

#

58-F-257C

W.P.[#] 104-57

Hwy[#] 11

MAGNETAWAN R.

BRIDGE KATRINE

31 Mar 59. ²⁰
46

W.P. 104-57

A+ "Katrine"
5 miles South
of Burke Falls.

MAGNETAWAN RIVER BRIDGE.

HWY #11 - DISTR 11 (HUNTSVILLE)

Road diversion

Roadway 30' + 2' curbs.

Span 120' - Built up steel girders.
as in Thessalon River bridge.

Skew to fit river (45°).

Vert. clearance old bridge 8.8'.

" " New " 9.0' ±.

Slow flow - deep water - 12' at centre

Sand & muck to hard clay.

Timber ^{BR. 1255} Piles - say 60' - ^{20 tons} but provide splice detail.

Space ~~4~~ 3' - Load 15 Tons max.

Footings 6' deep - Piles 3' into footing.

Footing base say 950.0.

Std. steel H. Rails.

Prelim plan. D4321 - P1.

See Boring data.

See Site.

B. Wilkie has
Survey plans

TROW, SODERMAN AND ASSOCIATES

58-257C

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

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ST. 8-5921

Project: C108/J224

July 3, 1958

Mr. A. M. Toye,
Bridge Engineer,
Dept. of Highways of Ontario,
280 Davenport Rd.,
Toronto, Ont.

Attention: Mr. S. McCombie

Foundation Investigation
Magnetawan River Bridge,
Highway 11, Katrine, Ont.

Dear Sirs:

Enclosed herewith is our report on the soil conditions underlying the above-noted bridge site. In it we have recommended support of the proposed structure on deep end-bearing piles, driven approximately 40 to 65 feet below present ground level. The safe capacity of a one foot diameter pile at these depths is estimated conservatively to be 28 tons.

Support at much lower capacity is also possible in the overlying deep deposit of varved clay. However, specific comments concerning settlement conditions for a foundation in this clay will require a knowledge of the bridge loads anticipated.

We shall be pleased to discuss any queries you may have after your review of the contents of this report.

Yours very truly,

W. A. Trow

William A. Trow (P. Eng.)

WAT/lt
Encls.

DEPARTMENT OF HIGHWAYS OF ONTARIO
280 DAVENPORT ROAD
TORONTO, ONTARIO.

FOUNDATION INVESTIGATION
MAGNETAWAN RIVER BRIDGE
HIGHWAY 11, KATRINE, ONT.

July 3, 1958.

Trow Soderman & Associates

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FOUNDATION INVESTIGATION
MAGNETAWAN RIVER BRIDGE
HIGHWAY 11, KATRINE, ONT.

This report describes the results of a foundation investigation performed along Line J of the proposed Highway 11 crossing of the Magnetawan River approximately 5 miles south of Burk's Falls, Ont. The field work associated with this investigation was performed during the period from June 9th to June 23rd, and consisted of six borings taken to depths of approximately 90 feet below present ground surface.

Description

The proposed bridge crossing of this river represents a revision in the road alignment and lies approximately 50 feet to the east of the existing bridge structure which is a simple span steel truss. The south approach to the river is over low-lying heavily-wooded ground, approximately 2½ feet above present river level. The highway route north of the site rises on a gentle wooded slope up to ground approximately 15 feet higher than the river level. No bedrock outcrops were in evidence in the immediate vicinity, although higher ground lies both to the north and to the south.

The Magnetawan River flows very sluggishly at this point and the river banks are overgrown with small bushes, a condition suggesting that rapid stream flow and high water are not common conditions here. The concrete abutments, supporting the existing light structure, appear to be in good condition. It is assumed that these abutments bear on piles. }

Field Investigation Methods

The subsoil conditions at this site were determined using conventional drilling equipment and 3-inch I.D. cased borings. This involves the repetitious procedure of driving the casing to the required sampling depth, washing out the enclosed material to this depth and obtaining a sample in the underlying soil. For the most part, the samples were recovered in a partially disturbed but representative state using a thick-walled two-inch O.D. split spoon. This spoon was driven into the ground under the standard energy of 350 ft.lbs. per blow and the number of blows required for 12 inches of penetration provided an empirical measure of the relative density of the soil. In addition to this field measurement, these samples were suitable for identification purposes and for revealing the presence of varving or fissuring in the soil. In the cohesive varved clay encountered a few feet below existing ground level, some thin-walled 2-inch I.D. Shelby tube samples were recovered. These undisturbed samples were sealed with a low melting point wax and transported in a car to the laboratory for strength determinations. In situ field vane measurements of shear strength also were obtained in this varved clay.

Before sampling at any borehole location, a 2-inch cone was driven in order to indicate the changes in density with depth. This cone was driven at the standard energy of 350 ft.lbs.per blow.

Considerable difficulty was encountered in advancing the boreholes below approximate Elev. 907 ft. This was because the underlying fine sand and gravel continued to rise in the casing and could not be washed out prior to sampling. Efforts were made to establish the final refusal depth by driving a cone or an open 1-5/8 inch O.D. A rod below the bottom of the casing. By this method, refusal was encountered in dense sand or gravel at depths ranging from Elev. 888 ft. to Elev. 878 ft. ^{953 - cut pit}
₈₈₈
_{85' to 75'}

All field work was supervised by a graduate soils engineer and the elevations and locations of the various borings were referenced to the location stakes established for Line J.

Description of Soil Conditions

Details concerning soil types and physical properties for each borehole location are recorded in drawings Nos. 3 to 8 of this report. In order to assist in the appraisal of the overall subsoil competence, this information has been summarized in the estimated stratigraphical chart shown in drawing No. 2.

On the south side of the river, the ground surface is underlain by a loose deposit of fine organic sand with layers of peat, twigs and other organic material. This deposit is approximately 20 feet thick and extends to Elev. 950 ft.

