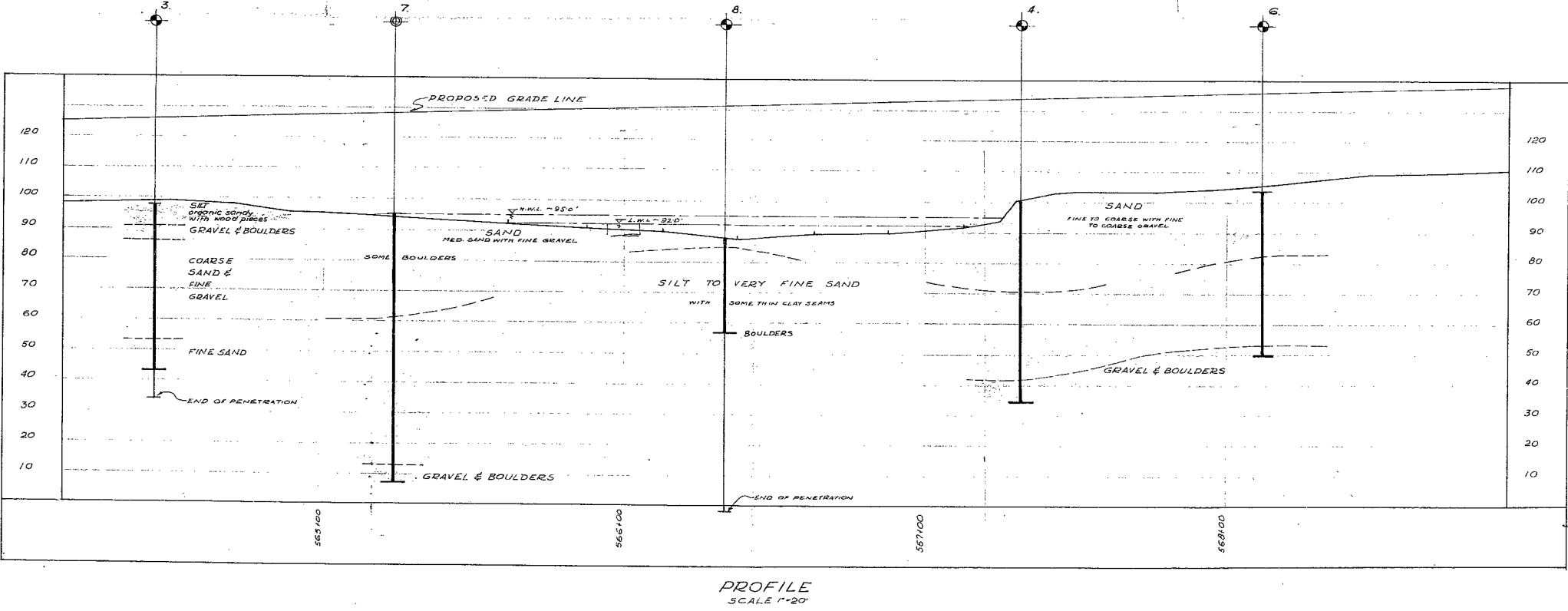
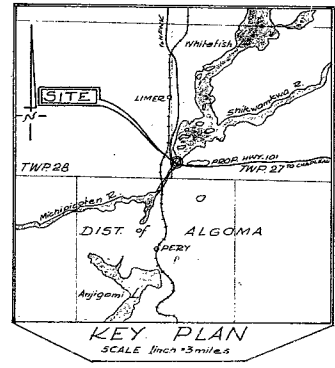
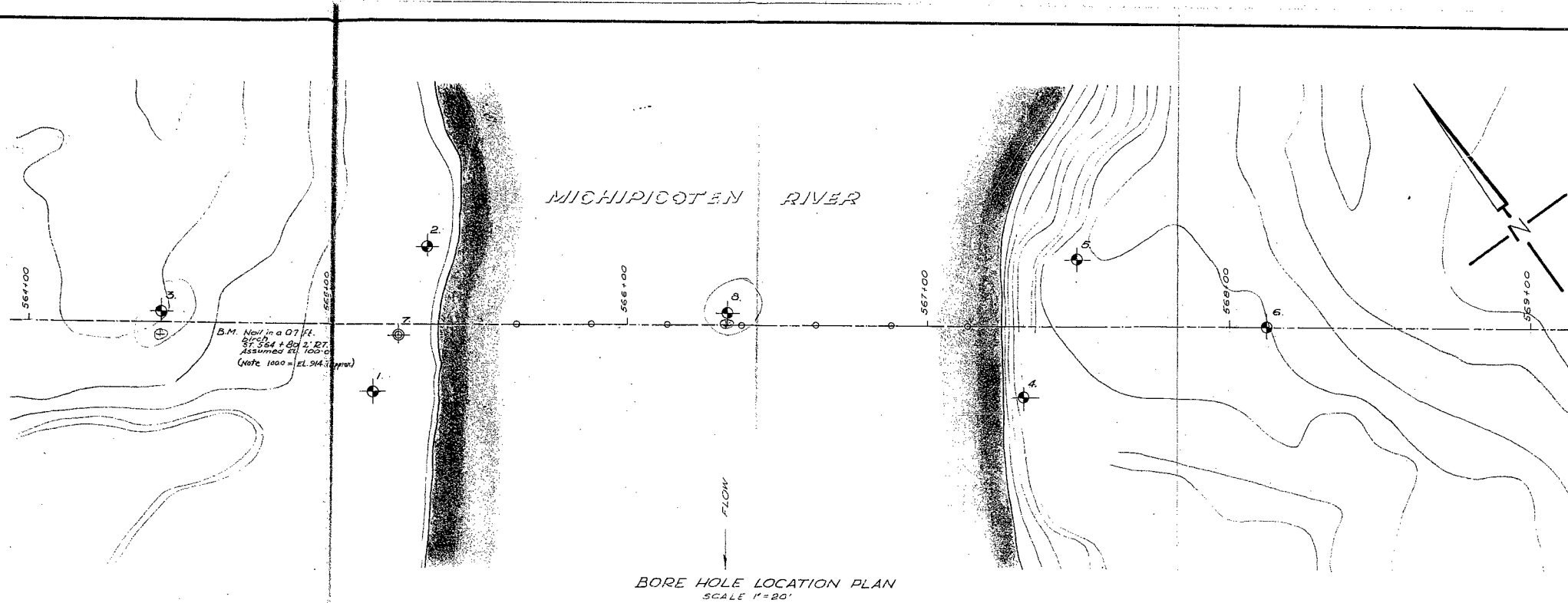


#62-F-239C

W.P. #80-62

HWY #101

MICHIPICOTEN R.



LEGEND			
	BORE HOLE & PENETRATION		
	WASH BORING		
	SOUNDINGS		
HOLE	ELEVATION	STATION	DIST. FROM E
1	94.1	565+17	22.5' RT.
2	96.3	565+33	24.5' LT.
3	97.5	564+44	3.0' RT.
4	100.1	567+31	24.0' RT.
5	101.9	567+48.5	21.5' LT.
6	103.3	568+12	£
7	95.7	565+25	4.5' RT.
8	87.4	566+33	£

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

**MICHIPICOTEN RIVER AND
PROPOSED HWY. 101. CROSSING**

W.P. NO. 80-62	PROJECT NO. 1987	DATE JULY, 1962	DWG. NO. 1
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23-64-177

**SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION**

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

July 24, 1962

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

Re: Foundation Conditions
Hwy. 101 Crossing Michipocoten River
W.P. 80 - 62

Enclosed herewith is our report on the foundation conditions underlying this bridge site.

As was the case in the other adjacent investigation along Highway 101, the subsoil consists almost entirely of silt and fine sand, although heavy stones or boulders were encountered at upper levels under the north bank.

The recommended supporting medium for piers and abutments is friction piles. The estimated safe capacity of a Class B timber pile driven to 30 feet below the stream bed is 20 tons; the safe load for a cylindrical pile 12 inches in diameter is 50 tons at 40 feet below the surface. Refusal to driving will not necessarily be experienced at these levels.

A stratum of boulders and large stones underlies the sand and silt at depths below the river surface ranging from 54 feet approximately under the south bank and 83 feet under the north bank. This stratum is dense enough to stop large displacement piles near its upper boundary. Although the vertical extent of this deposit was not confirmed, it is probable that H piles also would encounter refusal after shallow penetration into it.

The embankment approaches to this bridge will be quite stable during and following construction. Settlement will be almost immediate with load application.

If rip-rap is used for erosion protection, stones at least 5 inches thick should be used. The stones strewn in the river bed are about 6 to 9 inches thick.

We shall be pleased to discuss the findings of this report with you after you have had an opportunity to review the enclosed contents.

Yours very truly,

W A Trow

William A. Trow, P.Eng.

WAT/gc
Encls.

WILLIAM A. TROW AND ASSOCIATES LTD.

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

FOUNDATION CONDITIONS
HWY. 101 CROSSING MICHIPOCOTEN RIVER
W.P. 80 - 62

Project: J887

William A. Trow and Associates Ltd.

July, 1962

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FOUNDATION CONDITIONS
HWY. 101 CROSSING MICHIPCOTEN RIVER
W.P. 80 - 62

Project and Site

This investigation was performed at the site of the proposed Highway 101 crossing of the Michipocoten River at the location indicated on Dwg. 1.

The direction of the highway changes from west to north in the general vicinity of this bridge. It leads to Hawk Junction, about 10 miles to the north, and Chapleau is located about 50 air miles to the east.

No specific design plans have been drawn up for this river crossing, although a multi-span structure probably will be required, since the river, at this narrow crossing point, is about 185 feet wide. Earth embankments, reaching a height of about 30 feet, will form the approaches to the bridge.

The Michipocoten River in this vicinity has a very variable stream velocity. A good deal of the time the water level is low and the flow is quite slow. However, at irregular intervals, approximately once per day, water is released from the dam about $1\frac{1}{2}$ miles upstream. The water rises quickly about $1\frac{1}{2}$ feet when this occurs and the rate of flow increases to approximately 5 f.p.s. Because of this condition, drilling work on the river was a very treacherous undertaking. This project was terminated prematurely when the drilling raft was swept from its moorings during one of these flash floodings.

According to soundings taken in the river along centre line, the water reaches a maximum depth of approximately 8 feet at flood time. The deep water lies in the centre and toward the south bank. The north side of the river is shallower.

The north bank of the river is low and flat, and, at one time, before the dam was built, it probably formed the flood plain of the river. About 180 feet beyond the north shore, the ground rises in the form of steep banks reaching about 25 feet above this flood plain. The south shore rises about 8 to 10 feet above the river and therefore it is above the flood plain. Farther upstream and just beyond the south shore, hills of bedrock project above the ground surface.

Second growth timber covers both banks of the river. The river banks are grass covered, probably underlain by sand. During low water, stones 6 to 9 inches in size may be seen in the river bed. The river channel has been dredged about $\frac{1}{2}$ mile upstream. The dredged sand and gravel is piled on both shores. Several views of the site are included in the report.

