58-F-266C

W.P. # 164-57

HWY # 17

KAIBUSKONG

RIVER CROSSING

Mr. P. J. Harvey,
Location Plans Engineer.
Materials & Research Section.

April 18, 1958.

Re: W.P. [illegible] Hwy. 17, Kaibuskong River Crossing, Profile - 3865 , Sta. 193+00.

From a preliminary pedological survey, it was found that the soil in the immediate area of this structure consists of a varved clay, and in all varved clay areas, the soils conditions are a very important factor in determining the stability of a foundation site. However, the proposed fill is generally very shallow and, consequently, no stability problems regarding the approach fills are anticipated.

In this connection, we would agree that the grade line, as indicated, is satisfactory. We would, however, point out that before the design of the bridge is started, a complete foundation investigation be carried out to determine the best type of foundation applicable to this site.

F. C. Brownridge,
MATERIALS & RESEARCH ENGR.

Per:

(N. D. Smith)

NDS/MdeF

cc: Messrs. R. Surefield
S. McCombie
N. D. Smith
File

BP 73C

TROW, SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.SC., M.E.I.C., P.ENG.
L. G. SODERMAN, B.SC., D.I.C., P.ENG.

884 WILSON AVE.,
DOWNSVIEW, ONT.
ST. 8-5921

Project: C108/J201

May 5, 1958.

Mr. A. M. Toye,
Bridge Engineer,
Department of Highways of Ontario,
280 Davenport Road,
Toronto, Ont.

Attention: Mr. S. McCombie

Foundation Investigation
Kaibuskong River Crossing
Highway #17, District #13

Dear Sirs:

Attached hereto is our report on the results of a recently completed foundation investigation at the above site. Field work associated with this investigation was performed during the period April 17th to 22nd, 1958.

For your convenience the principal comments are repeated as follows:

(1) At a depth below existing ground surface not exceeding 6 feet at the abutment and pier locations a stratum of medium stiff varved clayey silt was encountered. The safe permissible bearing pressures for spread footings supported directly on this soil stratum is $1\frac{1}{4}$ to $1\frac{3}{4}$ tons/sq.ft.

(2) In view of the above limiting values of allowable spread footing pressures, footings supported on bearing piles driven to refusal at the underlying bedrock contact are recommended. A maximum pile length of 35 feet is noted for the east abutment location. Either high capacity steel bearing piles or timber piles appear suitable.

(3) Footings supporting the proposed centre pier will be subjected to flood water action. Protection against possible scouring should be provided either by carrying the footings below existing stream bed elevation or by providing perimeter sheet piling driven to below stream bed elevation.

(4) The subsoil underlying the approach embankments at this site is considered competent to support the proposed fill heights.

We are pleased to have had this opportunity to be of service to you and should any queries arise from your review of this report, please do not hesitate to call us.

Yours very truly,

L. G. Soderman

Lawrence G. Soderman (P. Eng.)

LGS/lt
Encl.

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

FOUNDATION INVESTIGATION
KAIBUSKONG RIVER CROSSING
HIGHWAY #17, DISTRICT #13

C108/J201 May 6th, 1958.

Trow Soderman & Associates

TABLE OF CONTENTS

ENCLOSURES

FOUNDATION INVESTIGATION
KAIBUSKONG RIVER CROSSING
HIGHWAY #17, DISTRICT #13

This report presents the results of a recently completed boring and sampling program at the above site. In addition to a detailed description of the type and occurrence of soil strata encountered, our recommendations pertaining to abutment and pier support are herewith submitted.

Site Location and Description

The Kaibuskong River crossing occurs at road centre line, chainage 192 + 80 on revision line B of King's Highway No.17, approximately 20 miles east of North Bay, Ontario. At this crossing point, the river is navigable for only very shallow draft crafts. Evidence of stream meander across the floor of the narrow river valley is provided by a flat flood plain on the east side of the existing stream and the proximity of the present water course to the west valley wall. The condition of the existing river banks indicates that the stream is not very active with respect to shoreline erosion or scour and fill.

Water levels in the area this year are reported, by local residents, to be the lowest ever witnessed. An elevation taken during this investigation shows river level to be 683.9; this is not appreciably lower than the low water level shown on D.H.O. Plan E-3316-1.

Field and Laboratory Work

Field work associated with this investigation was carried out during the period April 17th to 22nd, 1958. A total of 5 sampled borings and 5 dynamic cone penetration profiles were carried out at the locations and to depths shown on the attached profiles.

Boreholes were put down using a standard diamond drilling rig and conventional wash boring procedure. Samples were taken at depth intervals of 5 feet, or where a change in strata was noted during the driving and washing procedure. A continuous inspection of the wash water return was carried out between sample intervals.

