground



Site From West Approach Along CL



Giles Creek From East Bank Looking To Southwest



Downstream Of Site Looking West

TABLE 1 - STATIC PENETRATION TEST RESULTS USING  $\frac{1}{2}$  INCH DIAMETER ROD

Hole	Depth Feet	Penetration Of Rod Into Soil Ins.	Soil	Total Force lbs.	Friction Factor K			Corresponding K From A Rod Test	
					For $N^*=30$	40	100	Dwg. 10 $N=40$	$N=100$
3	17.7	9	Fine Sand	185	2.03	1.89	1.03	14 ft. 1.95	1.67
	27.7	9	Silt	151	.52	.42	0	24 ft. 1.43	1.31
	38.2	14	Silt	136	.17	.08			
	47.9	$10\frac{1}{2}$	Very fine sand.	486	1.23	1.11	.13	1.63	1.58
	48.1	$12\frac{1}{2}$		558	1.23	1.14			
6	18.2	15	Fine Sand	119	.7	.61			
	27.9	$11\frac{1}{2}$	Silt	214	.895	.785			
	28.1			258	1.02	.92			
8	16.8	22	Fine Sand	100	.42	.37			

\* End Bearing Capacity Factors - Assume  $N_q = N_\gamma = N$ .

TABLE 2 - ESTIMATED SAFE BEARING VALUES FOR TIMBER AND CYLINDRICAL  
STEEL PILES <sup>+</sup> IN TONS

Depth ** Below Stream Bed	Timber Pile*	Steel Pile <sup>++</sup>
30	17	32
40	37	48
50	57	70
60		95

+ Assume friction factor  $K = 1$ ; factor of safety = 2;  $N_q = N_\gamma = 30$  (Very conservative)

\* Tip Diameter = 8 ins., Average Diameter = 10 ins.

++ O.D. = 12 inches.

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

Note: Some of the capacities indicated are beyond the practical load limit.

Recommended load for timber piles = 20 tons @ 32 feet approx.

" " " steel piles = 50 tons @ 42 feet approx.

BORERHOLE NO. 1  
PROJECT Giles Creek Crossing, W.P. 70-62, Hwy. 101  
LOCATION About 1 mile south of Prairie Bee Lake  
HOLE LOCATION Sta 145+27 - 20 ft. Right - See Dwg. 1.  
HOLE ELEVATION 1411.4 ft.  
DATUM Nail 0.8 ft. Stump 40 ft. Right Sta 1455+90  
= El 1418.37 ft.