Investigation of Soil Conditions

A total of 7 sampled borings were made at this site. Two borings were made adjacent to the highway centre line at each shore; another boring was made about 100 feet back from each shore in order to determine conditions under each approach embankment; the remaining and last boring was made from a raft in the centre of the river. This last hole was not completed, since the raft and casing were swept away, following the release of water from the dam. A cone penetration test was made beside each boring. This test had been completed in the river before the raft failed.

Samples were taken at approximately 5 foot intervals of depth to a maximum of about 60 feet. These were recovered in the disturbed state using the conventional 2 inch O.D. split spoon. In many instances there was no recovery of the sample by this method and the soil was eventually recovered using a side sampler.

The depth of 60 feet was the lower limit of sampling, since it was very difficult to drive the casing deeper or to withdraw it even from this level. In order to determine the conditions existing below 60 feet, the boring was either continued by washing ahead or by driving a cone.

Holes 4 and 5 were terminated in dense boulders. Attempts were made to continue the borings through the boulders, but, after coring 5 feet into them, work was discontinued. There was a tendency for the bottom lengths of the casing to become unscrewed during this operation.

Because boulders were encountered at lower levels in holes 4, 5 and 6 under the south bank, another attempt was made to locate this refusal stratum under the north shore. Hole 7, an unsampled hole, made midway between holes 1 and 2, was put down for this purpose. The boulder deposit was encountered about 83 feet below the river bank.

In order to obtain some indication of the resistance that would be developed around a pile, static penetration tests were made in 2 borings. This test was performed at 5 foot intervals of depth. In each test, a $\frac{1}{2}$ inch diameter steel rod was forced into the soil below the bottom of the casing. The resistance for each inch of penetration was recorded. When the maximum loading limit of the test arrangement was reached, the force required to withdraw the rod also was measured. The rod, then, was withdrawn, and the same test was repeated at the same levels using an A drill rod with plugged end.

The tests on the $\frac{1}{2}$ inch rod were discontinued when it broke during withdrawal from the sand. The results of this testing program are indicated on Dwg. 10.

The soil types encountered in each boring are shown on the logs, Dvgs. 2 to 9 of this report. They are also presented in summarized form in the stratigraphical profiles of Dwg. 1. Since this information is quite self-explanatory, no purpose is served by repeating this information in this report.

It is seen that the soil is described, generally, as a silty to very fine sand with some thin layers of clay. On the basis of penetration resistance measurements, these materials appear to be in a denser condition than was noted at other bridge sites for Highway 101 farther to the east. Considerable amounts of heavy gravel and large stones were encountered at upper levels under the north shore, down to a depth of about 20 feet. In the early stages of the work, it was felt that these large stones would prevent the penetration of displacement piles. However, since it was possible to drive a 2 inch cone through these deposits, and since the stones were not so evident in the river or under the south bank, it was concluded that displacement piles would penetrate this material.

Although bedrock was not reached, holes both under the north and south banks penetrated into a deposit of dense stones and boulders. These boulders lie somewhat closer to the surface under the south bank. They were not encountered until a depth of 83 feet was reached at the north bank location.

Foundations

In view of the variations in penetration resistance or N values recorded at upper levels in this sand subsoil, and the difficulties associated with the installation of footings below river bed level in a fine-grained soil, the use of a pile foundation would seem to be the only reasonable means of support for this bridge.

On the basis of field measurements, both for this project and for other sites to the east, it is concluded that cylindrical friction piles will be the most suitable. However, end-bearing displacement piles, driven to refusal in the deposit of dense boulders located 60 feet below the surface under the south bank and 83 feet below the surface under the north shore, also could be considered. Because of limitations to the drilling equipment, it was not possible to define the depth of this boulder deposit, and therefore the refusal depths for an H pile are not known. It is considered, however, that a displacement pile would definitely encounter refusal near the top surface of the boulder deposit. The permissible loading for a pile driven to refusal in this deposit should be equal to its safe structural capacity as a short column.

In recent previous reports for other bridge sites in the area, it was concluded that cylindrical piles, either of timber or steel, would develop sufficient capacity through friction, at relatively shallow depths to warrant their use as an economic foundation medium. The

estimated safe capacities of timber and steel piles were given, based both on theoretical calculations and on engineering judgments of the magnitude of skin friction that would be developed.

On the basis of recorded full-scale pull-out tests on Raymond step-tapered piles and on static penetration test measurements at the site, it was concluded that large displacement piles develop considerable friction. For purposes of estimating pile capacity, a skin friction coefficient $K = 1$ was used. This value was believed to be conservative.

The results of these calculations and estimates are repeated in Table 1. For preliminary design purposes, it was estimated that a timber pile could carry a load of 20 tons safely after it has been driven 30 feet below river bed level or below the ground surface if the latter could be protected against river erosion. Similarly, a 12 inch steel pile would develop a safe load of 50 tons at 40 feet. In general, a tapered displaced pile is to be preferred to one with straight sides and the latter type should not have a square plate welded to the bottom, since the projections of the plate beyond the cylinder will tend to reduce the friction force around the shaft.

Discussion of Soil Capacity

On the other bridge site investigations along the Highway 101 route, certain field and laboratory tests were performed in order to support the opinion that the soil was denser than penetration tests indicated. A similar program was carried out on this soil.

In the field, static penetration tests were made and the increase in resistance with depth was measured. Because the soil was generally denser than at other bridge sites closer to Chapleau, it was necessary to use a small diameter rod in these tests. A larger rod, 1-5/8 inch in diameter, also was used, but it could not be pushed into the soil very far.

The results of these tests are summarised in graphical form on Dwg. 10. It is seen that there is a definite increase in resistance with depth even for the tests on the small $\frac{1}{2}$ inch diameter rod. A much greater increase in resistance was noted for penetration tests with the 1-5/8 inch O.D. A rod and these tests reached the installation capacity after very shallow penetration.

The increase in resistance with depth, according to theoretical calculations, is contributed by increases both in end-bearing and in shaft friction. In previous analyses, it was concluded that the major portion of the increase is due to shaft friction. In this test, however, since the sand is denser, it is probable that a significant contribution also is made by end-bearing resistance.

The relatively low resistance offered during withdrawal of the rods from the ground, could be taken as evidence that the support provided by skin friction is only a minor contribution to the overall strength increase measured in the loading tests. Although this interpretation is reasonable, it implies that very high values of the bearing capacity factors N_y and N_q were generated. It is felt, rather, that the skin friction generated during the withdrawal of the rods will be much less than the resistance offered as the rods are forced into the ground. In the latter case, the soil grains around the shaft are forced against one another to a much greater degree.

Regardless of the manner in which resistance is developed, however, the trend of increased capacity with depth shown in Dwg. 10 cannot be disputed. Taking the test results from a depth of 23 feet in hole 4 as an example, it is seen that the rate of increase is about 46 lbs. per inch of depth on the $\frac{1}{2}$ inch rod and about 450 lbs. per inch of depth on the 1-5/8 inch O.D. A rod. If these results were to be extrapolated to a 12 inch diameter pile shaft, the resistance increase would be in the order of 6.7 tons per foot and 16.7 tons per foot respectively.

The increase in resistance is not so marked with a $\frac{1}{2}$ inch O.D. rod, since less displacement of soil takes place. It is noted that the overall and unit resistance to the 1-5/8 inch A rod is much greater. It is concluded that the friction resistance generated around a large displacement pile will be greater still. The estimates of safe capacity given in Table 1, are based upon this premise.

As additional corroboration of the dense condition of the sand, certain laboratory tests also were made. These tests were designed to provide some indication of the in-place density and the relative density of the sand. The in-place density was determined indirectly from measurements of moisture content and specific gravity. Undisturbed samples could not be recovered. Using conservative values for in-place density, the relative density of the soil was computed. These computations and test results are presented in Table 2. It is noted that the relative density of the soil is in excess of 80 percent, which is indicative of a very dense condition.

As a final indication of the physical strength of the sand, an attempt was made to measure the angle of internal friction of a sample prepared at a density believed to be reasonably representative of in-situ conditions. The result of this test is indicated on Dwg. 11. The estimated angle of internal friction of the sand, from this test is seen to be about 46 degrees.

Typical grading curves for the sand and silt at this site are shown in Dwg. 12. The higher clay content, noted in the sample from 40 feet in hole 4, results from the inclusion of a thin layer of clay. These layers of clay represent a very small volume of the subsoil and therefore they will not affect its bearing qualities.

On the basis of these measurements and sample tests, it is concluded that the soil is quite competent to support the pile loads at the depths indicated in Table 1. A refusal-to-pile-driving condition is not necessarily implied in this table, although refusal may be encountered at some locations, particularly in the vicinity of hole 8 in the river, where the soil appeared to become quite dense below 30 feet.

Erosion Protection

Since the velocity of flow of the stream is quite high, the tops of the piles must be protected. This protection could either take the form of interlocking steel sheet piling driven around each pier or heavy well-graded rip-rap. A stone particle at least 5 inches thick is theoretically required for a flow of 5 f.p.s.

Embankments

Since the soil below the bridge approaches is almost entirely composed of silt and sand, there is no danger of failure as the embankment fill is placed. Settlement also should occur instantaneously with load application and it will be of a very small order.

Recommendations

1) The subsoil underlying the site of the Highway 101 crossing of this river consists generally of dense fine sand and silt, and at depths ranging from approximately 54 to 83 feet respectively below the water surface under the south and north banks a dense boulder deposit is encountered.

2) The recommended foundation medium for the proposed bridge is friction piles driven into the sand and silt. The estimated safe load for Class B timber piles driven 30 feet below the stream bed is 20 tons. The corresponding load for 12 inch diameter cylindrical steel piles driven to 40 feet is 50 tons. It may be necessary, above 20 feet, to jet the piles past boulders in the vicinity of the north shore. Refusal to pile driving may not necessarily be met at the depths indicated. If refusal is encountered, care must be taken to avoid damage to the piles by over-driving.

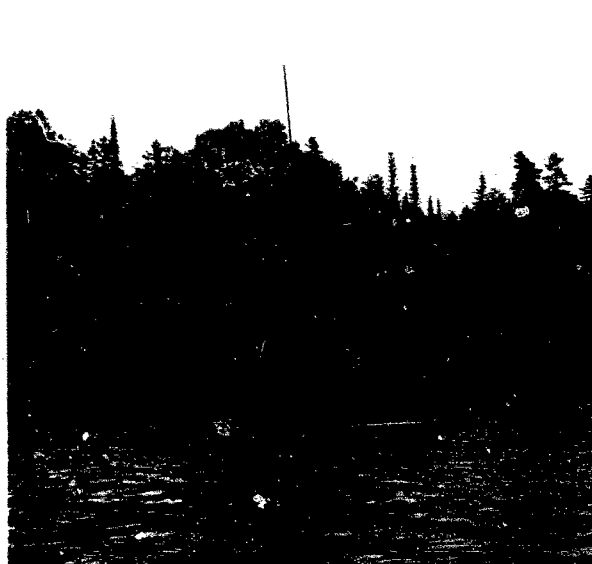
3) The alternative means of support is end bearing piles driven to refusal in the stratum of boulders found 54 to 83 feet below the river surface.