In non-cohesive strata standard penetration tests were performed using a 2-inch diameter split spoon sampler. In the cohesive strata, i.e., the varved clayey silt deposits, relatively undisturbed samples were recovered using 2-inch diameter thin-walled Shelby tube samplers. Immediately upon recovery, these sample tube ends were wax sealed to prevent moisture change prior to laboratory testing.

As a supplement to the Standard Penetration Tests, a dynamic cone penetration resistance profile was established adjacent to each borehole location. In this field test a 2-inch diameter cone is fixed to the end of a length of "A" drilling rod. The rod with cone attached is then driven dynamically from surface to refusal depth using a driving energy equivalent to that specified for the Standard Penetration Test. The driving resistance recorded not only supplements the S.P.T. results but tends to give a more realistic indication of the in situ density of buried strata under artesian pressures.

In the cohesive materials in situ shear strength measurements were made using a four-bladed rotating vane. These tests were conducted at a minimum depth of 1½ feet below the bottom of the borehole at each test elevation.

Upon receipt in the laboratory all samples were carefully examined and identified. Representative undisturbed specimens of the cohesive materials were selected for strength tests in addition to routine index tests such as Atterberg Limits, natural moisture and unit weight determinations.

In varved clayey silt deposits, the shear strength is not generally independent of confining pressures and because of this, triaxial shear tests were carried out at confining pressures equal to existing total overburden pressures. These tests provide a measure of the apparent cohesion in terms of total stresses and this strength value can be used directly in bearing capacity computations. The in situ rotating vane tests gave values of apparent cohesion generally in agreement with the laboratory test results.

All phases of the field work was supervised by a qualified soils engineer who personally conducted all the field tests and prepared the field boring logs. Boreholes were located by chaining from centre line stations on revision line 'B' and elevations were obtained using hand sight level. The deck of the existing structure has been taken as Elev. 698.2 and used as a reference in establishing the elevations of the boreholes.

Discussion of Soil Types

Reference to the attached soils profile and borehole logs shows the following soil strata encountered at this bridge location.

Loose brown fine to coarse sand: At each borehole location the surface meadow mat was underlain by a shallow layer of fine to coarse sand. The thickness of this deposit varied from a minimum thickness of 4 feet at holes numbered 4 and 5, to a maximum thickness of 7 feet at hole number 1. Penetration tests indicate the material is in a loose state, not suitable for direct footing support.

Varved clayey silt: Immediately underlying the surface layer of loose sandy material, a stratum of varved clayey silt was intersected. The upper surface of this deposit corresponded to elevation 690 at the east abutment location and elevations varying between 690 and 694 at holes 3 and 4 at the west abutment position. The stratum was oxidized and brown in colour for very nearly its entire depth at the west abutment location, whilst at the east abutment location no zone of oxidation was encountered and the stratum was grey in colour throughout.

The varved deposit is characterized by alternate layers of high liquid limit, reddish brown fissured clay and layers of grey clayey silt to silt with very little clay size present. The thickness of each clay layer was of the order of ¼ of an inch; the silty layer thickness varied from ¾ of an inch to 1½ inches. Moisture content determination and Atterberg Limit tests were carried out on material typical of each layer as well as on a specimen as recovered from the Shelby tube sampler. Results indicate that the values of moisture, density reported for the entire sample are virtually those of the silty layer which predominates.

The properties of the oxidized zone of the varved deposit are not appreciably different from those of the grey non-oxidized zone. Strength values are, however, slightly higher due to the effect of dessication. Average stratum thicknesses at the east and west abutment locations have been determined as 16 feet and 12 feet respectively. At hole number 5 located approximately at mid-span, a thickness of 6 feet was intersected. Elevations of upper and lower horizons are noted on the borehole profile attached.

Engineering properties of this stratum have been determined as follows:

Natural unit weight = 118 p.c.f.
Natural moisture content = 37%

Total stress strength parameters

ϕ = angle of shearing resistance = 0°

For oxidized zone C = apparent cohesion = 1000 p.s.f.
Maximum Vane strength = 1750 p.s.f.

For non-oxidized zone C = 930 p.s.f.
Maximum Vane strength = 1080 p.s.f.

<u>Grey silty sand</u>: At each borehole location the previously described clayey silt deposit was underlain by a stratum of medium dense grey silty sand. Occasional lenses of gravel and small stones were encountered at random depths within this deposit. At the bedrock contact the percentage of sand and gravel size increased at each borehole location and this more pervious zone was found to be water bearing at hole number 5. The lower horizon of the silty sand layer corresponded to bedrock elevation at holes 1, 3 and 5. At holes 2 and 4, bedrock was not proven and refusal to sampling was obtained in the sand and gravel overlying bedrock.