## 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 (SI) +

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

## ATTERBERG LIMITS

LIQUID LIMIT —○—

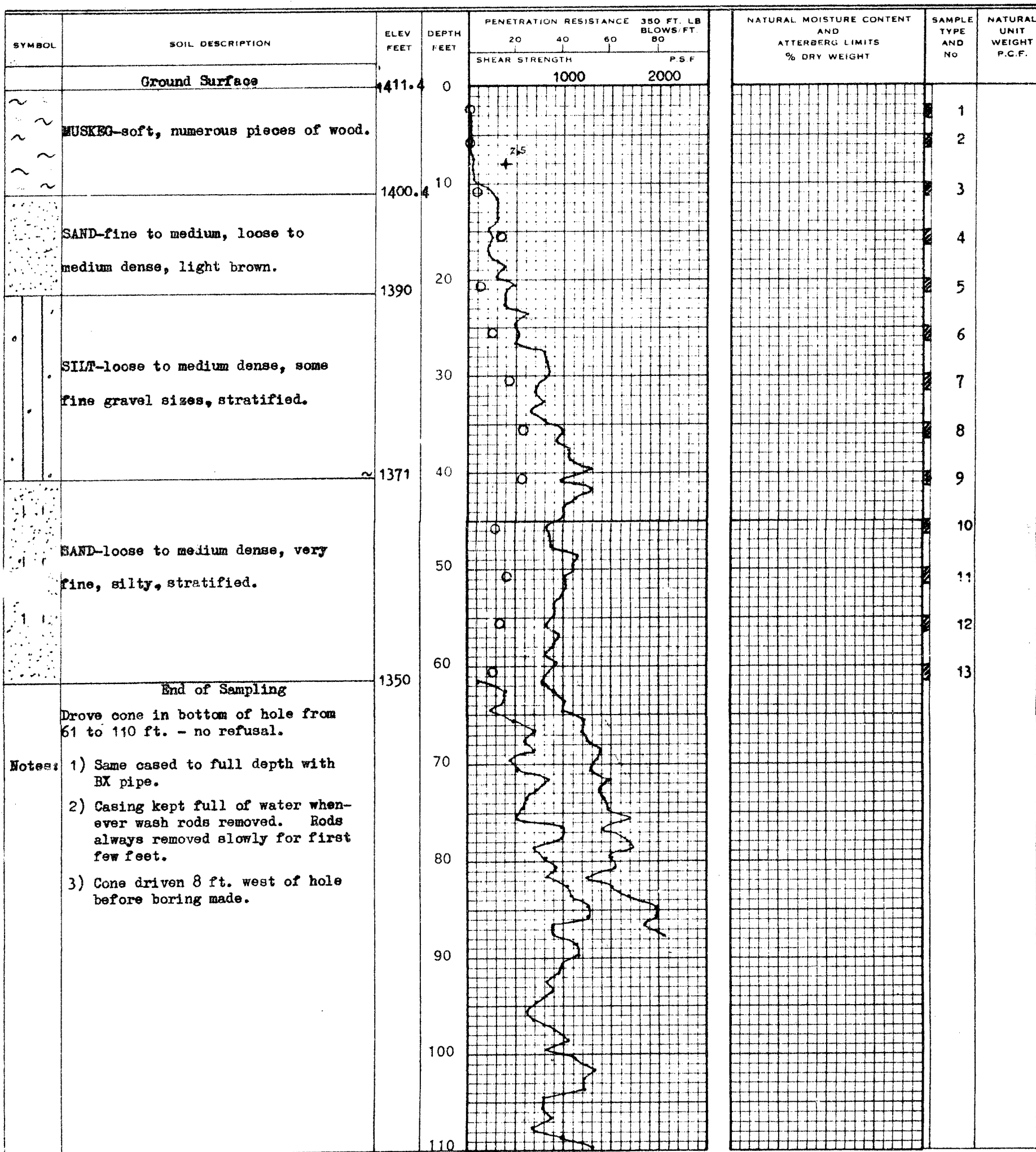
PLASTIC LIMIT ———

## SAMPLE TYPE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

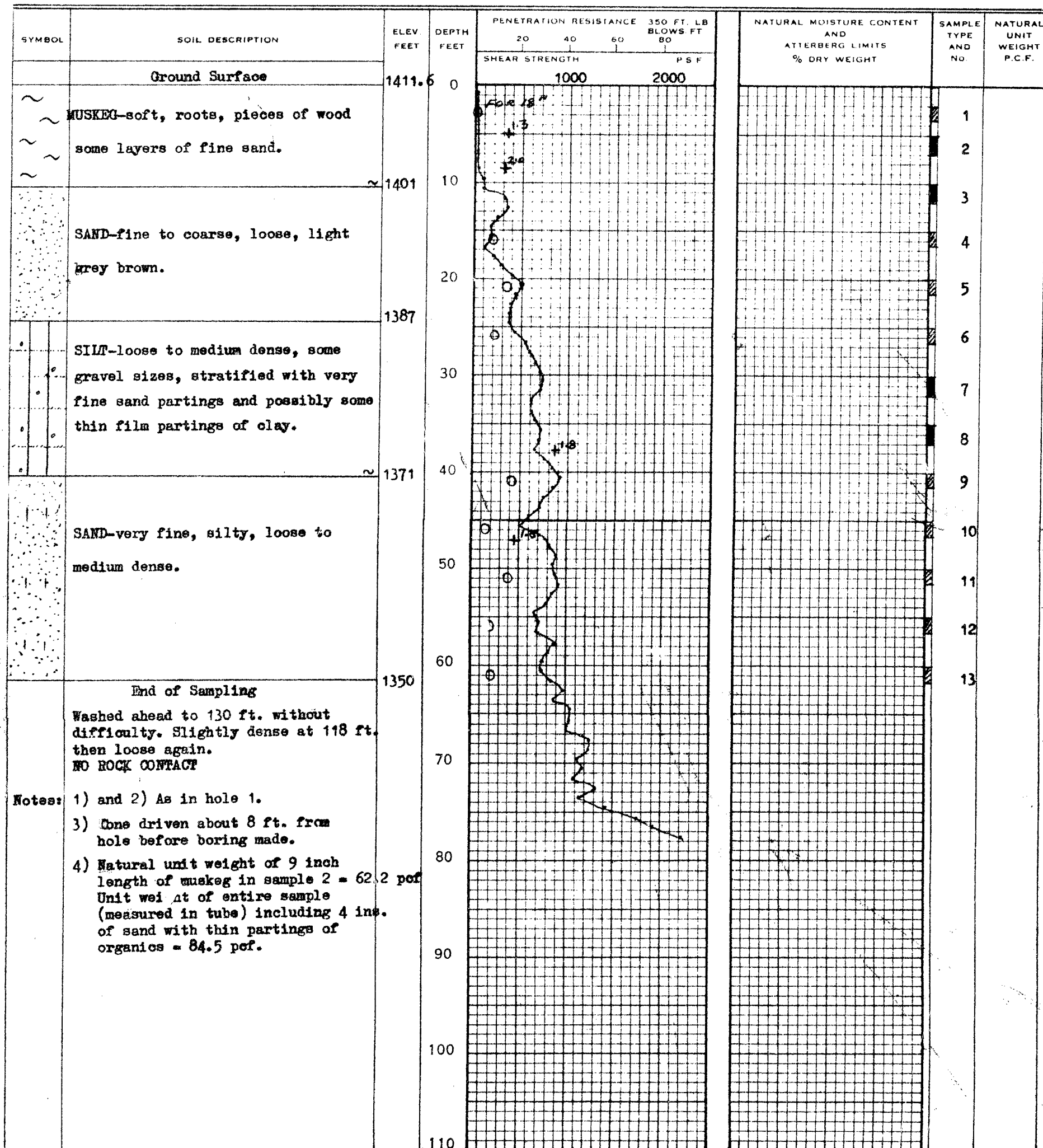
3" O.D. SHELBY TUBE



BOREHOLE NO. 2  
PROJECT Giles Creek Crossing, W.P. 70-62, Hwy. 101  
LOCATION About 1 mile south of Prairie Bee Lake  
HOLE LOCATION Sta 1451+38.5 - 21 ft. left  
HOLE ELEVATION 1411.6 ft.  
DATUM As hole 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 15, 18