4) No embankment stability problem exists at this site. Settlements of the foundation will be complete as soon as the fill load has been applied.

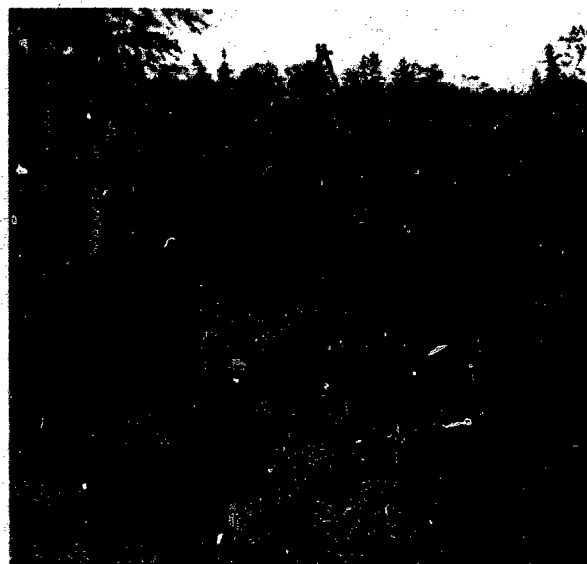
W. Trow

William A. Trow, P.Eng.

WAT/gc
J887
July, 1962



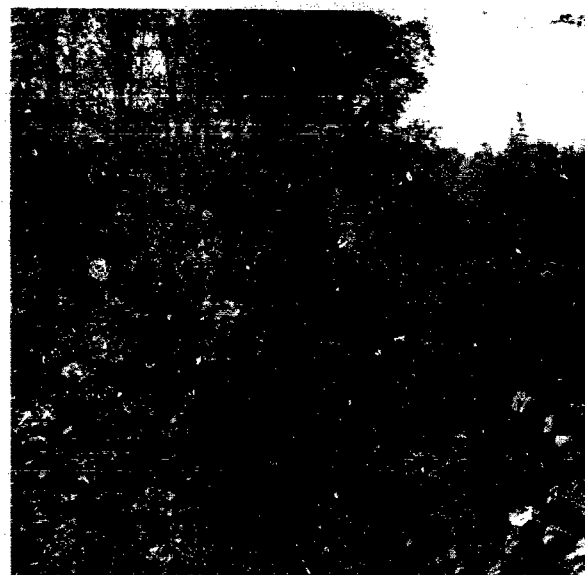
View from the North
Bank. Drill on Hole 8.



From the North
Along CL. Drill on Hole 7.



Looking Across River
From the South Bank
Along Centre Line



From the North
Looking Along the Line
Cone #3 Location



View from the North
Bank. Drill on Hole 8.



From the North
Along CL. Drill on Hole 7.



Looking Across River
From the South Bank
Along Centre Line



From the North
Looking Along the Line
Cone #3 Location



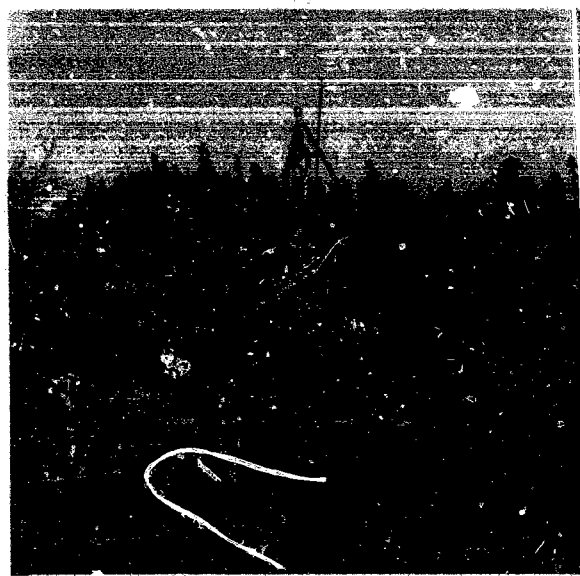
Looking Downstream
From Hole 8



Looking Upstream
From Raft on Hole 8



North Bank, West
of Line, Looking West



Hole 1 From
The West



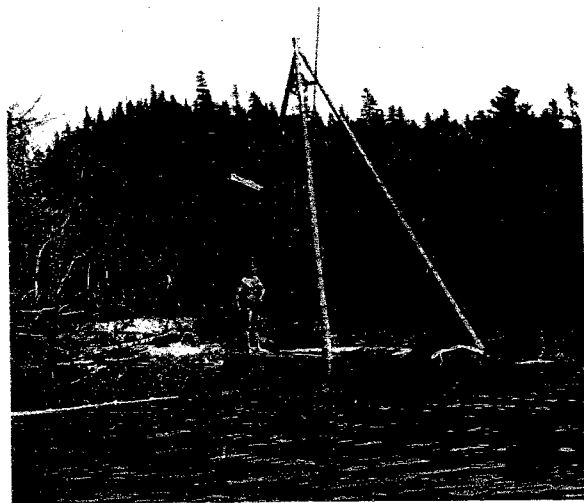
Looking Downstream
From Hole 8



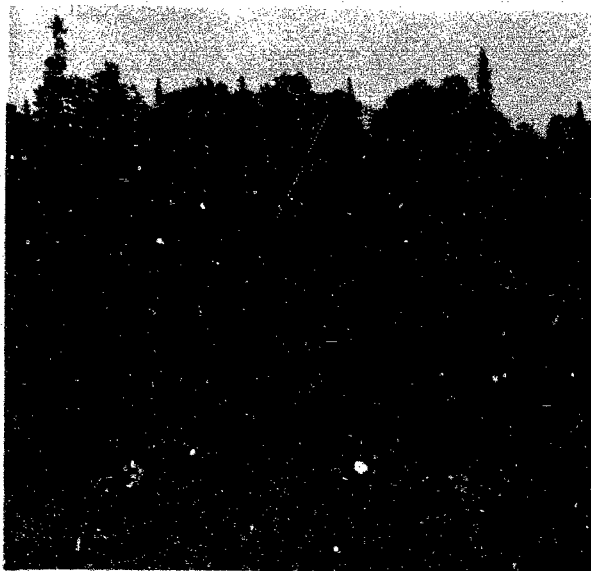
Looking Upstream
From Raft on Hole 8



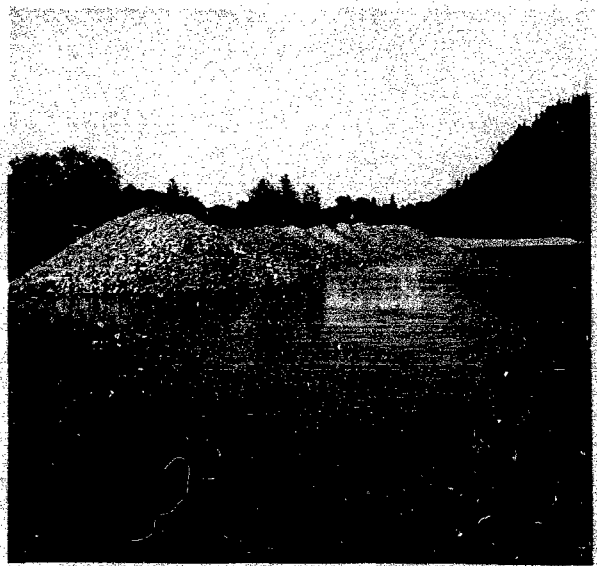
North Bank, West
of Line, Looking West



Hole 1 From
The West



View of South Bank
Along Centre Line



Gravel Dredged From
River Bed $\frac{1}{2}$ Mile Upstream



View From the West
Looking Upstream
Toward Bridge Site;
Island on the Right



View of South Bank
Along Centre Line



Gravel Dredged From
River Bed $\frac{1}{2}$ Mile Upstream



View From the West
Looking Upstream
Toward Bridge Site;
Island on the Right

TABLE NO. 1

SUMMARY OF ESTIMATED PILE CAPACITIES IN TONS⁺ - MICHIPOCOTEN RIVER CROSSING HWY. 101

Depth ** Below Stream Bed	Timber Pile*	Cylindrical Steel Pile ⁺⁺
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:

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

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

γ = 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_{γ} & 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.

SUMMARY OF LABORATORY TEST MEASUREMENTS

(a) Moisture Content Determinations

Depth (ft.)	10	15	20	25	30	35	40	45	50	55	60
<u>Hole 1</u>				18.3	16.1	18.0	21.7*	15.7		18.3	19.8 ⁺ 18.9*
<u>Hole 4</u>	19.5	19.9		18.8	18.0 ⁺	19.2	21.4*		22.4 [‡]	23.7 [‡]	
				19.9*	17.8						
<u>Hole 5</u>							19.5				
<u>Hole 6</u>			17.1	19.5							

* Test on entire bag sample including moisture adhering to walls of bag.

+ Test on sample wrapped in tinfoil and placed with rest of soil in plastic bag.

‡ Clay pockets in samples.

(b) Specific Gravity and Unit Weight Determinations

	Depth	S.G.	Moisture Content	Dry* Unit Wt.
Hole 1	40 ft.	= 2.63 (assume)	20	107.5
Hole 4	25 ft.	2.60 & 2.62 Average = 2.61	22	104.2
Hole 4	40 ft.	2.64	20	106.9
Hole 5	40 ft.	2.63	21.7	104.2

* For 100% saturation

$$\gamma_D = \frac{62.4S}{WS + 1}$$

$$\text{for Hole 4 25 ft.} = \frac{62.4 \times 2.61}{.22 \times 2.61 + 1} = 104.2$$

(c) Relative Density Determinations

Depth (ft.)	25			40		
	γ max. pcf ⁺	γ min. pcf ^x	R*%	γ max. pcf ⁺	γ min. pcf ^x	R*%
Hole 1				112.6	68.3 (69.2)	87
Hole 4	108.1 (108.2)	76.5	97	117.5 (123.8) (121.2)	66.7 (65.4)	80
Hole 5				111.0 (106.9)	77.1 (79.1)	92

$$* R = \frac{\gamma \text{ max.} (\gamma - \gamma \text{ min.})}{\gamma (\gamma \text{ max.} - \gamma \text{ min.})}$$

+ Vibrated in saturated condition under load for at least 1 hr. with drainage permitted.

^x Allowed to settle in water in accordance with procedure for hydrometer test. No dispersing agent used. Values in brackets indicate results of check values.