Directly beneath the loose sand lies a thick deposit of varved or layered stiff clay consisting of alternate layers of clay and silt. The clay phase ranges in thickness from 0.2 inches to 1.5 inches and the silt layers vary from 0.05 inches to 0.15 inches. Atterberg Limit tests indicate that the clay portions of the soil are quite plastic with a liquid and plastic limit of the order of 62% and 27% dry weight respectively. The natural moisture content of the clay is relatively close to the liquid limit. Tests also were performed on the silt phase of each varve but accurate measurements were not obtained because of the difficulty in separating the thin silt layers from the clay. Field vane measurements indicated a cohesive strength ranging from 900 to 2500 p.s.f. which values were well above the results of undrained triaxial tests made on samples from holes 2 and 3. This discrepancy has been noted on previous occasions with this type of deposit; a more comprehensive program would be necessary to determine which method of shear strength measurement is the more reliable. Additional testing on samples from the other borings did not appear warranted however, for the reasons given in the following discussion. At approximate elevation 913 feet, the varved clay grades gradually into silt and then into fine silty sand. This soil exists in a medium dense to dense condition. It becomes denser and more granular

with depth and offered refusal to the drilling equipment between Elev. 878 and 888 feet. Between Elev. 898 feet and 888 feet in hole 1, the sand was under a slight artesian pressure and sampling difficulties were encountered below approximate Elev. 907 feet in all borings because sand continued to rise well up into the casing.

On the north side of the river, similar soil types were encountered with the exception that the layer of varved clay was much thinner and it was overlain by about 10 feet of stiff sandy clay and silt. It is possible that the clay lenses out against the hillside to the north. Refusal on the north side of the river ranged from Elev. 883 feet to Elev. 885 feet, or approximately 80 feet below present ground surface.

953 cur pil
883
70 to 72'

Discussion of Results

The two strata which could be considered as supporting media for the proposed bridge are the stiff varved clay and the underlying fine silty sand. All materials above the varved clay are unsuitable because of their loose organic nature or because they terminate above river bed level. A piled foundation, therefore, appears warranted.

The capacity of a pile foundation floating in the varved clay will be determined by the final shear strength of the clay after pile driving operations are complete, and by its consolidation characteristics under full dead load. The existing shear strength of the clay has been determined by in-situ field vane measurements and by undrained triaxial tests on undisturbed samples tested at overburden pressure conditions. As stated in the foregoing section a considerable discrepancy was noted when comparing these two independent methods for measuring shear strength. Although it may be desirable to resolve this uncertainty, it is equally important to consider what strength will be available after piles are driven. The ultimate capacity of a pile in clay is given by the expression:

$$Q = 9cA + cPL$$

where c is the cohesion of the remoulded clay film immediately adjacent to the pile
 A is the pile tip area
 PL is the surface area exposed to the clay on the pile shaft.

In "normally loaded" clays where shear strength has been determined and developed by the weight of existing overburden pressures, the original strength value for the cohesion, c , is quickly regained in the remoulded clay film after pile driving operations have been completed. Therefore, the measured shear strength, prior to construction, can be used for computing pile capacity provided a suitable factor of safety is used to account for minor variations in the soil and for long term settlement.

When piles are driven into precompressed clays, however, the complete recovery of original shear strength does not occur. Here again, the soil is remoulded by pile driving operations, but the recovery of shear strength will be determined by the existing weight of overburden rather than by the weight or prestress which was effective in precompressing the clay. In this instance, therefore, the remoulded clay will return to the normally loaded state. It follows, then, that the end result is the same for either type of deposit.

Although the relationship between effective pressure, P , and shear strength, c , for this varved clay has not been determined, experience with other clays in the area suggests that a ratio of $c/P = 0.3$ is reasonable. At a depth of 20 feet and at 60 feet under the south side of the river, the value for P ranges from 1000 to 3600 p.s.f. The corresponding values for c would be 300 and 1080 p.s.f. respectively. Therefore, referring to the above formula, the estimated safe capacity for a one-foot diameter cylindrical pile, driven just above the bottom of the clay would be about 15 tons. This value incorporates a factor of safety of 3. A lower capacity would be available with shallower penetration.

A significant observation of this investigation is that the clay stratum under the north side of the river is much thinner than exists under the south bank. Because of this inconsistency a lower capacity per pile will be available under the north abutment unless the piles are driven into the underlying sand. In either event, however, less settlement should be anticipated under the north abutment than will occur to the south because a thicker deposit of compressible soil underlies this latter location. In view of this differential state of compressibility, it would appear wise to carry pile loads through and below this varved clay. More definite statements concerning the anticipated settlement of pile foundations floating in the varved clay would require a knowledge of the loads anticipated for this structure.

Cylindrical end-bearing piles driven into the underlying silt and silty sand should encounter refusal at approximate Elev. 900 feet under the south abutment and at Elev. 910 feet to the north. Because of their thin section, steel H piles may continue to depths of 85 feet, or approximate Elev. 885 feet before satisfactory bearing is obtained.

The ultimate capacity of a cylindrical pile can be computed both theoretically and by empirical means. The latter method is based upon field penetration measurements and is expressed by the relationship:

$$Q = \frac{4 N A_s}{3}$$

where N is the standard penetration test value equal approximately to $2/3$ of the cone penetration measurement.
 A_s is the area of the pile tip
 3 is the factor of safety.

For a cone penetration value of $N = 40$, $Q = 28$ tons for a one-foot diameter pile.

Reference to drawing No. 2 indicates that this value of N , and hence this capacity, should be obtained at a depth of 65 feet, or Elev. 905 under the south abutment and at Elev. 930 feet under the north bank. The ultimate structural capacity of each pile will be obtained with penetration to the aforementioned refusal depths at Elev. 900 feet and 910 feet.

The theoretical capacity of a single pile can be computed from the expression:

$$Q = \frac{N_q A \gamma D}{F}$$

where $N_q = 135$ for $D \geq 10$
 A is the area of the pile tip
 γ is the in situ unit weight of the soil
 D is the depth below ground surface in feet
 F is the factor of safety, taken here as 3.

For a one-foot diameter pile driven to a depth of 60 feet, the value for Q is 63 tons. For the 40 foot piles of the north abutment the computed capacity of a one-foot diameter pile is 42 tons.