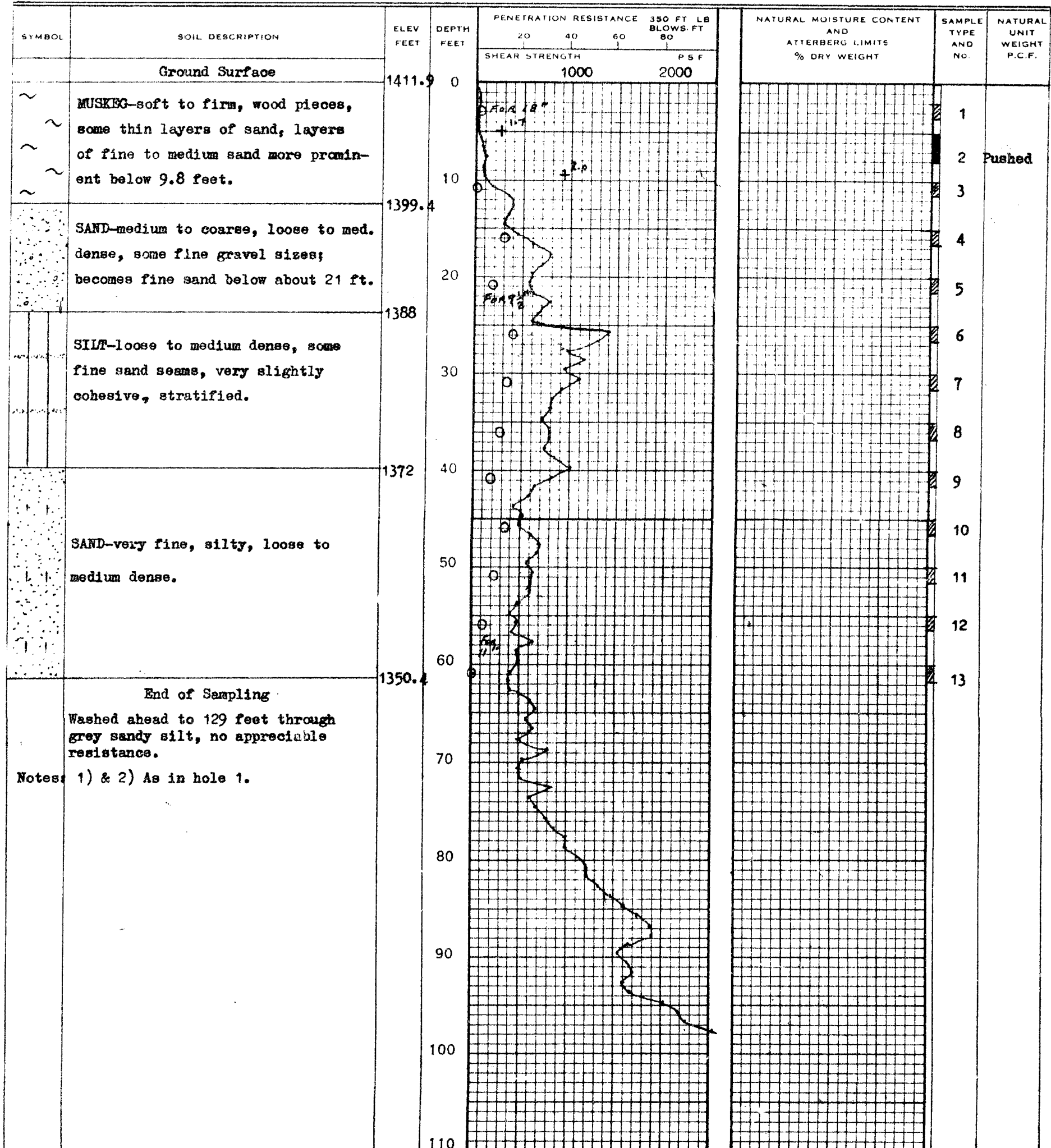
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX LI  
X  
ATTERBERG LIMITS  
LIQUID LIMIT —○—  
PLASTIC LIMIT —|—  
SAMPLE TYPE  
2" O.D. SPLIT TUBE —■—  
2" I.D. SHELBY TUBE —■—  
3" O.D. SHELBY TUBE —■—



BOREHOLE NO. 3  
 PROJECT Giles Creek Crossing, W.P. 70-62, Hwy. 101  
 LOCATION About 1 mile south of Prairie Bee Lake  
 HOLE LOCATION Sta 1450-43.5, 17 ft. Right  
 HOLE ELEVATION 1411.89 ft.  
 DATUM Nail on 0.5 ft. Stump, 67 ft. Left Sta 1443+54  
 = 1441.74 ft.