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) ⊕^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS

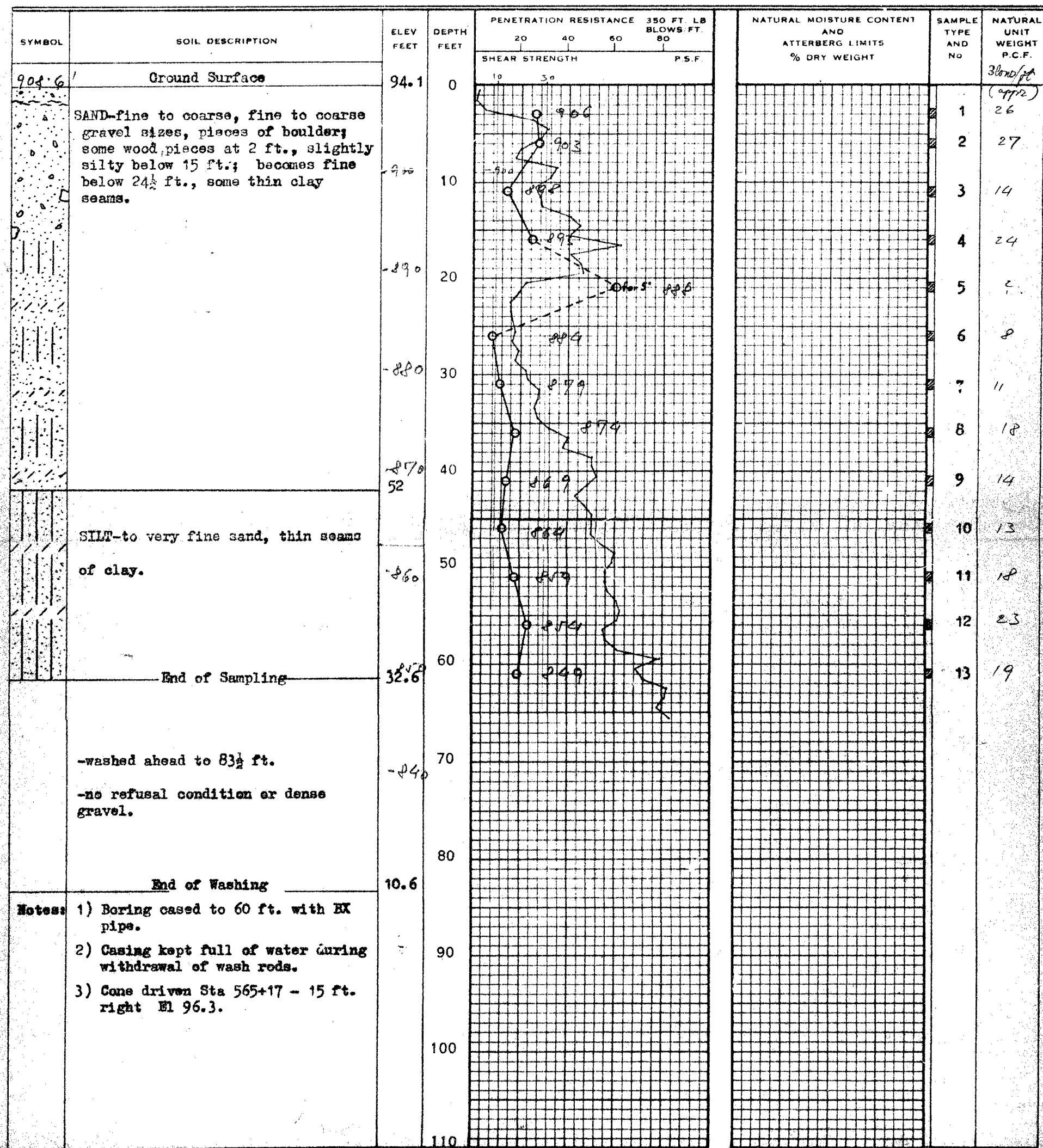
LIQUID LIMIT —○—

PLASTIC LIMIT ———

SAMPLE TYPE

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

BOREHOLE No. 1
 PROJECT Michipocoten River Crossing, W.P. 80-62
 LOCATION See Dwg. 1.
 HOLE LOCATION Sta 565+17, 22½' Right
 HOLE ELEVATION 94.1 ft.
 DATUM See Dwg. 1.



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

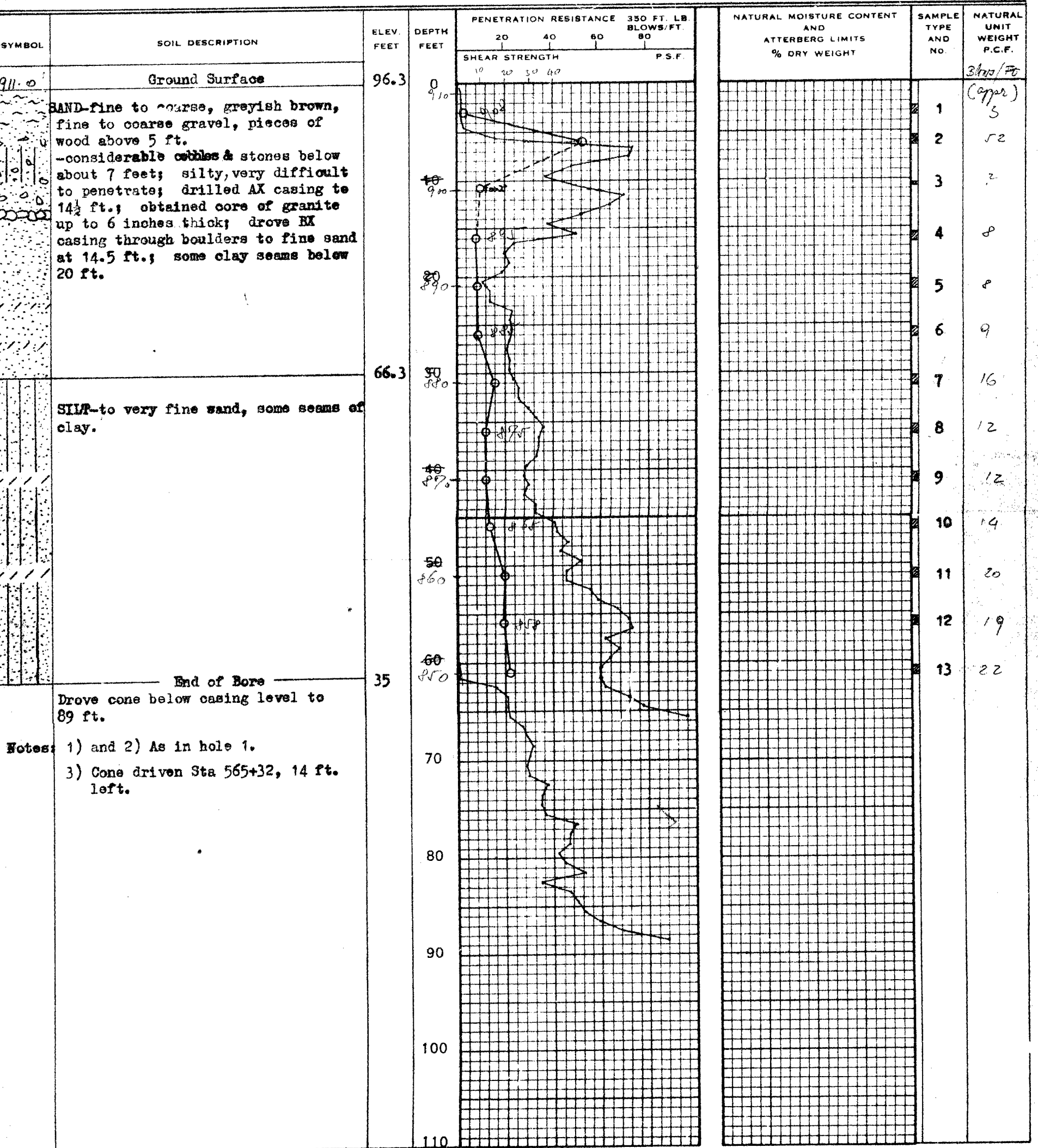
ATTERBERG LIMITS

LIQUID LIMIT
PLASTIC LIMIT

SAMPLE TYPE

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

BOREHOLE No. 2
PROJECT Michipicoten River Crossing, W.P. 80-62
LOCATION See Dwg. 1.
HOLE LOCATION Sta 565+33, 24.5 ft. Left
HOLE ELEVATION 96.3 ft.
DATUM See Dwg. 1.



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 X^{LI}

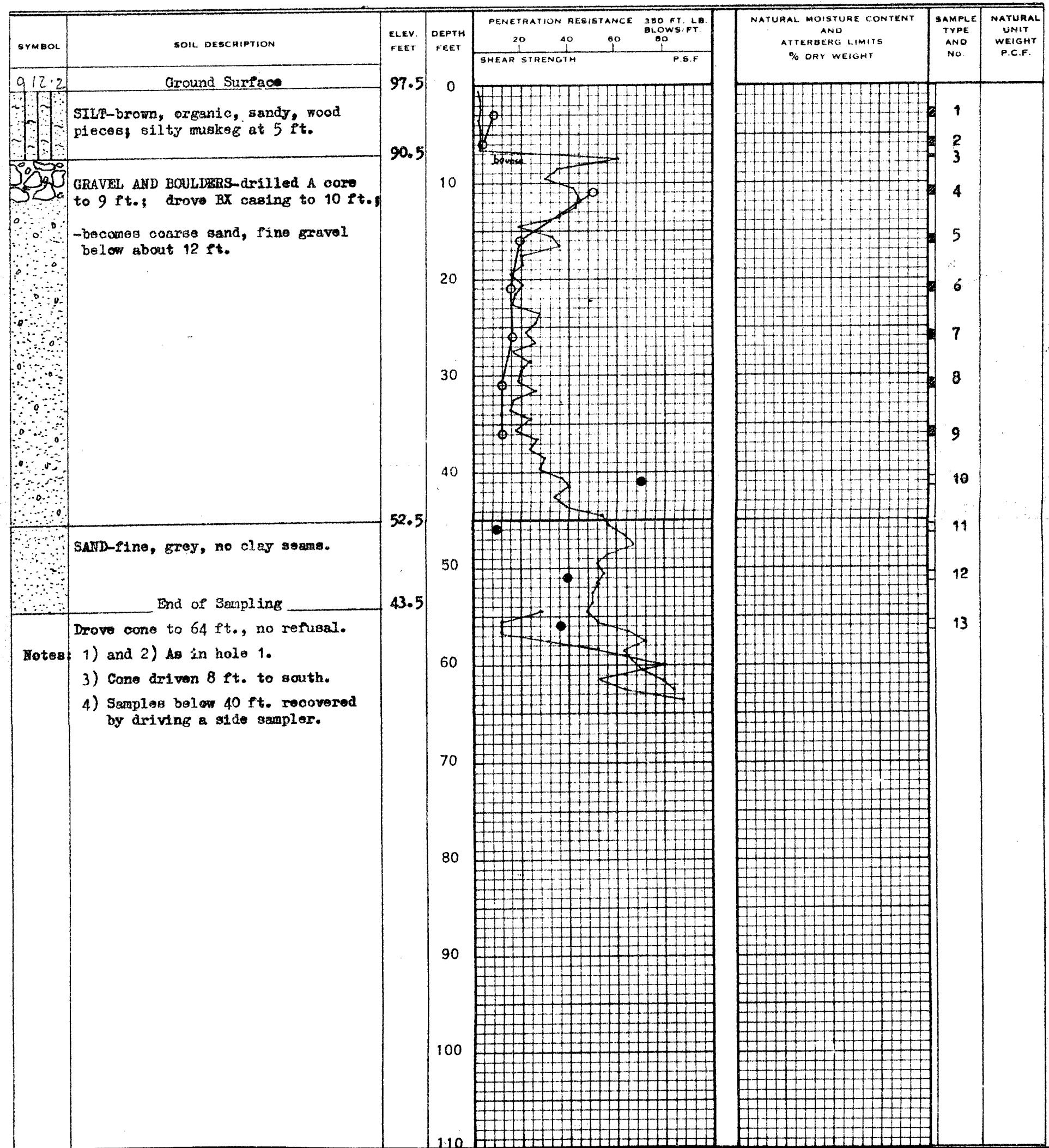
ATTERBERG LIMITS

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 Michipicooten River Crossing, W.P. 80-62
 LOCATION See Dwg. 1.
 HOLE LOCATION Sta 564+44, 3 ft. Right
 HOLE ELEVATION 97.5 ft.
 DATUM See Dwg. 1.