These latter estimates of capacity are considerably in excess of the value recommended on the basis of empirical rules. Some reduction should be made in order to allow for the minor compressibility of the silt and silty sand. In view of this, it may be desirable to select the previous empirical estimate of 28 tons although this value may be somewhat too conservative. Since a major portion of the capacity of piles in sand is developed in end-bearing, close pile spacing can be permitted. 3' 0"

Conclusions

The foregoing comments can be summarized briefly as follows:

- (1) The site of the revised crossing of the Magnetawan River lies near the north end of a stretch of low-lying ground. The site is underlain by a thick deposit of stiff varved clay and then by dense silty sand, as shown in Drawing No. 2. The thickness of the clay increases sharply toward the south.
- (2) Since the upper levels of ground are too loose and compressible, support for the proposed bridge structure must be on piles driven either into the clay or into the underlying sand. Support in the clay is not recommended because of the anticipated differential settlement. The estimated safe capacity for a single pile driven almost through the deeper section of the clay is 15 tons.
- (3) Cylindrical end-bearing piles should encounter satisfactory support at depths of approximately 65 feet under the south abutments and 40 feet under the north bank. The safe capacity for a one-foot diameter pile

at these depths is conservatively estimated to be 28 tons. Refusal to cylindrical piles may require an additional penetration of 20 feet under the north abutment.

(4) Steel H piles may extend to a depth of 85 feet before encountering refusal. Their safe capacity will be determined by their structural properties when considered as short columns.

WAT/lt
July 3, 1958.
G108/J224



W. A. Trow

William A. Trow (P. Eng.)

SUMMARY OF LABORATORY TEST RESULTS
MAGNETAWAN RIVER CROSSING

Borehole No.2	Depth feet	Undrained Triaxial Strength p.s.f.	Consistency % Dry Weight					
			Clay Phase			Silt Phase		
			Wn	L.L.	P.L.	Wn	L.L.	P.L.
	31	680	44.5	45	25 (composite of silt and clay)			
	36	840	55.8	62	27	46.5	50	25
	41	800	67.9	68	29	31.2	35	21
	46	510	45.9	57	25	40.9	47	23
Borehole No.3	26	600	39.5 (composite of silt and clay)					