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 ———





## LEGEND

BOREHOLE NO. **4**  
PROJECT **Giles Creek Crossing, W.P. 70-62, Hwy. 101**  
LOCATION **About 1 mile south of Prairie Bee Lake**  
HOLE LOCATION **Sta 1450 + 57, 24 ft. Left**  
HOLE ELEVATION **1411.6 ft.**  
DATUM **As hole 3**

## 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

## ATTERBERG LIMITS

LIQUID LIMIT —○—

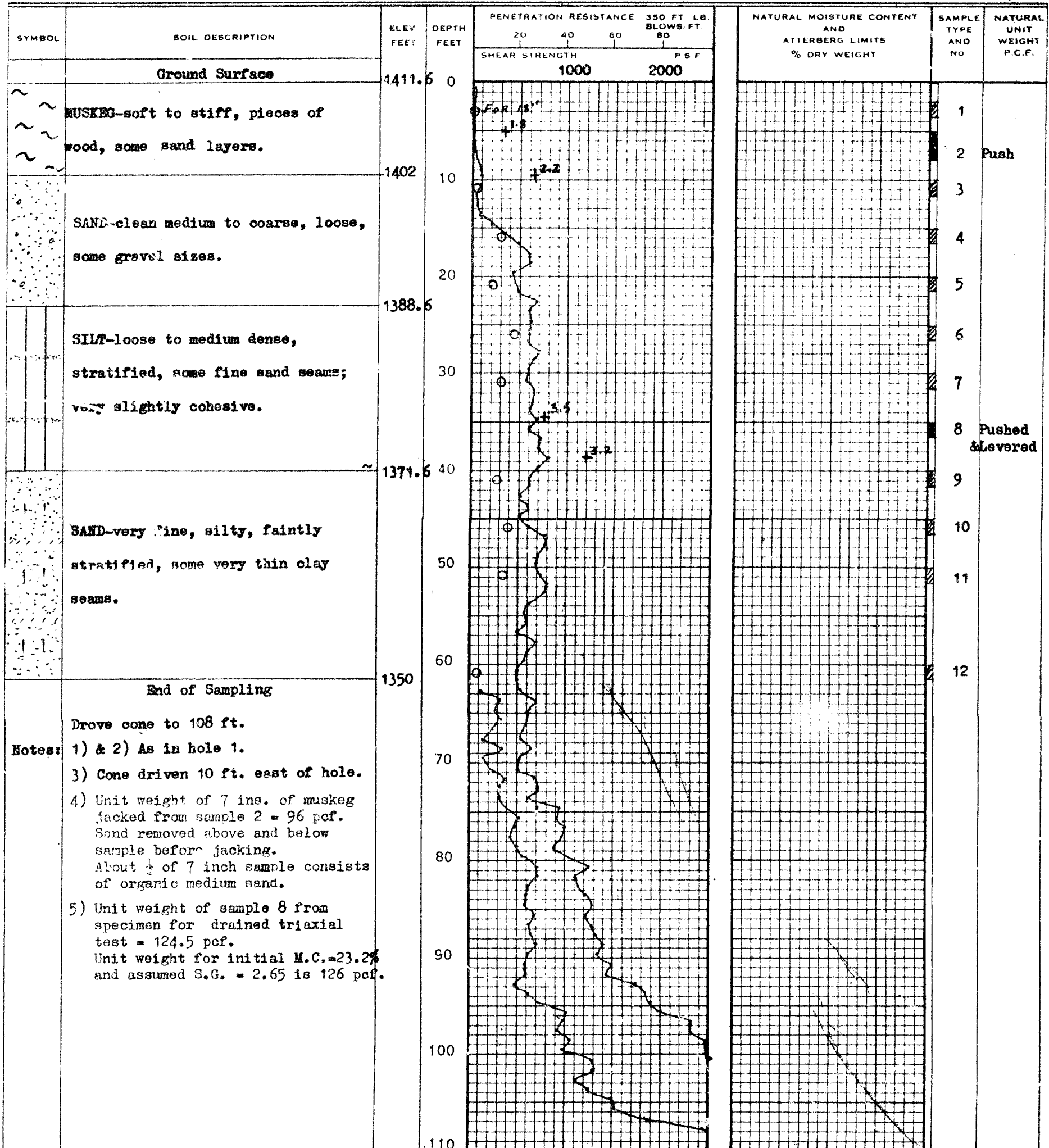
PLASTIC LIMIT ———

## SAMPLE TYPE

2" O.D. SPLIT TUBE —■—

2" I.D. SHELBY TUBE —■—

3" O.D. SHELBY TUBE —■—





## PENETRATION RESISTANCE

2 O.D. 3/4" TUBE      ○ — ○ — ○ —  
2 I.D. 3/4" BY TUBE    \* — \* — \* — \* —  
2 DIA. 1" LINE        —————

### SHEAR STRENGTH

UNDRAINED TRIAXIAL  $\oplus$   
AT OVERBURDEN PRESSURE  
UNCONFINED COMPRESSION  $\otimes$   
VANE TEST AND SENSITIVITY IS  $\dagger$

### NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

 $X^{Li}$ 

ATTERBERG LIMITS

LIQUID LIMIT —●—  
PLASTIC LIMIT —+—

SAMPLE TYPE

2" O D SPLIT TUBE \_\_\_\_\_  
2 1 D SHELBY TUBE \_\_\_\_\_  
3 O D SHELBY TUBE \_\_\_\_\_

BOREHOLE NO. 5  
PROJECT Giles Creek Crossing, W.P. 70-62, Hwy. 101  
LOCATION About 1 mile south of Prairie Bee Lake  
HOLE LOCATION 1448+64, 1 ft. Left  
HOLE ELEVATION 1411.4 ft.  
DATUM As hole 3.