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) ⊕^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

X^{LI}

ATTERBERG LIMITS

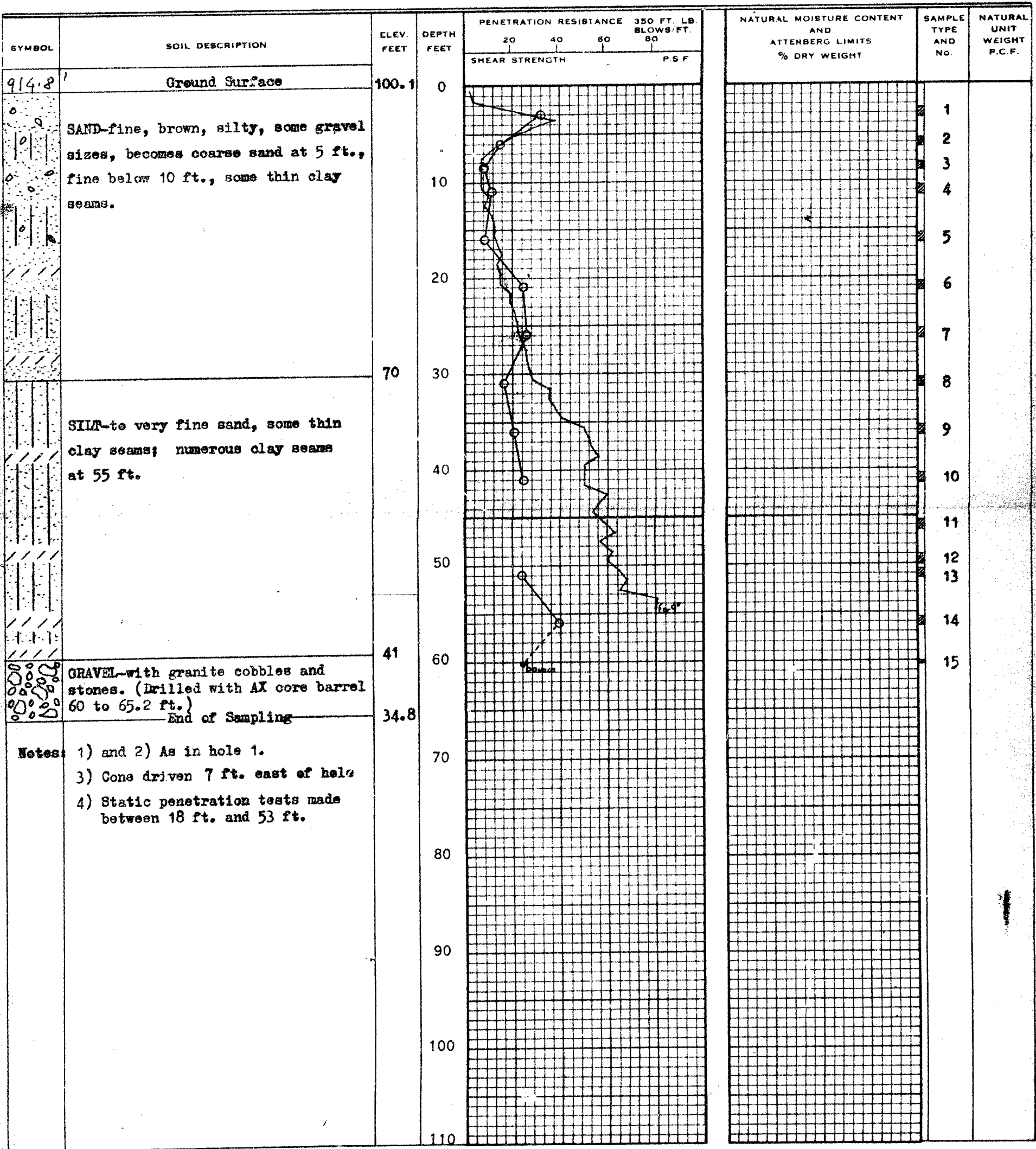
LIQUID LIMIT —○—

PLASTIC LIMIT ———

SAMPLE TYPE

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

BOREHOLE No. 4
 PROJECT Michipicoten River Crossing, W.P. 80-62
 LOCATION See Dwg. 1.
 HOLE LOCATION Sta 567+31, 24 ft. Right
 HOLE ELEVATION 100.1 ft.
 DATUM See Dwg. 1.



LEGEND

BOREHOLE NO. 5
PROJECT Michipicoten River Crossing, W.P. 80-62
LOCATION See Dwg. 1.
HOLE LOCATION Sta 567+48.5, 21½ ft. Left
HOLE ELEVATION 101.9 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 —○—

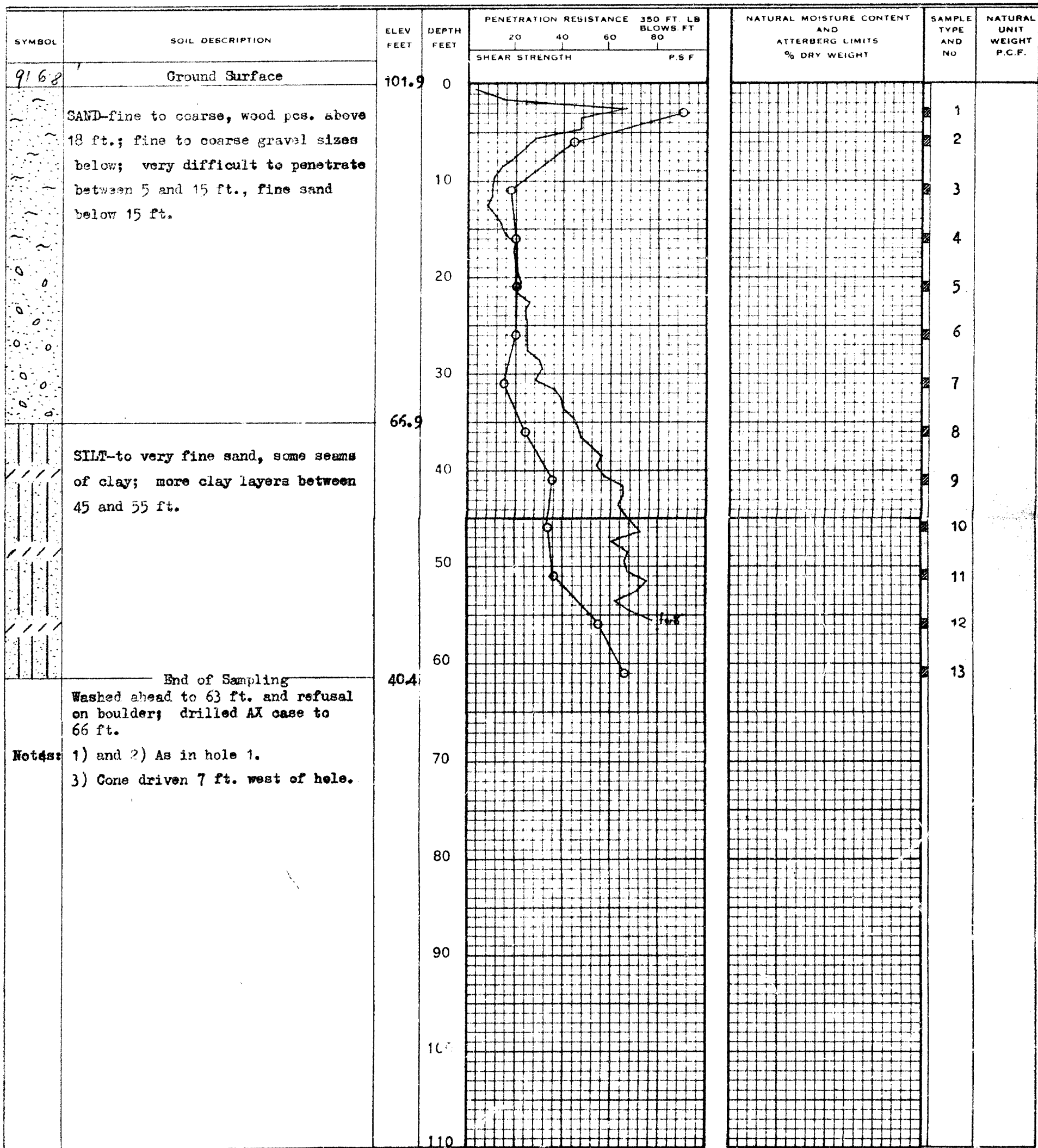
PLASTIC LIMIT —|—

SAMPLE TYPE

2" O.D. SPLIT TUBE —■—

2" I.D. SHELBY TUBE —■—

3" O.D. SHELBY TUBE —■—



BOREHOLE NO. 6
 PROJECT Michipicoten River Crossing, W.P. 80-62
 LOCATION See Dwg. 1.
 HOLE LOCATION Sta 568+12. on CL
 HOLE ELEVATION 103.3 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_v) ⊕^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTEBERG LIMITS