LEGEND: Wn - Natural moisture content
 L.L.- Liquid limit
 P.L.- Plastic limit

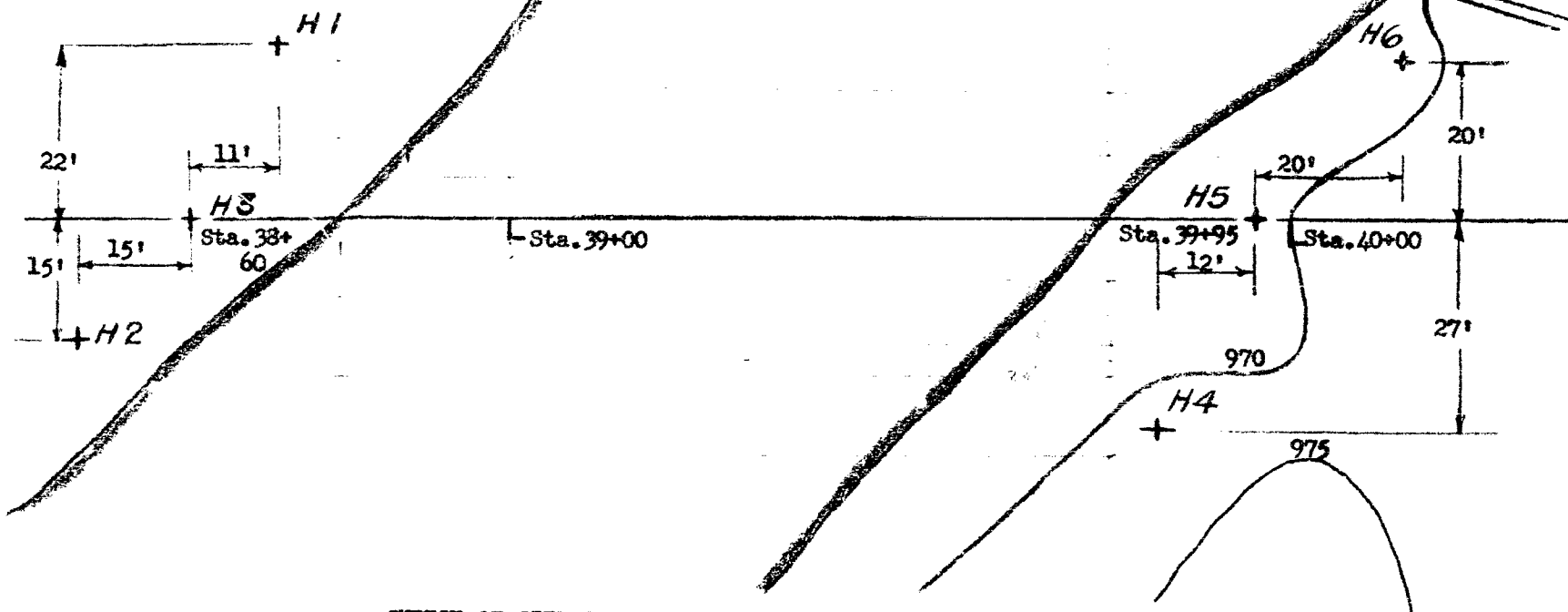
Highway #11
To Huntsville

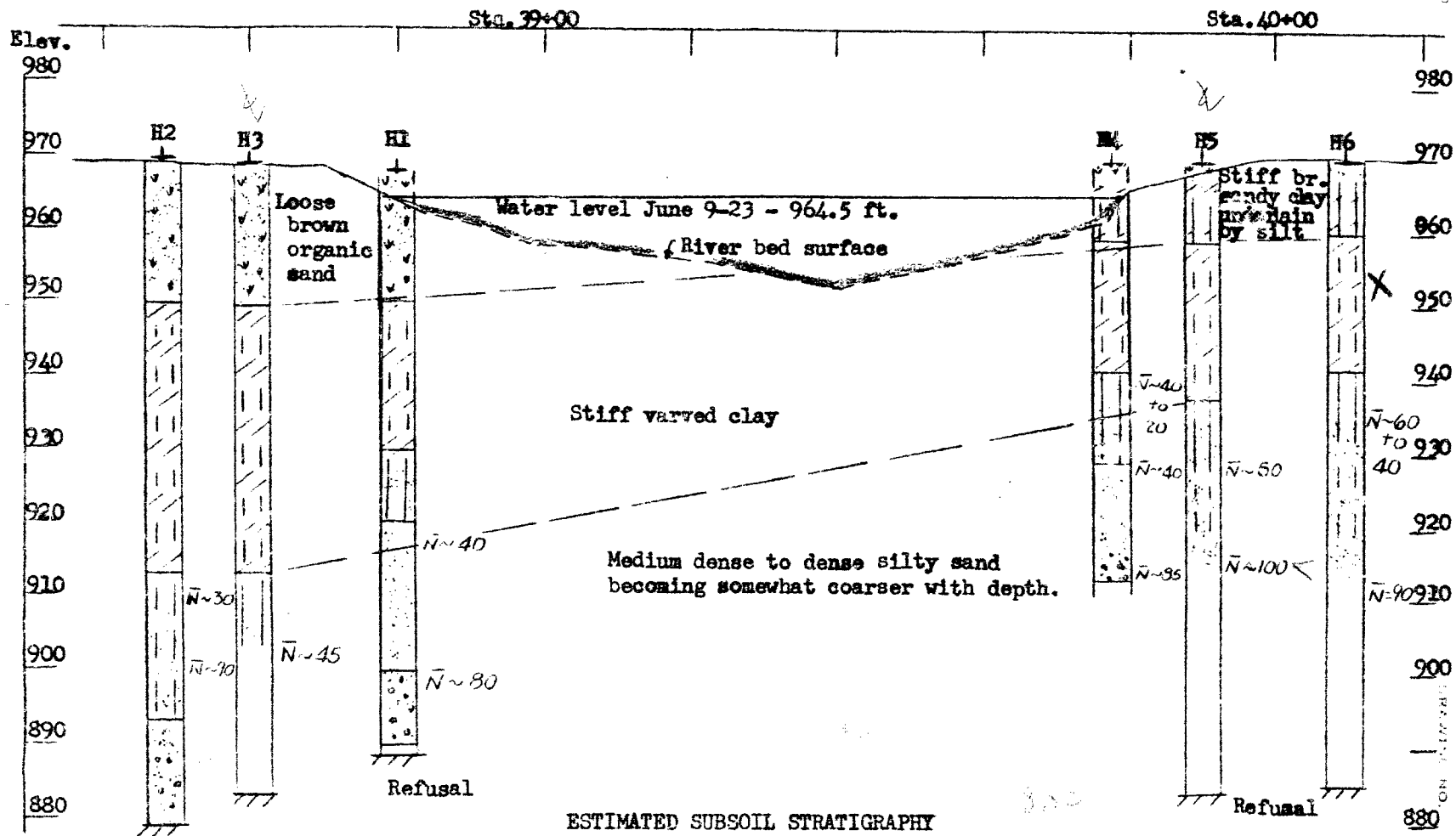
970

Existing Bridge

To Birk's Falls
(approx. 5 mi.)

Magnetawan River

SKETCH OF SITE SHOWING BOREHOLE LOCATIONS
Scale 1" = 20'



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Magnetawan River Crossing
 LOCATION Highway #11, Katrine, Ont.
 HOLE LOCATION See Dwg.1

HOLE ELEVATION AND DATUM 967.7 BM Sta. 40+82; 52 ft. right
 El. 976.40

BOREHOLE NO. 1

FIELD SUPERVISOR

DRILLER

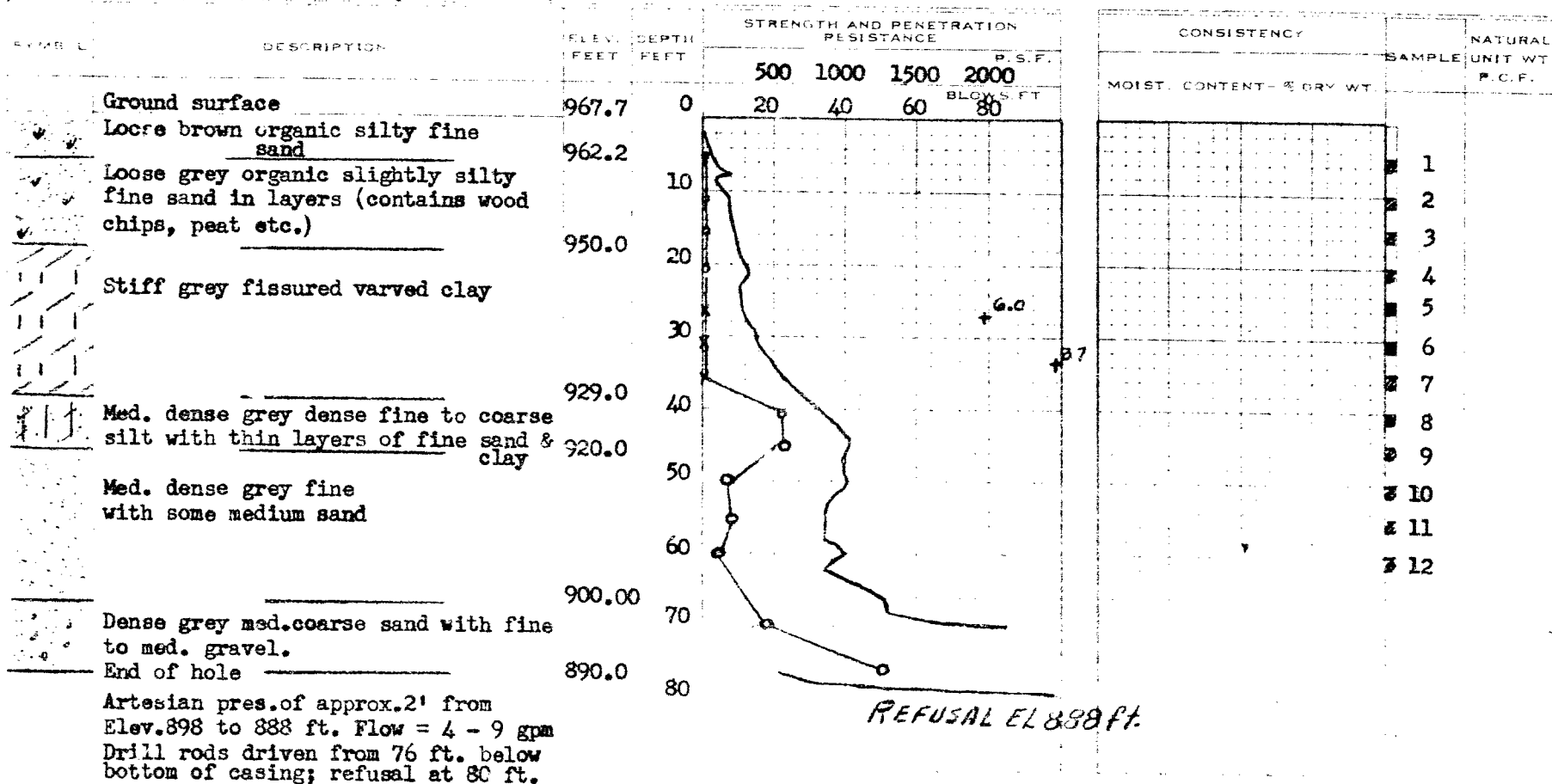
KP

MC

KP

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (QU)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT - Magnetawan River Crossing