<u>Bedrock</u>: AXT size core of the bedrock formation was recovered at holes numbered 2, 3 and 5. Elevations of rock contact at these holes have been noted as 666.0, 667.4 and 668.0. At the east abutment location, a difference in rock elevation of at least 6 feet was noted in borings 1 and 2.

The quality of the core, the percent recovery, and the frequency of blocking during drilling, indicate that the bedrock generally consists of sound non-fragmented igneous rock. At hole 3 however, a 2-foot zone of fragmentation was noted.

Bearing Capacity Considerations

In evaluating the safe bearing pressures for spread footings for abutments and piers, it has been assumed that the footings would be founded on or slightly above the stratum of varved clayey silt. A typical footing dimension of 6 x 32 has been assumed at the abutment locations. The average total stress shear strength parameters within a depth below the footing equal to its width have been given as:

$$C = 1000$$
$$\emptyset = 0$$

Natural density of the upper sand layer has been taken as 100 p.c.f. The safe bearing capacity for spread footings is expressed as follows:

$$qa = \frac{C\ Nc}{F} + P_o$$

where qa = allowable footing pressure in Tons/sq.ft.
C = apparent cohesion in terms of total stresses
Nc = bearing capacity factor dependent upon shape and depth of footing = 6.5
D = depth of footing
F = safety factor on shear strength = 3
P_o = existing total overburden pressure = γ D

substitution for qa value gives

$$qa \simeq 1\tfrac{1}{4} \text{ tons/sq.ft.}$$

At the west abutment area the bearing capacity is slightly higher but not in excess of $1\frac{3}{4}$ tons/sq.ft.

Pile Support Consideration

The relatively shallow depth to ultimate refusal for bearing piles at this bridge site indicates that this type of footing support is the least problematical. Either large displacement timber piles or higher capacity steel piling appears suitable. A maximum pile length not greater than 35 feet will be required, assuming cut off elevation at least 6 feet below existing ground elevation at the abutment locations. At the location of borehole number 5, bedrock was proven at a depth of 17 feet below existing ground surface.

Approach Embankment Stability

Reference to D.H.O. drawing No. E-3316-1 indicates a maximum embankment height of the order of 10 feet occurring at the west approach. This value of embankment loading is approximately 1/3 of the safe bearing capacity of the foundation material and embankment instability due to foundation failure is considered impossible.

Summary

(1) At a depth below existing ground surface not exceeding 6 feet at the abutment and pier locations, a stratum of medium stiff varved clayey silt was encountered. The safe permissible bearing pressures for spread footings supported directly on this soil stratum is $1\frac{1}{4}$ to $1\frac{3}{4}$ tons/sq.ft.

(2) In view of the above limiting values of allowable spread footing pressures, footings supported on bearing piles driven to refusal at the underlying bedrock contact are recommended. A maximum pile length of 35 feet is noted for the east abutment location. Either high capacity steel bearing piles or timber piles appear suitable.

(3) Footings supporting the proposed centre pier will be subjected to flood water action. Protection against possible scouring should be provided either by carrying the footings below existing stream bed elevation or by providing perimeter sheet piling driven to below stream bed elevation.

(4) The subsoil underlying the approach embankments at this site is considered competent to support the proposed fill heights.

L G Soderman

Lawrence G. Soderman (P. Eng.)

LGS/lt
C108/J201
May 6, 1958.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
L. G. SOD
PROVINCE OF ONTARIO

TABLE NO.1

SUMMARY OF LABORATORY & FIELD TESTS

Hole No.	Sample No.	Sample Depth feet	Material Description	Mois. %	Shear Strength p. s. f. Field Vane	Shear Strength p. s. f. Lab. Triax.	Unit Wt. p.c.f.	Liquid Lt. %	Plastic Limit %	P.I.	N blows per foot
1	SS1	5- 6.5	Brown fine to coarse sand								5
1	SS2	10-11.5	Grey varved clayey silt		960						2
1	TW3	15-16.5	" "	37.3	1080	1380	119.0	32.0	24.0	12.0	pushed
1	TW4	20-21.5	" "	36.2	680		118.5				pushed
1	TW5	25-26.5	Grey fine sandy silt								pushed
1	SS6	30-31.5	Grey very fine sand								11
1	SS7	36-37	Red to brown med. sand								refusal
2	SS1	5- 6.5	Brown med.to coarse sand								
2	TW2	10-11.5	Grey varved clayey silt	36.0	920	948	119.0				pushed
2	TW3	15-16.5	" "	36.5	1000	685	118.0	34.0	23.6	11.4	pushed
2	TW4	20-21.5	Grey very fine sand & silt								pushed
2	SS5	25-26.5	Grey very fine sand & silt								10
3	SS1	5- 6.5	Brown med. to coarse sand								6
3	TW2	10-11.5	Brown varved clayey silt	39.0		990	119.0				pushed
3	TW3	15-16.5	Grey very fine sand and silt	29.0							pushed
3	SS4	20-21.5	Grey very fine sand and silt								7
3	SS5	25-26.5	Brown fine to coarse sand								23
4	TW1	5- 6.5	Brown sand, some brown silty clay								pushed
4	SS1A	6-7	Brown varved clayey silt		1360						pushed
4	TW2	10-11.5	Brown varved clayey silt	39.8	1760		118				pushed
4	TW3	15-16.5	Grey varved clayey silt	33.0			117				pushed
4	SS4	20-21.5	Grey very fine sand and silt								10
4	SS5	25-26.5	Grey silt, fine sand and stones								refusal