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE				NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.	
				20	40	60	350 FT. LB BLOWS FT 80				
				SHEAR STRENGTH PSF							
	Ground Surface	1411.4	0	1000 2000							
~	MUSKEG-soft to firm, pieces of wood; some fine to medium sand below 13 ft.			FOR 18"					1		
~				+ 2					2	Pushed	
~			10	+ 2.5					3	Pushed	
~									4		
~	SAND-fine to medium, loose, stratified.	1395.4							5		
		1388.4	20						6		
	SILT-fine sandy, loose to medium dense, stratified.		30						7		
			40	12" UNDER ROD WEIGHT 2 BLOWS FOR 6"					9		
			50	12" UNDER ROD WEIGHT 3 BLOWS FOR 6"					10		
				12" UNDER ROD WEIGHT & HAMMER 4 BLOWS FOR 6"					12		
			~ 1351	60	12" UNDER ROD WEIGHT 4 BLOWS FOR 6"					13	
		SAND-loose, very fine.		70						14	
			1340							15	
		End of Sampling			CONE 9' DE BH.						
		Cone driven to 110 ft. from bottom of hole.		80							
				90							
			100	CONE IN BH.							
				FOR 9"							
			110								

Notes: 1) & 2) As hole 1.  
3) Cone driven 9 ft. east of hole.

## LEGEND

BOREHOLE No. 6  
 PROJECT Giles Creek Crossing, W.P. 70-62, Hwy. 101  
 LOCATION About 1 mile south of Prairie Bee Lake  
 HOLE LOCATION Sta 1446+14 - 2 feet Right  
 HOLE ELEVATION 1412.2 ft.  
 DATUM As hole 3.

## 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  $15_1 +^s$

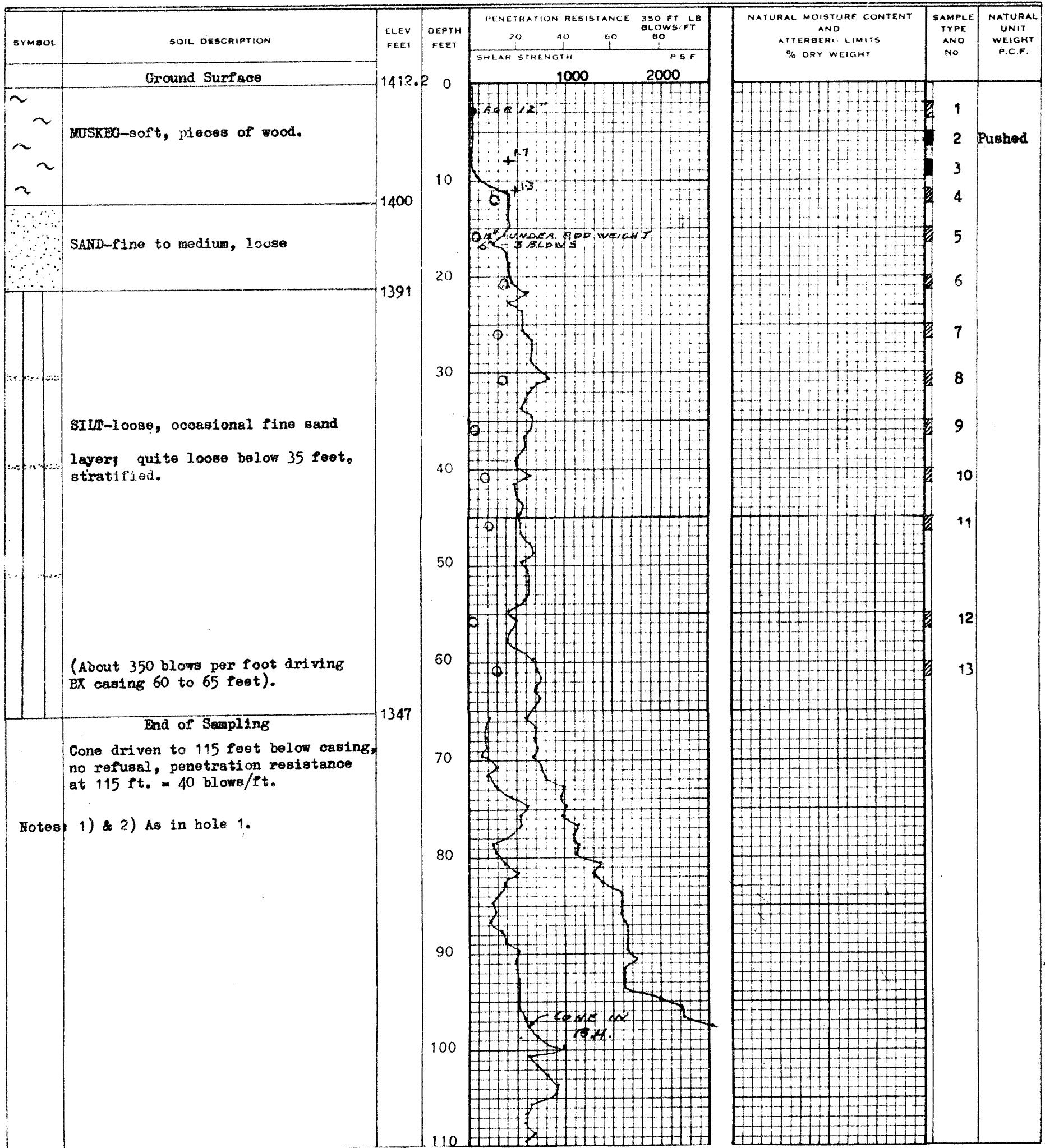
## NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

## ATTERBERG LIMITS

LIQUID LIMIT —○—  
 PLASTIC LIMIT ————

## SAMPLE TYPE

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



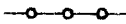
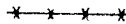

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
DRAWING No. 8  
PROJECT No. J863

## 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

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. 7  
PROJECT Giles Creek Crossing, W.P. 70-62, Hwy. 101  
LOCATION About 1 mile south of Prairie Bee Lake  
HOLE LOCATION Sta 1454+65.5, - 2 ft. Right  
HOLE ELEVATION 1411.3 ft.  
DATUM As hole 1.


SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE		350 FT. LB BLOWS/FT 80	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
				20	40				
	Ground Surface	1411.3	0	SHEAR STRENGTH		P S F			
				1000		2000			
~	MUSKEG-soft, wood pieces.	1402.8		2.4				1	
~		1401.1	10	3.6				2	
	SAND-fine to medium.			REFUSAL. SPAN = 10.3"			10" REFUSAL 10" 10"		
	BEDROCK-granite								
	100% recovery AX core.								
	End of Hole	1391.1	20						
Notes: 1) & 2) As in hole 1.									
3) Refusal to cone at 10.75 ft., refusal to a rod driven 24 ft. to west was at 6 feet. Cone driven 10 feet east of hole on centre line.									
			30						
			40						


### LEGEND

BOREHOLE NO. 8  
PROJECT Giles Creek Crossing, W.P. 70-62, Hwy. 101  
LOCATION About 1 mile south of Prairie Bee Lake  
HOLE LOCATION Sta 1452+85 - 11 ft. Right  
HOLE ELEVATION 1410.9 ft.  
DATUM As hole 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

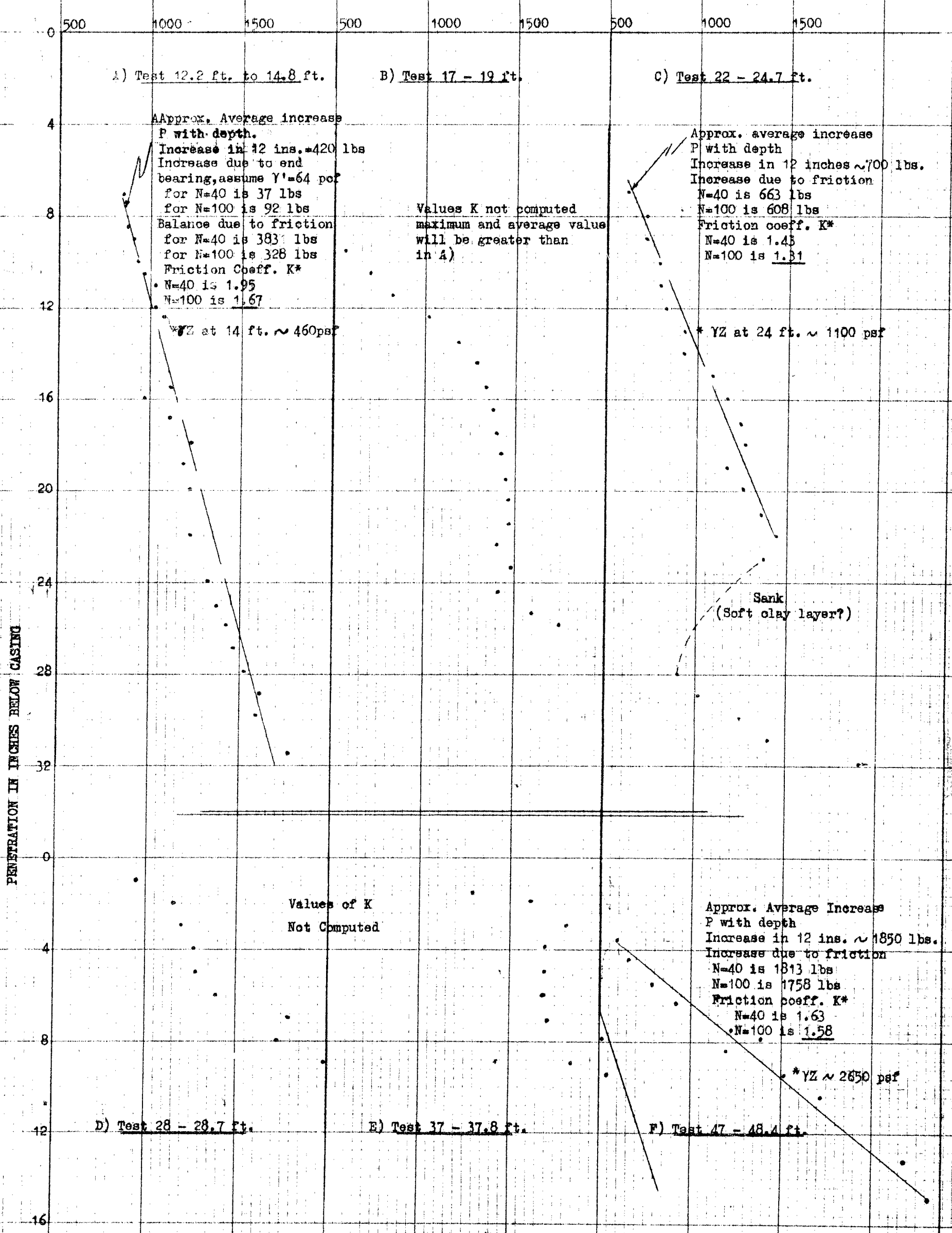
LIQUID LIMIT  
PLASTIC LIMIT

SAMPLE TYPE

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

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE		350 FT LB BLOWS FT 80	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
				20	40				
	Ground Surface	1410.9	0	1000		2000			
~ ~ ~ ~	MUSKEG--firm, pieces of wood and grass.			18"				1	
				27"				2	Levered
		1401	10	27"				3	
	SAND--medium to coarse, fine gravel sizes, loose.							4	
		1393	20					5	Levered
	SILT--loose to medium dense, some fine gravel sizes, stratified			29"				No sample	Levered
			30						Levered
		1376	40						
	SAND--very fine, silty, loose, some very thin clay seams.								
		1363.9	50	REAR END OF 11"					
	BEDROCK			REAR END OF 11"					
	granite - 88 % recovery AX core hit 1 inch layer of sand 51.4 feet. Drill on pressure above and below	1359.2	50						
Notes:	1) & 2) As in hole 1.		60						
	3) Cone driven 10 ft. west of hole.								
	4) Natural unit weight of silt measured in Shelby tube, sample 5 = 128 pcf.		70						
Check	Moisture Content of silt = 22.8%		80						
	Specific Gravity = 2.65								
	Saturated Unit Weight = 126 pcf		90						
			100						
			110						

Total Load, P, on 'A' Drill Rod In Lbs.



A) Test 12.2 ft. to 14.8 ft.

B) Test 17 - 19 ft.

C) Test 22 - 24.7 ft.

Approx. Average increase  
P with depth.

Increase in 12 ins. ~420 lbs

Increase due to end  
bearing, assume  $\gamma' = 64$  pcf

for  $N=40$  is 37 lbs

for  $N=100$  is 92 lbs

Balance due to friction

for  $N=40$  is 383 lbs

for  $N=100$  is 328 lbs

Friction coeff.  $K^*$

$N=40$  is 1.95

$N=100$  is 1.67

\*YZ at 14 ft. ~ 460 psf

Values K not computed  
maximum and average value  
will be greater than  
in A)

Approx. average increase  
P with depth

Increase in 12 inches ~700 lbs.

Increase due to friction

$N=40$  is 663 lbs

$N=100$  is 608 lbs

Friction coeff.  $K^*$

$N=40$  is 1.43

$N=100$  is 1.31

\*YZ at 24 ft. ~ 1100 psf

Sank  
(Soft clay layer?)

Values of K  
Not Computed

Approx. Average Increase  
P with depth

Increase in 12 ins. ~ 1850 lbs.