LOCATION Hwy. #11, Katrine, Ont.

HOLE LOCATION See Dwg. 1

HOLE ELEVATION AND DATUM 968.5 BM Sta. 40+82: 52' Rt.
= 976.40

BOREHOLE NO. 2

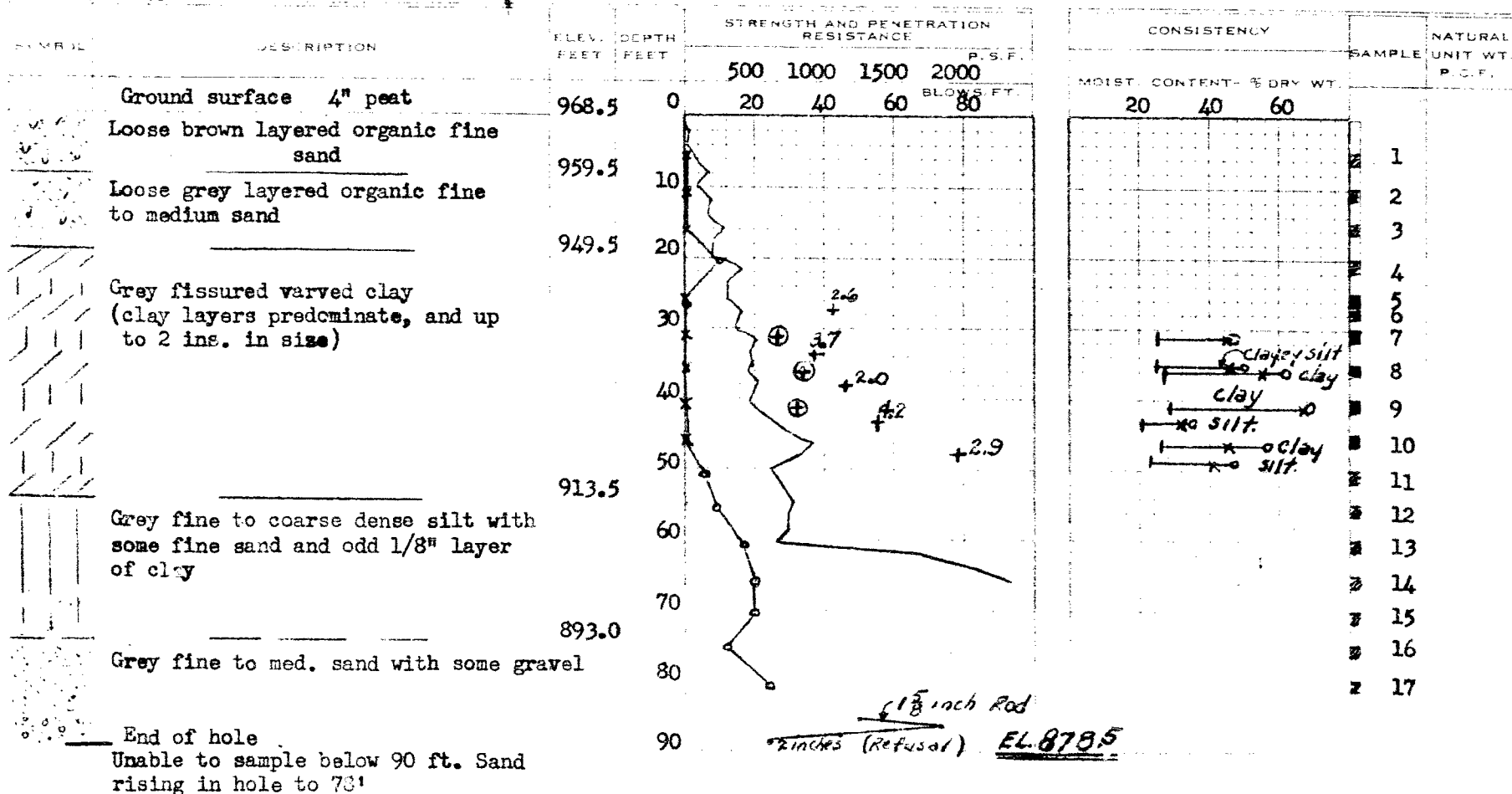
FIELD SUPERVISOR KP

DRILLER KP

PREP KP

LEGEND

- 1 DIA. SPLIT TUBE
- 2 SHELBY TUBE
- 3 SPLIT TUBE
- 4 DIA. CONE
- CASING
- 5 SHELBY
- 6 UNCONFINED COMPRESSION (Qu)
- 7 VANE TEST (C) AND SENSITIVITY (S)
- 8 NATURAL MOISTURE AND LIQUIDITY INDEX
- 9 LIQUID LIMIT
- 10 PLASTIC LIMIT



PROJECT NO.

C108 J 224

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Magnetawan River Crossing

LOCATION Hwy. #11 Katrine, Ont.