LIQUID LIMIT —○—

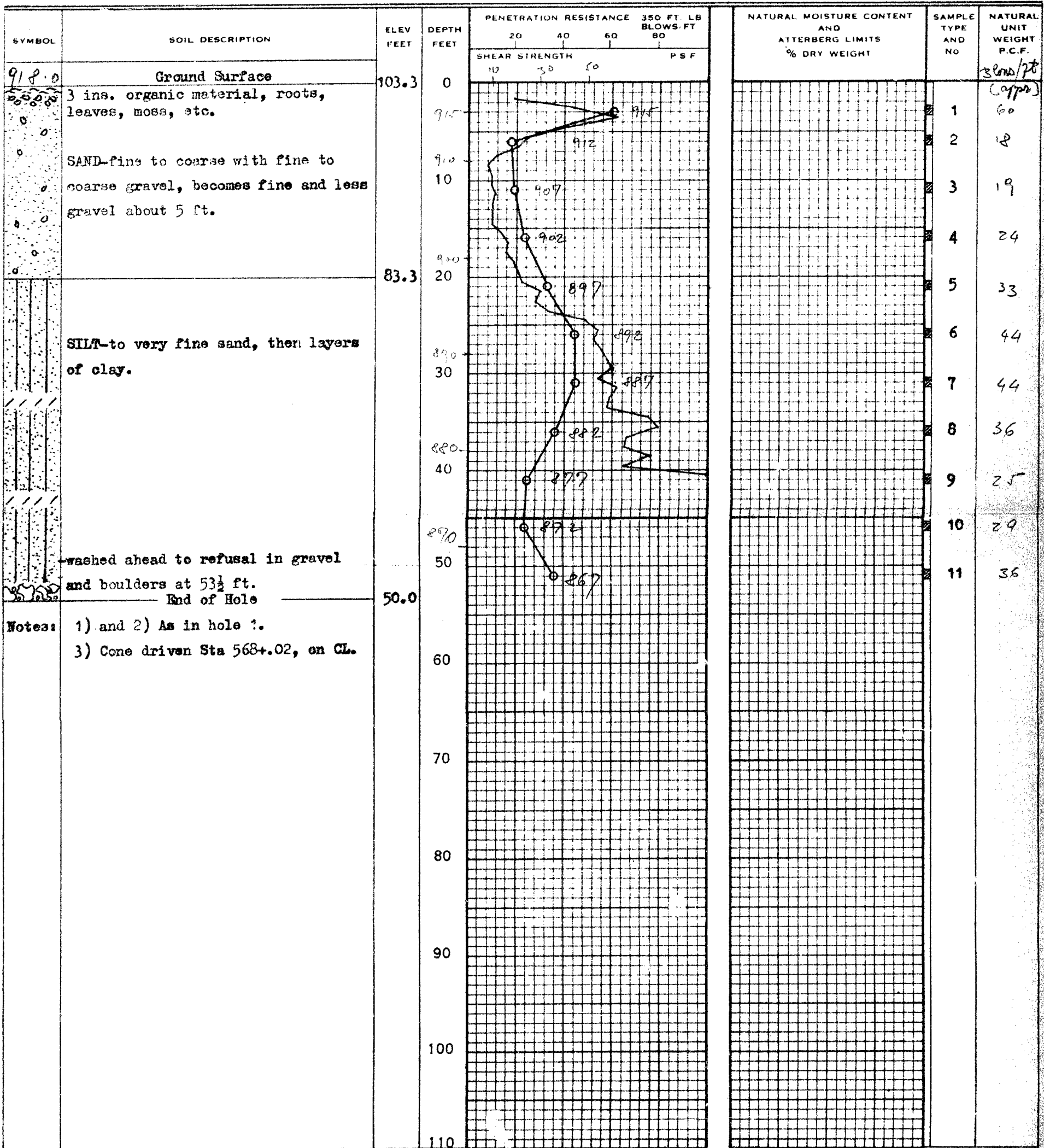
PLASTIC LIMIT ———

SAMPLE TYPE

2" O.D. SPLIT TUBE —■—

2" I.D. SHELBY TUBE —■—

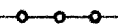
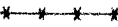
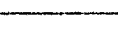
3" O.D. SHELBY TUBE —■—






LEGEND

BOREHOLE No. 7
 PROJECT Michipicoten River Crossing, W.P. 80-62
 LOCATION See Dwg. 1.
 HOLE LOCATION Sta 565+25, 4.5 ft. Right
 HOLE ELEVATION 95.7 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 

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			
	Ground Surface	95.7	0	SHEAR STRENGTH				
	WASH BORING ONLY							
	Essentially sand and gravel above							
	20 ft., some boulders between 5 and							
	15 ft.		10					
								
			20					
	Silty sand below 20 ft., some thin							
	clay layers.		30					
								
	BX casing driven to 60 ft.		40					
								
			50					
								
			60					
	AX casing driven below it to 83 ft.							
			70					
								
			80					
								
			90					
								
			100					
								
			110					
	GRAVEL AND BOULDERS.	12.4						
	End of Hole	7.4						
Notes:	1) Static penetration tests made on A red from 25 to 60 ft.							

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

LEGEND

DRAWING NO. 9
PROJECT NO. J887

BOREHOLE NO. 8
PROJECT Michipicoten River Crossing, W.P. 80-62
LOCATION See Dwg. 1.
HOLE LOCATION Centre of River
HOLE ELEVATION 87.43 ft. (River Bed)
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

ATTEBERG LIMITS

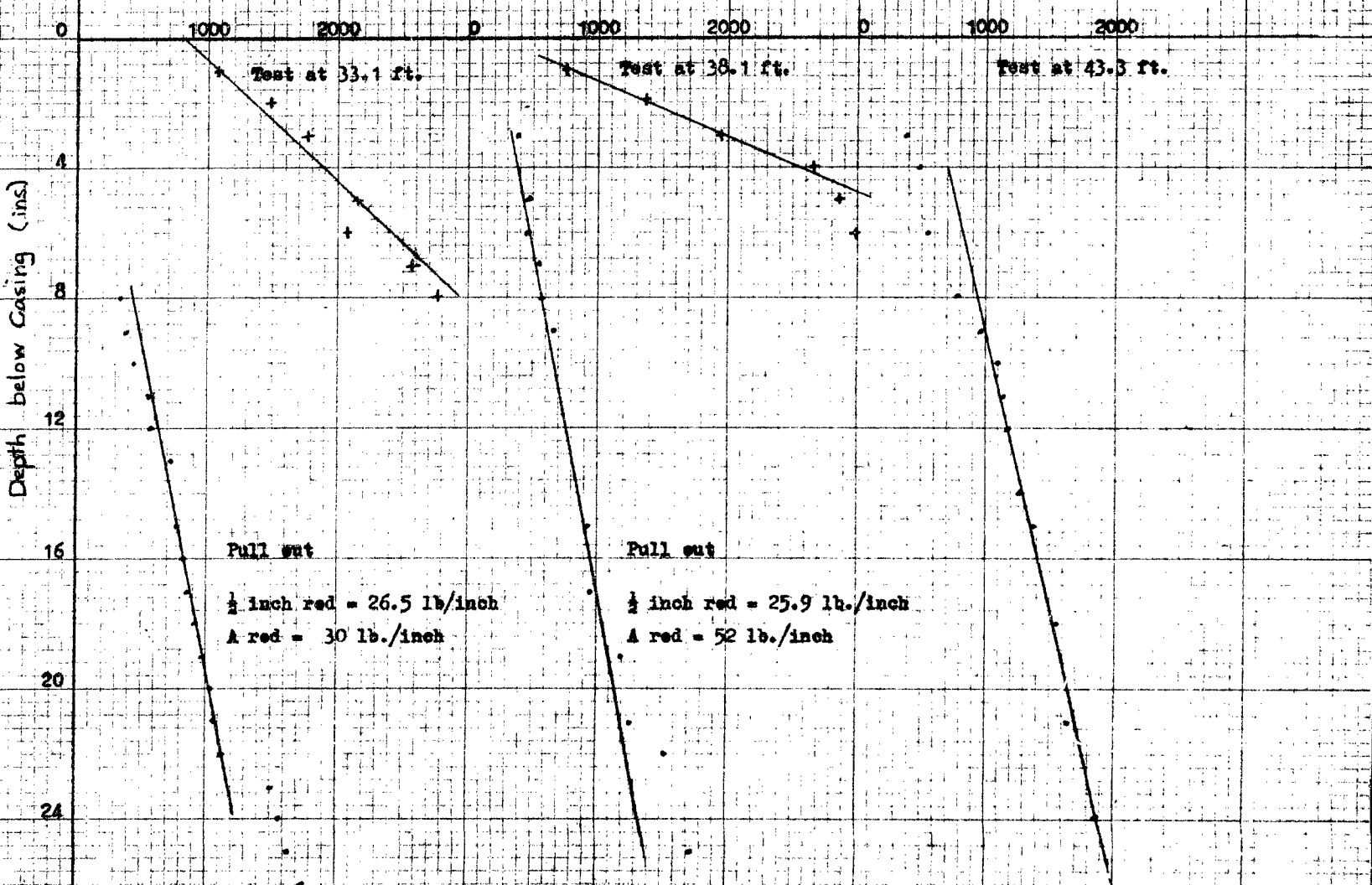
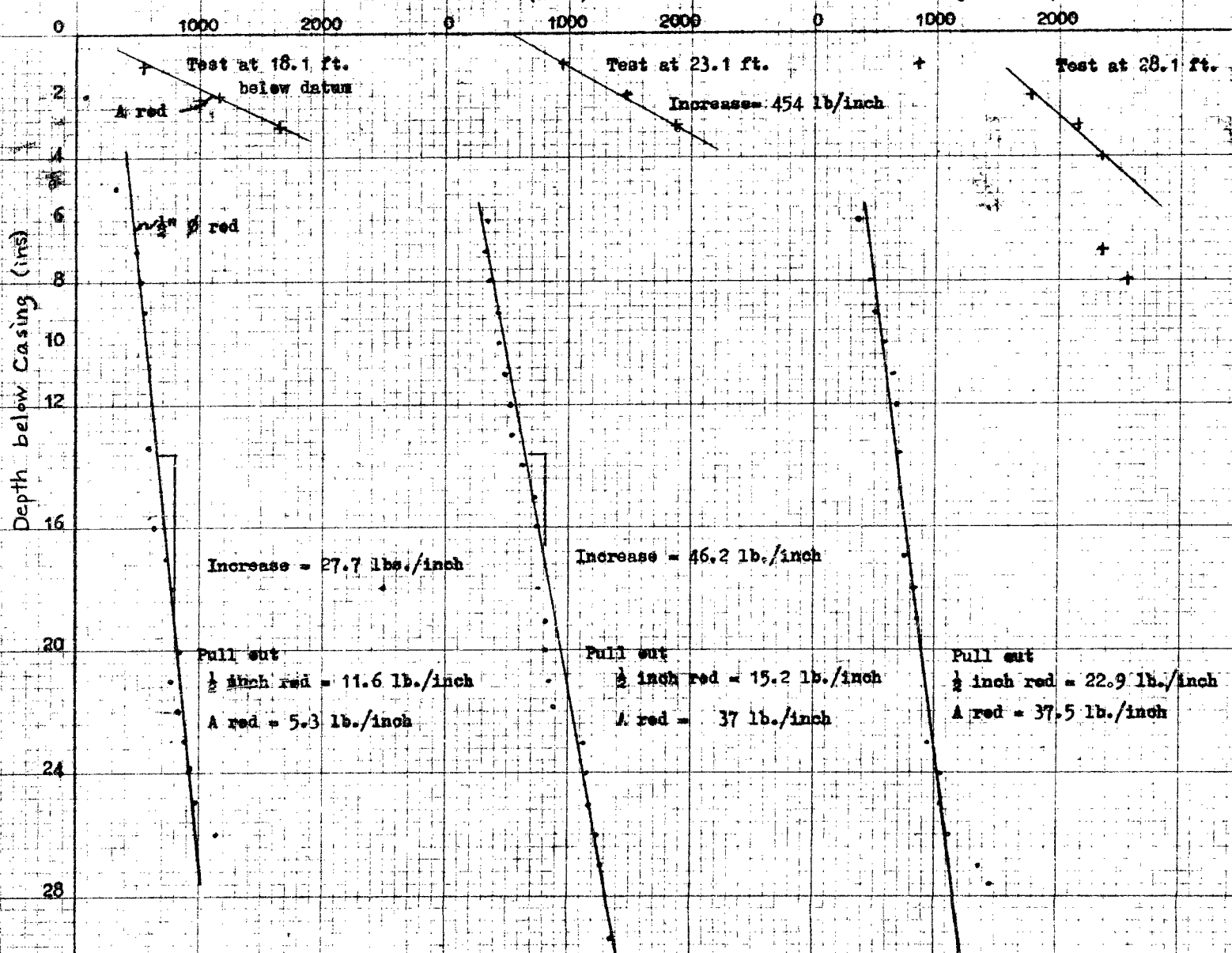
LIQUID LIMIT —○—