Increase due to friction

$N=40$  is 1813 lbs

$N=100$  is 1758 lbs

Friction coeff.  $K^*$

$N=40$  is 1.63

$N=100$  is 1.58

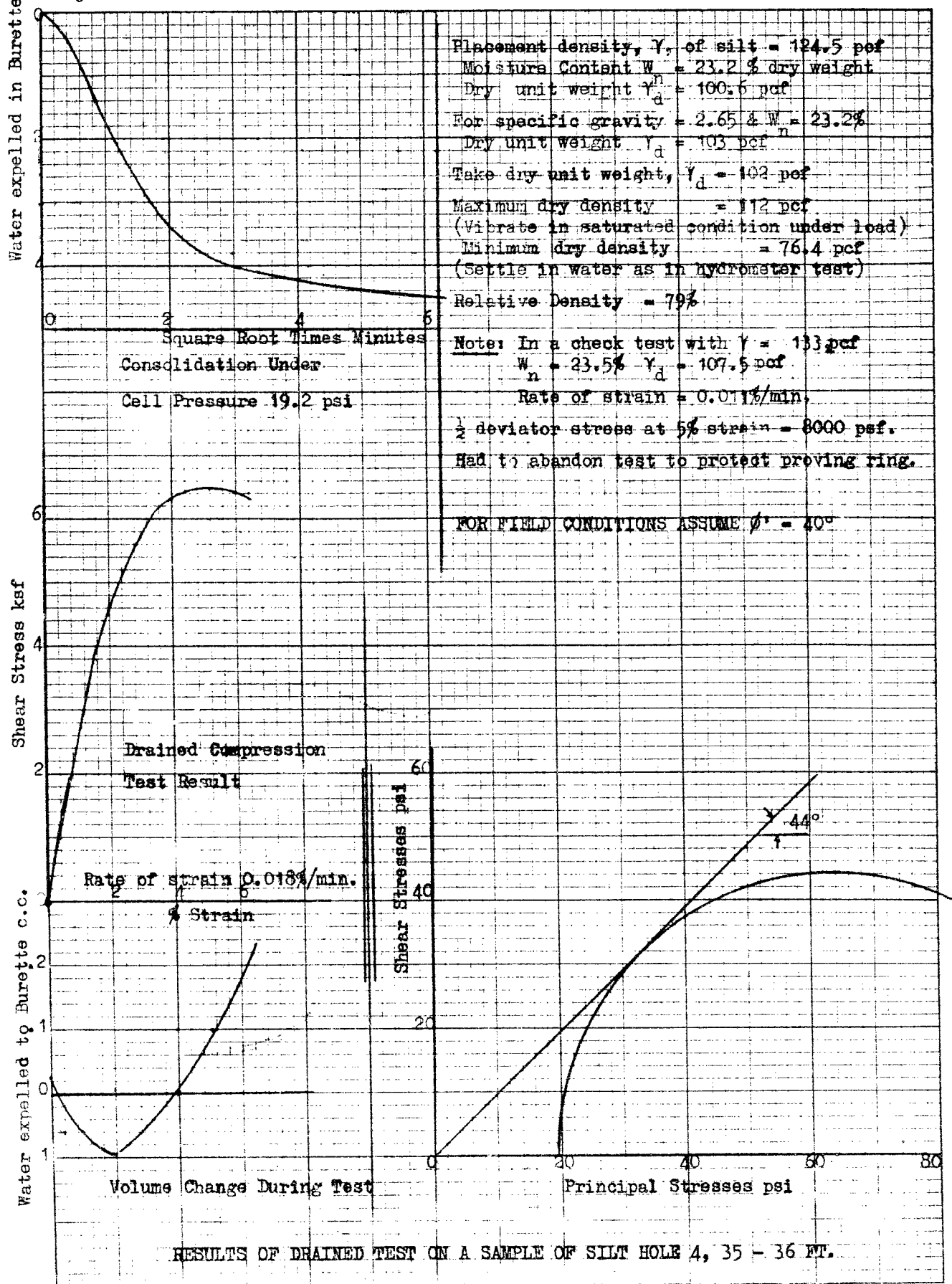
\*YZ ~ 2650 psf

D) Test 28 - 28.7 ft.

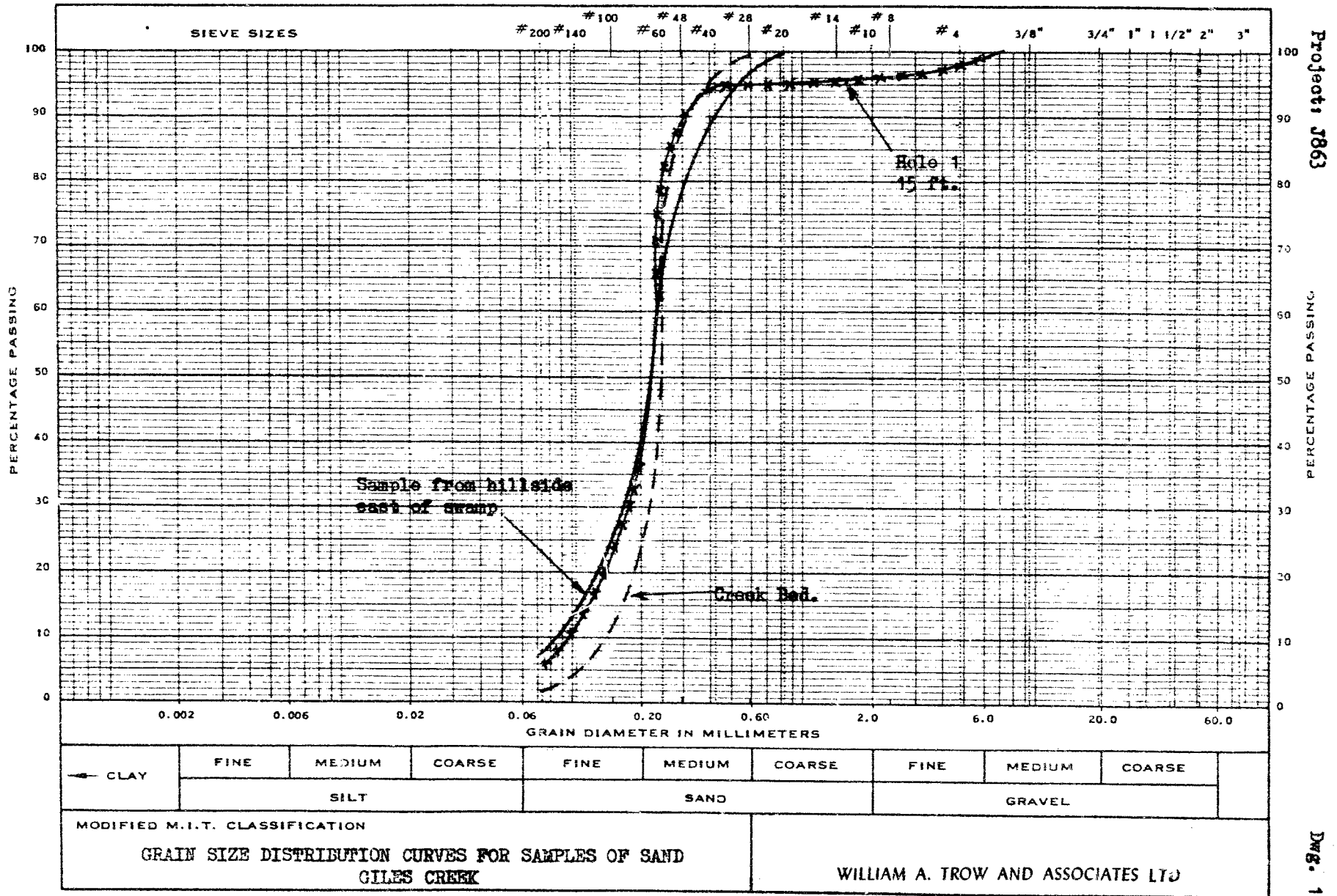
E) Test 37 - 37.8 ft.

F) Test 47 - 48.4 ft.

SUMMARY OF STATIC PENETRATION TESTS  
USING A ROD IN SPECIAL BORING 10 FEET SOUTH OF HOLE 1



# MECHANICAL ANALYSIS





Project: J863