HOLE LOCATION See Dwg.1

HOLE DEPTH AND ELEVATION 968.2 - BM Sta. 40+82: 52 ft.
Ft. right- 976.40

BOREHOLE NO. 3

FIELD SUPERVISOR KP

DRILLER MC

KP

DRAWING NO. 5

LEGEND

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

CASING

2" SHELBY

1.2 UNCONFINED COMPRESSION (QU)

LAKE TEST (C) AND SENSITIVITY (S)

NATURAL MOISTURE AND

LIQUIDITY INDEX

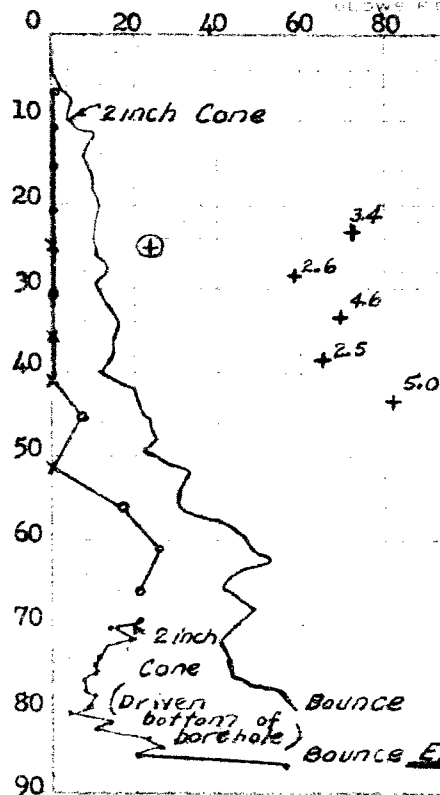
LIQUID LIMIT

PLASTIC LIMIT

DESCRIPTION	DEPTH FEET	ELEVATION FEET
Ground surface	0	968.2
Loose brown layered fine organic sand		962.0
Loose grey fine organic sand (layers of brown peat)		949.0
Stiff grey varved clay with trace of fine sand		913.0
Dense grey varved silt with fine sand and some very thin clay layers above 65 feet.		
Refusal on boulders encountered at Elev. 882.7		

STRENGTH AND PENETRATION RESISTANCE

500 1000 1500 2000



CONSISTENCY

MOIST. CONTENT (%)

20 40 60

NATURAL

SAMPLE UNIT W.

P.S.F.

- 1
- 2
- 3
- 4
- 5
- 6
- 7
- 8
- 9
- 10
- 11
- 12
- 13

Bounce EL. 882.7

PROJECT NO. C108 J224

DRAWING NO. 6

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Magnetawan River Crossing

LOCATION Hwy. #11 Katrine, Ont.

HOLE LOCATION See Dwg.1

HOLE ELEVATION AND DATUM 969.5 BM Sta.40+82: 52 Ft.
Rt. - 976.4

BOREHOLE NO. 4

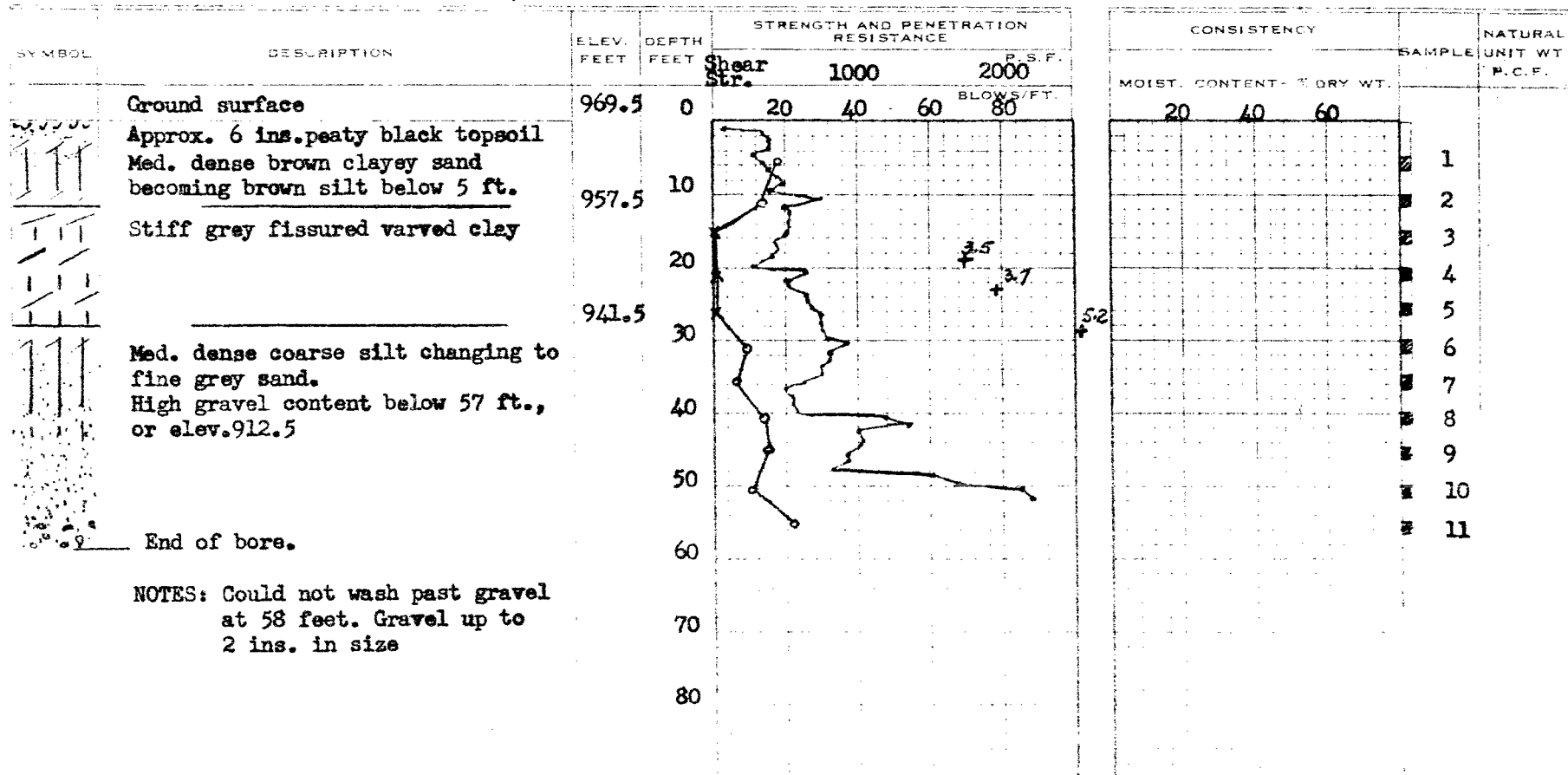
FIELD SUPERVISOR KP

DRILLER MC

PREP. KP

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (QU)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Magnetawan River Crossing

LOCATION Hwy. 11 Katrine Ont.