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		NATURAL MOISTURE CONTENT AND ATTEBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
				20	40			
				350 FT. LB. BLOWS FT. 80				
				SHEAR STRENGTH P.S.F.				
902.1	SAND-medium sand, fine gravel.	87.43	0					Blows/ft (approx)
		84.43	9.10					
	SILT-to very fine sand, brown; some clay seams; more clay seams below 20 ft., gravel and boulders below 30 ft.		10				1	13
			890				2	14
			20				3	12
	Sampling terminated at 30 ft. when cable support for raft broke, stream flow very fast.		28				4	32
			882				5	44
	End of Sampling	57	28				6	32
			870					
Notes:	1) and 2) As in hole 1. 3) Could not continue the hole because of very fast flow of water, raft washed to shore and almost lost, 30 ft. of casing lost.		40					
			50					
			60					
			70					
			80					
			90					
			100					
			110					



$$\text{Bearing Capacity} - Q = A(0.3YH_f + YZK_q) + YH_pK$$

Typical Calculation at 23 feet:

For $\frac{1}{2}$ inch red: diameter $D = 1/24$ ft.; Area $A = .001362$ sq.ft.; $\text{Perim. } p = .131$ ft.

For $1-5/8$ " red: " $D = .1355$ ft.; $A = .0144$ sq.ft.; $p = .425$ ft.

$Y = 65$ p.c.f.; $Z = 24$ ft.; $l = \text{depth of red in feet below casing.}$

For $H_f = H_q = H = 100$

$D = \frac{1}{2}$ inch friction factor $K = 2.59$

$D = 1-5/8$ in. " " $K = 8.08$

Pull out test for

$D = \frac{1}{2}$ inch " " $K_u = 0.89$

" for

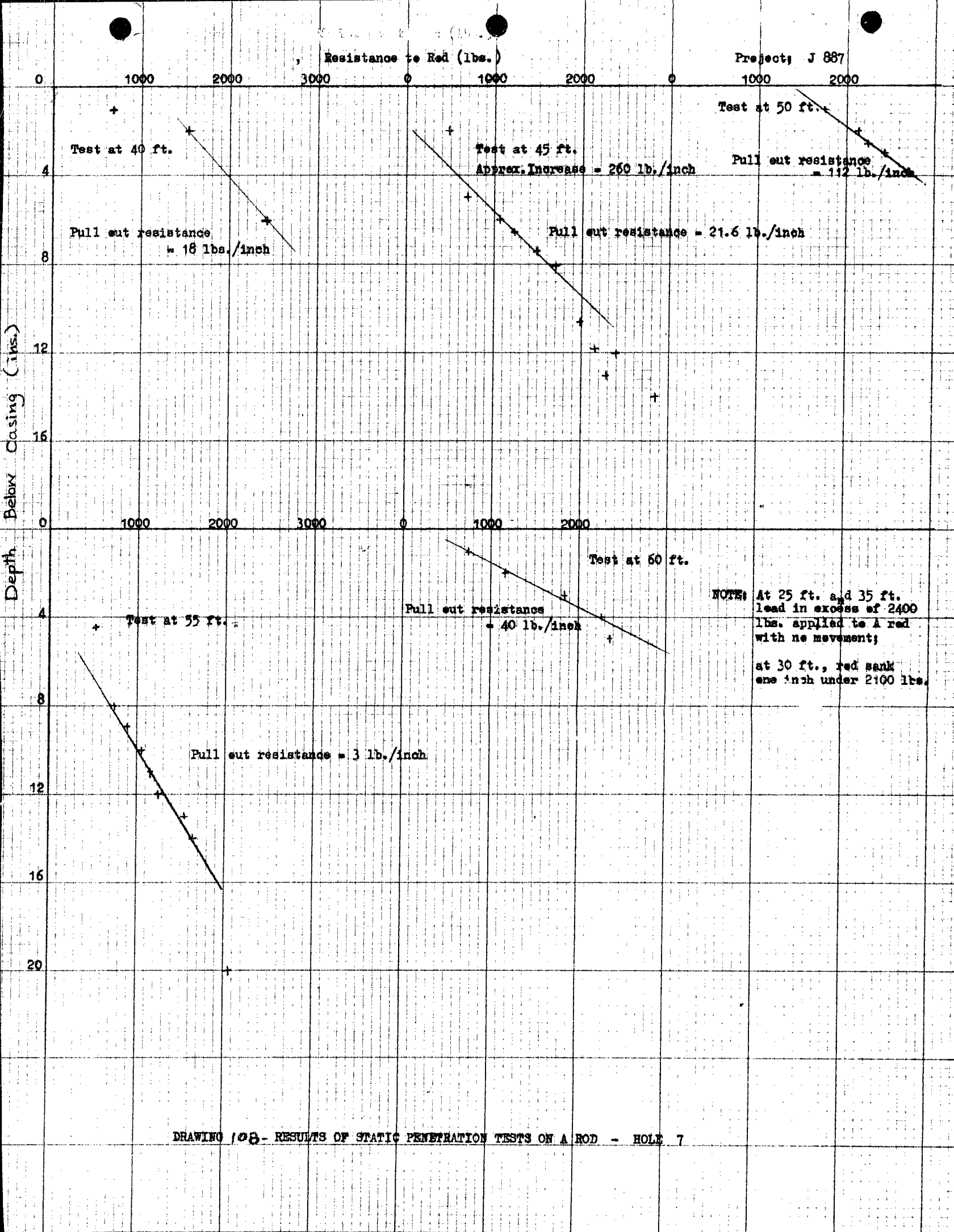
$D = 1-5/8$ in. " " $K_u = 0.67$

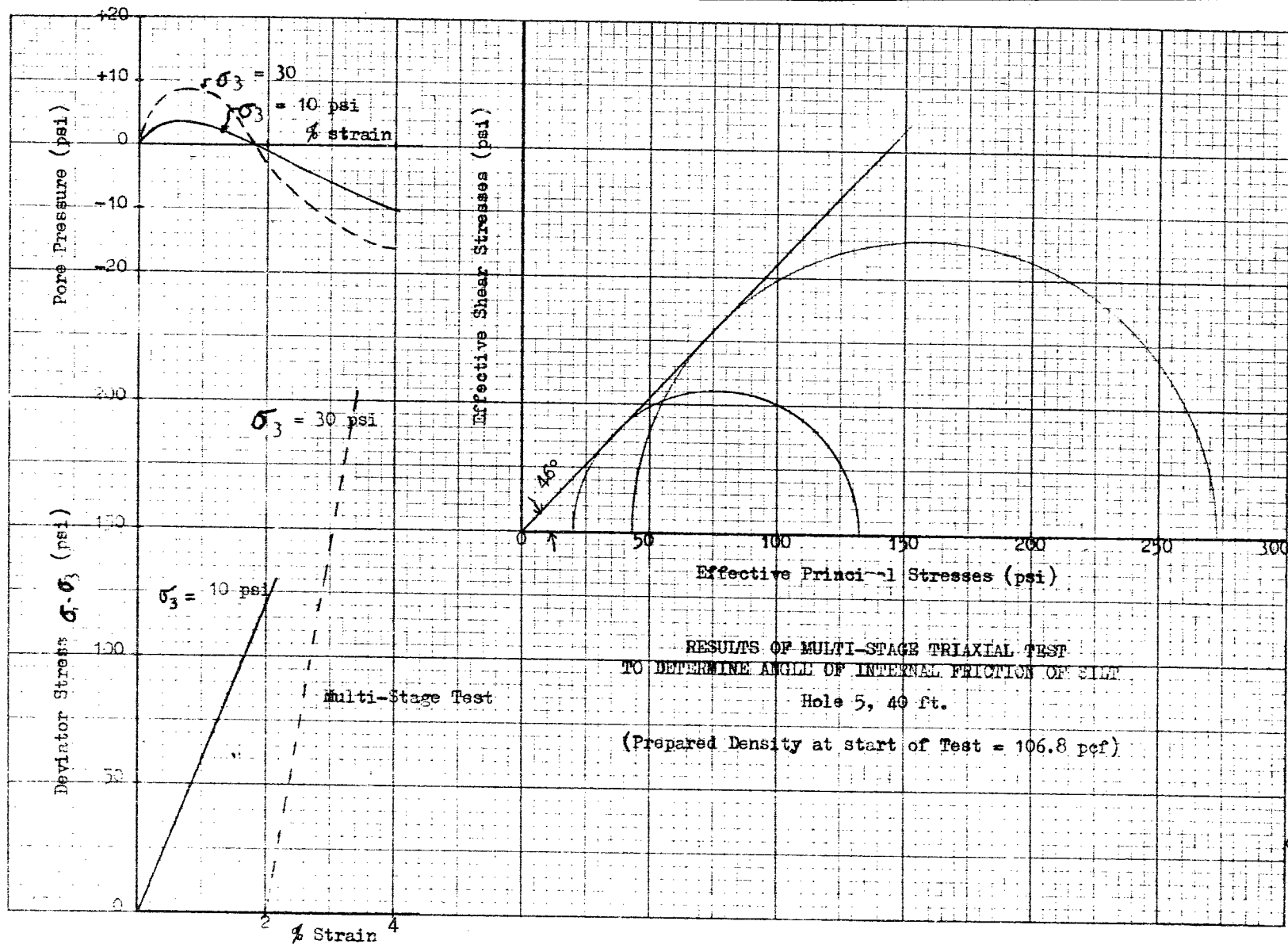
Using values of K_u in bearing capacity formula:

for

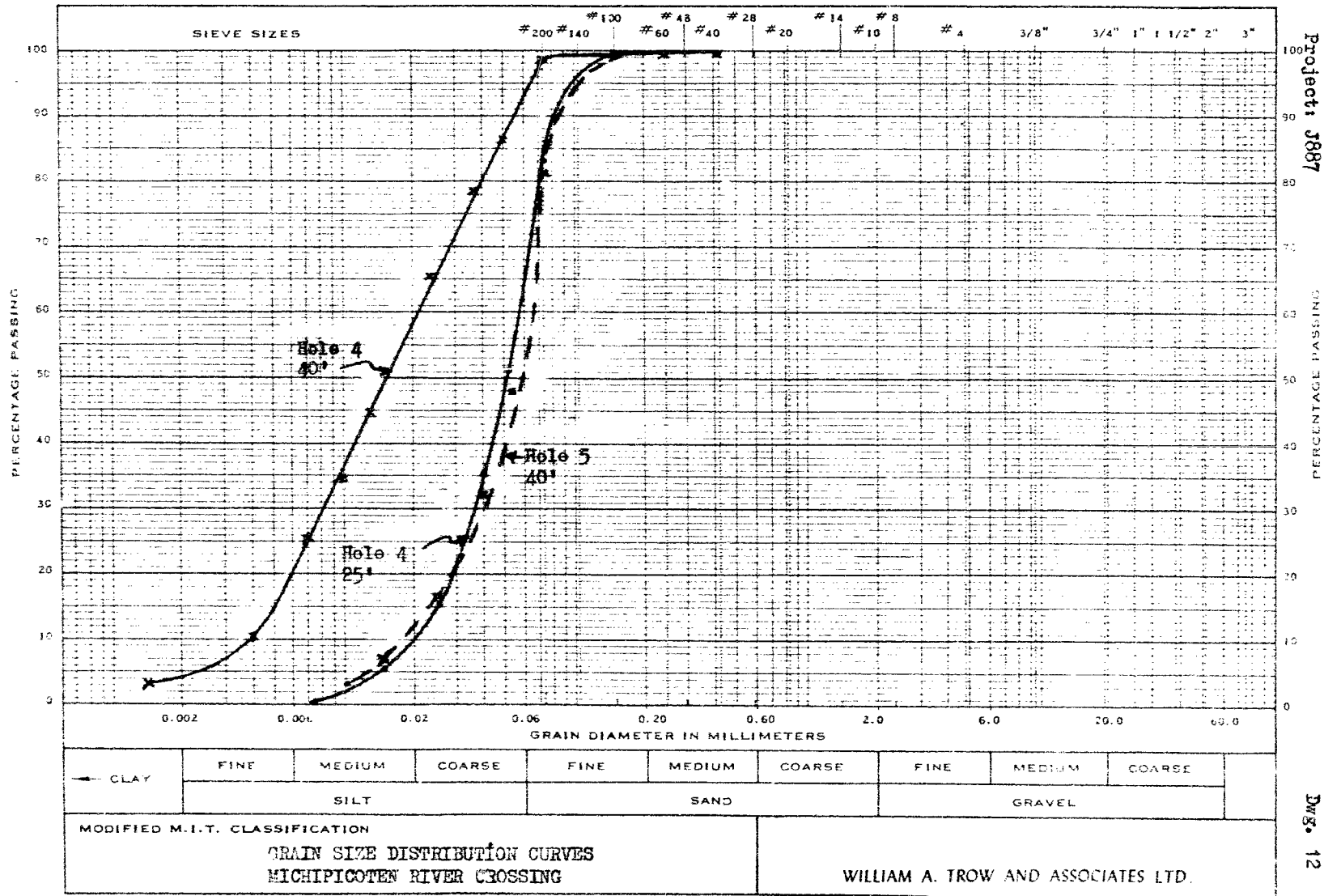
$D = \frac{1}{2}$ inch-bearing capacity factor $H = 421$

$D = 1-5/8$ in. " " $H = 534$





MECHANICAL ANALYSIS



MEMORANDUM

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

From: J. C. McAllister

Date: April 23, 1963.

Our File Ref.

In Reply To

Subject: W.P. 80-62
Mitchipicoten River
Hwy. 101 District #18

Attached please find one print of plan
D 5205-P1 for the above crossing. The foundation
has been designed in accordance with the foundation
report submitted by W. A. Trow & Associates.

J. C. McAllister

JCMcA/et

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

*210 Comanche
07-12-63
April 24/63*

*Bridge office informed
on May 6th 1963 by M. Devata
May 7, 1963 Agt. M. Devata*

Materials and Research Division

June 8, 1962.

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

Attention: Mr. W. A. Trew.

Re: W.P. 80-62, Hwy. 101, Michipicoten River,
District #18, Sault Ste. Marie.

Dear Sir:-

Please consider this your authority to carry out a foundation investigation at the above site, and the related approach fills as discussed verbally. A site plan of this location will be forwarded to you in the very near future.

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 prior to July 18, 1962. 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 invoices 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.

WAW/ndef

Yours very truly,

cc: Messrs. J. McCombie
H. McArthur
D. P. Collins
E. R. Saint
K. D. Smith (2)
Mrs. T. Tate
Foundations Office ✓
Gen. Files (2)

A. Rutka
A. Rutka,
MATERIALS AND RESEARCH DEPT.

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

August 7, 1962.

FOUNDATION INVESTIGATION REPORT

By: Wm. A. Frow & Assoc., Ltd.

Attention: Mr. E. McGhie.

Re: Hwy. #101 Crossing Michipicoten River,
W.P. 30-62 -- District #18.

Attached, we are forwarding to you the report for the above-mentioned foundation investigation submitted by Wm. A. Frow & Associates, Ltd.

We have reviewed the report, and on the basis of the presented factual data and information, we agree with the conclusions and recommendations contained therein.

We believe that the given recommendations will prove to be adequate for your future design work. However, should there be any other questions in connection with this project that you would like to discuss, please do not hesitate to call on our office.

WAF/MLK
-attach.

cc: Messrs. A. M. Foye (2)
E. A. Fregaskes
R. C. McMillan
H. McArthur
J. W. MacDougall
R. B. Saint
T. J. Kovach
J. Day
S. A. Gruspler
F. Moran
J. Watt
Foundations Office
Gen. Files.

WAF
W. A. Frow,
SUPERVISING FOUNDATION ENG.
Per:

A. B. Starnes,
PRINCIPAL FOUNDATION ENG.

cc: Mr. N. D. SMITH

Materials and Research Division

May 3, 1962.

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

Attention: Mr. Wm. A. Trow.

Dear Sir:-

The following is a list of jobs to be done on Hwy. 101
which were discussed at the meeting of today's date.

Little Pine Lake Crossing:

Revised line - Hwy. 101
Mileage 22,
Station 1160+00 (approx.)

WP 187-60

A.C. & H.B. Rlwy. Crossing:

Hwy. 101
Mileage 62,
Station 325+00 (approx.)
Plan No. G 2438.

WP 79-62 ✓

Mitchipicoten River Crossing:

Hwy. 101
Station 560+00 (approx.)
Plan No. E 2875-1

WP 52-62 ✓

Kiniwabi River (3 crossings):

Station 910+00 (approx.) -- Plan No. E 2840-1
Station 955+00 (approx.) -- Plan No. E 2839-1
Station 1000+00 (approx.) -- Plan No. E 2838-1

Trow 62

WP 187-60
WP 79-62
WP 52-62
WP 187-60
WP 79-62
WP 52-62

cont'd. /2 ...

Mr. Wm. A. Trow

May 3, 1962.

It has not been decided as yet, whether investigations will be necessary at all of the above locations. This information should be forthcoming by the end of May, and in the case of Little Pine Lake, by the end of June.

Surveys are required at all locations to establish the exact crossing points. As soon as definite information is available, all necessary plans, etc., will be forwarded to you, together with the relevant letters of authority.

Yours very truly,

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.
Per:

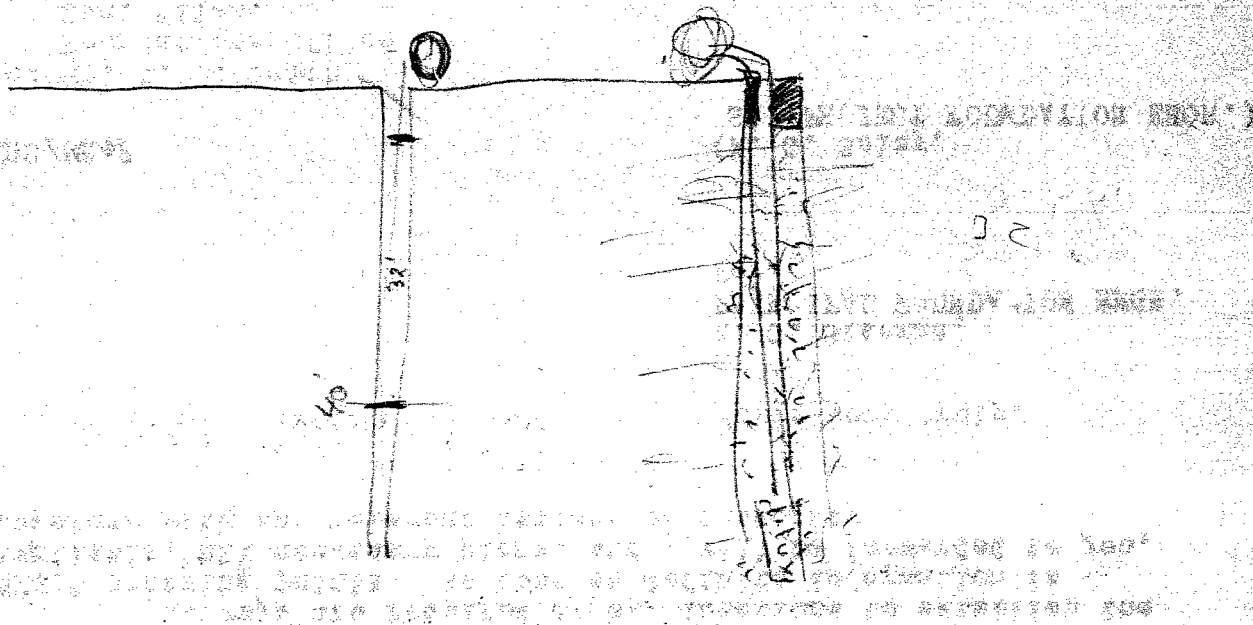
KGS/MdEF

cc: Mr. N. D. Smith ✓
Foundations Office ✓
Gen. Files.

8-64
(K. G. Selby,
SR. PROJECT FOUNDATION ENGR.)

Sept / 64

Nov. 23 / 65



Nov. 23 / 65

Nov. 23 / 65

Nov. 23 / 65