HOLE LOCATION See Dwg.1

HOLE ELEVATION AND DATUM 969.0 BM Sta.40+82: 52 Ft.
Rt. - 976.40

BOREHOLE NO. 5

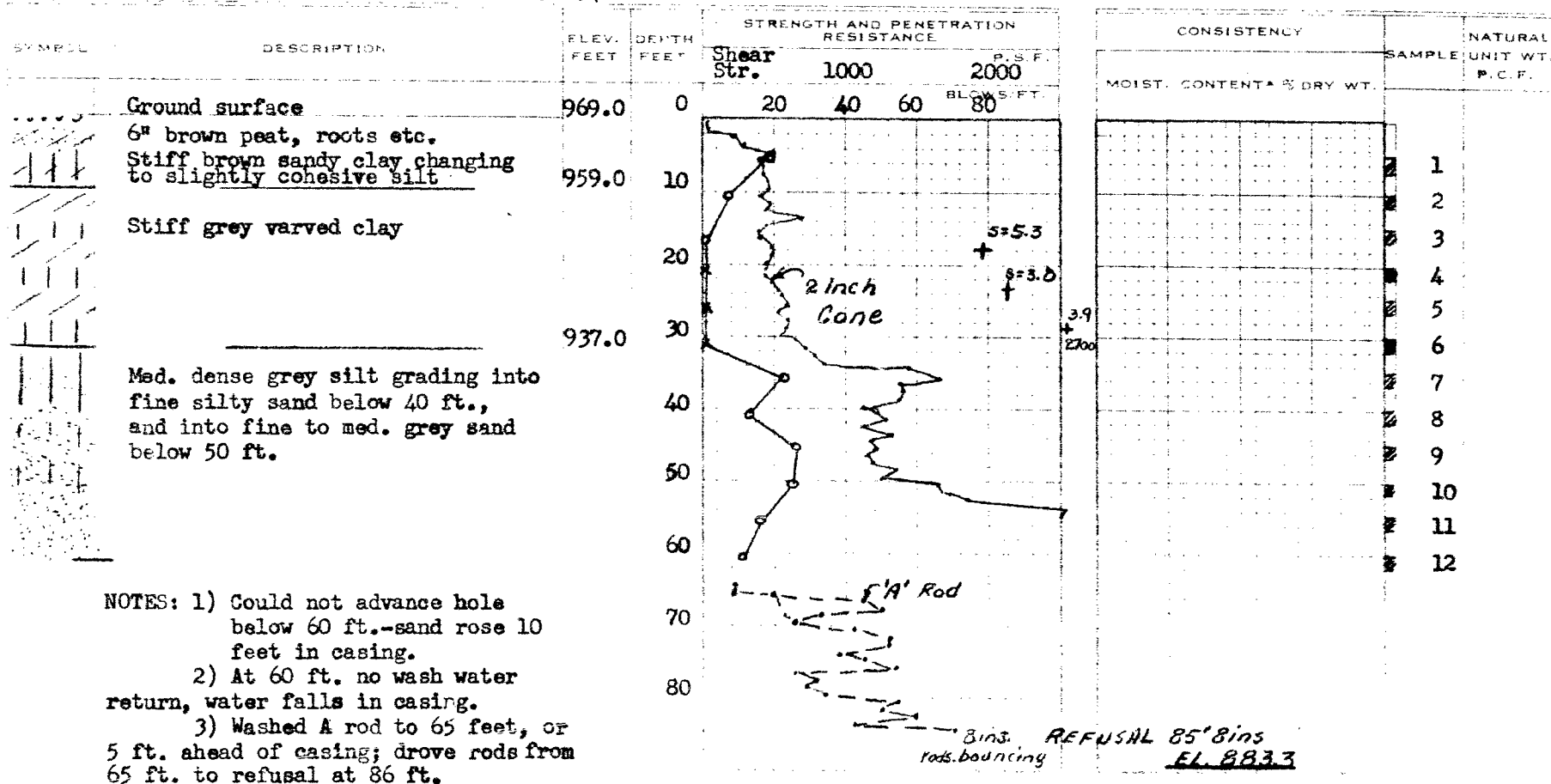
FIELD SUPERVISOR KP

DRILLER MC

PREP KP

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (Qu)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



PROJECT NO.

C108 J224

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

DRAWING NO. 8

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1.2 UNCONFINED COMPRESSION (Qu)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT

PROJECT Magnetawan River Crossing

LOCATION Hwy. 11 Katrine, Ont.

HOLE LOCATION See Dwg.1

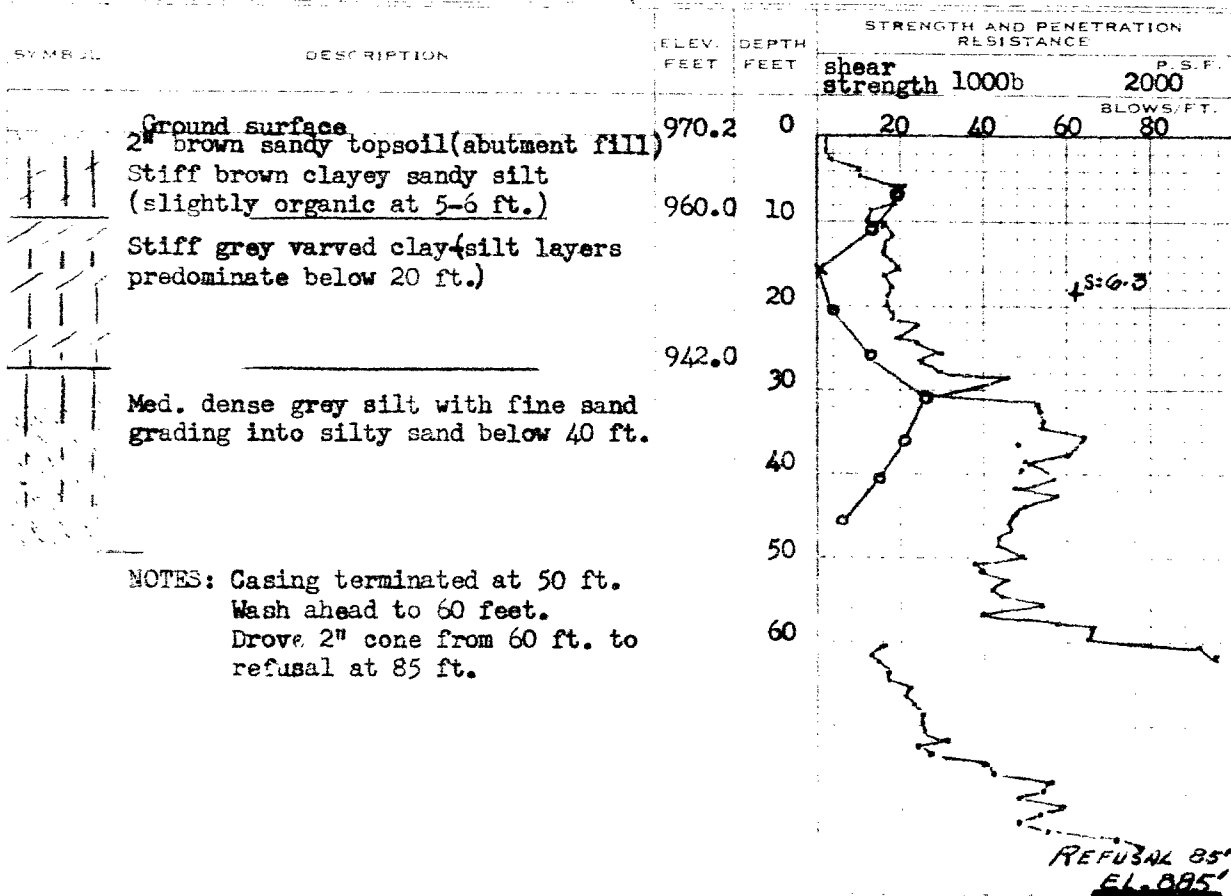
HOLE ELEVATION AND DATUM 970.2 BM Sta.40+82: 52'
Rt. - 976.4

BOREHOLE NO. 6

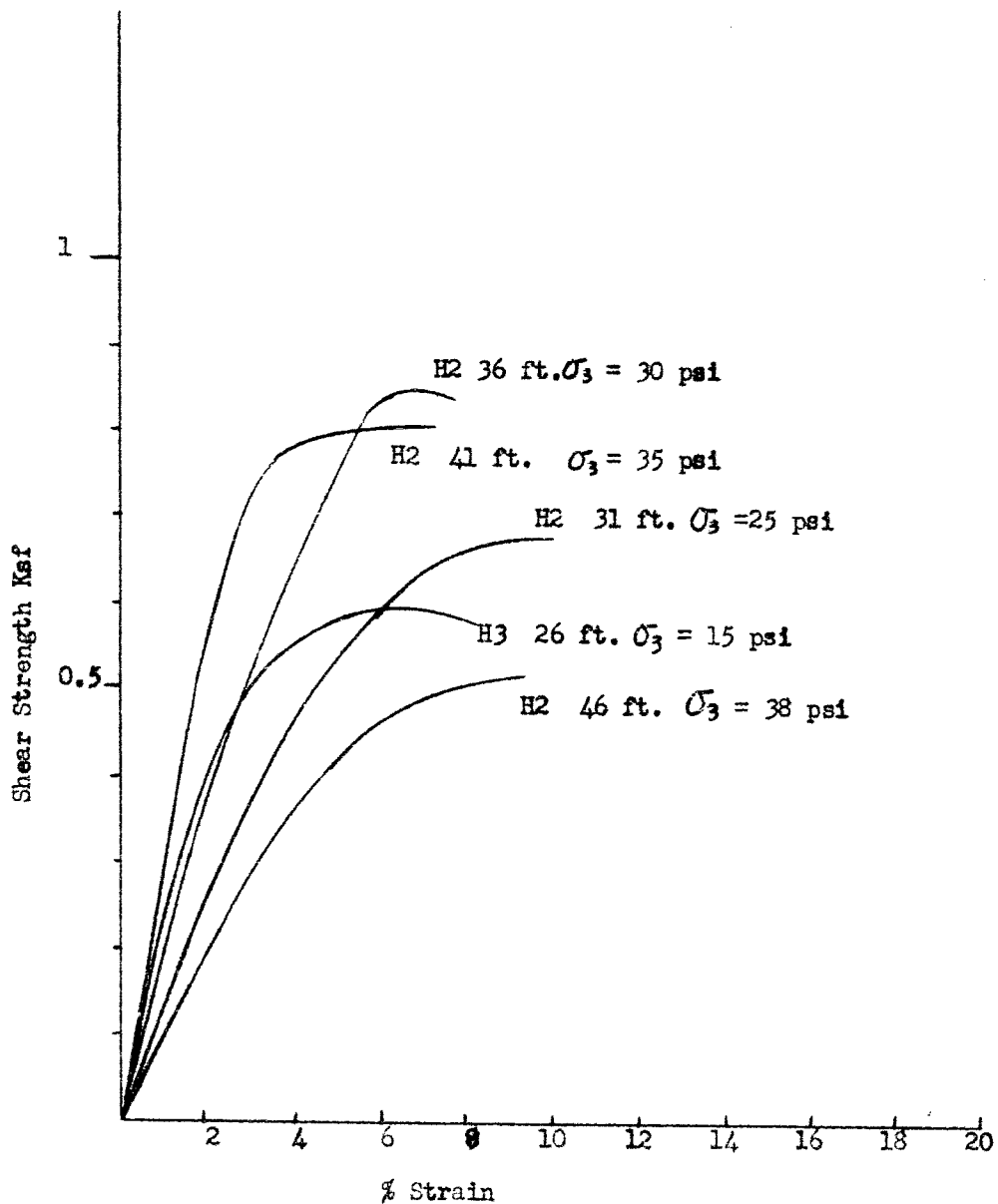
FIELD SUPERVISOR KP

DRILLER MC

PREP KP



CONSISTENCY		NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.		
		1
		2
		3
		4
		5
		6
		7
		8
		9



STRESS STRAIN CURVES FOR UNDRAINED TRIAXIAL TESTS