TABLE NO.1 - Page 2

Hole No.	Sample No.	Sample Depth feet	Material	Description	Mois. %	Shear Strength Field Vane (p. s. f.)	Shear Strength Lab. Triax. (p. s. f.)	Unit Wt. p.c.f.	Liquid Limit %	Plastic Limit %	P.I.	N blows per foot
5	TW1	5- 6.5	Fine to med. sand, grey clayey silt									pushed
	TW2	7- 8.5	Grey varved clayey silt		34.6			118.5				pushed
	TW3	10-11.5	Grey silt and fine sand									pushed
	SS4	15-16.5	Grey coarse silt.									refusal

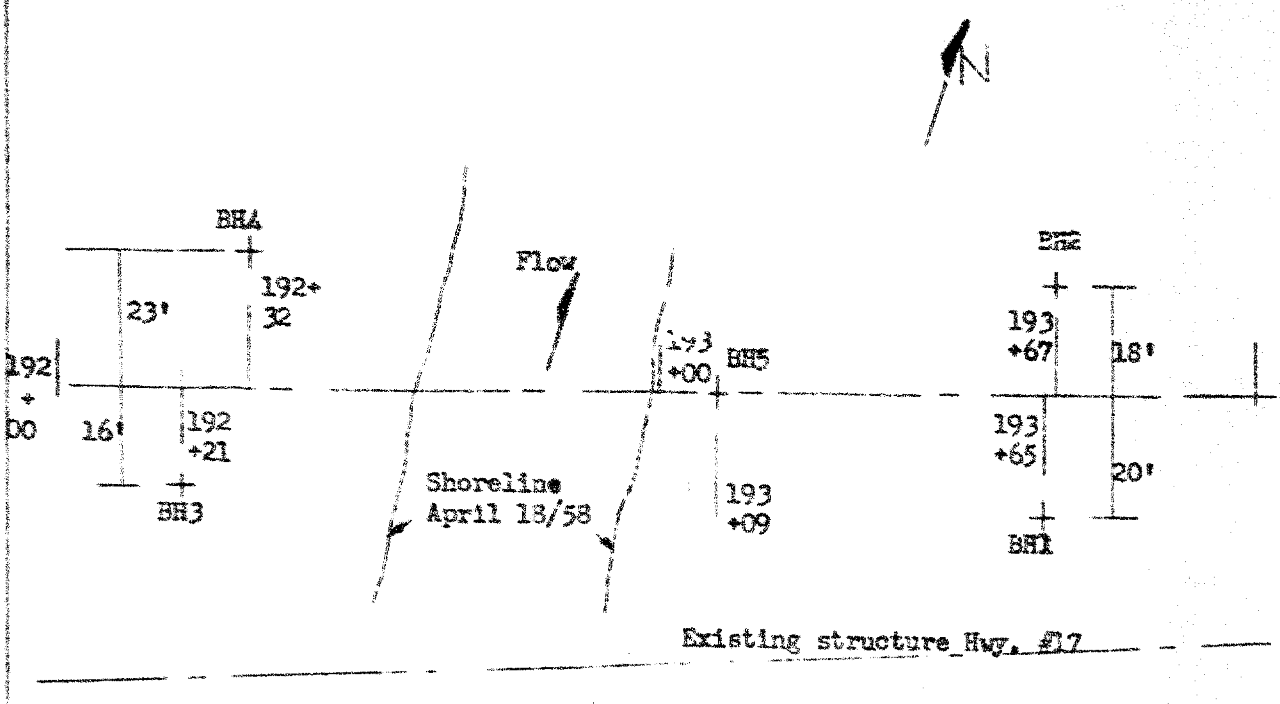
Laboratory shear strengths determined from undrained triaxial tests carried out at cell pressures equivalent to existing total overburden pressure.

The varved clay material consists of typically 1" silt seams and 1/8" brown to red clay seams.

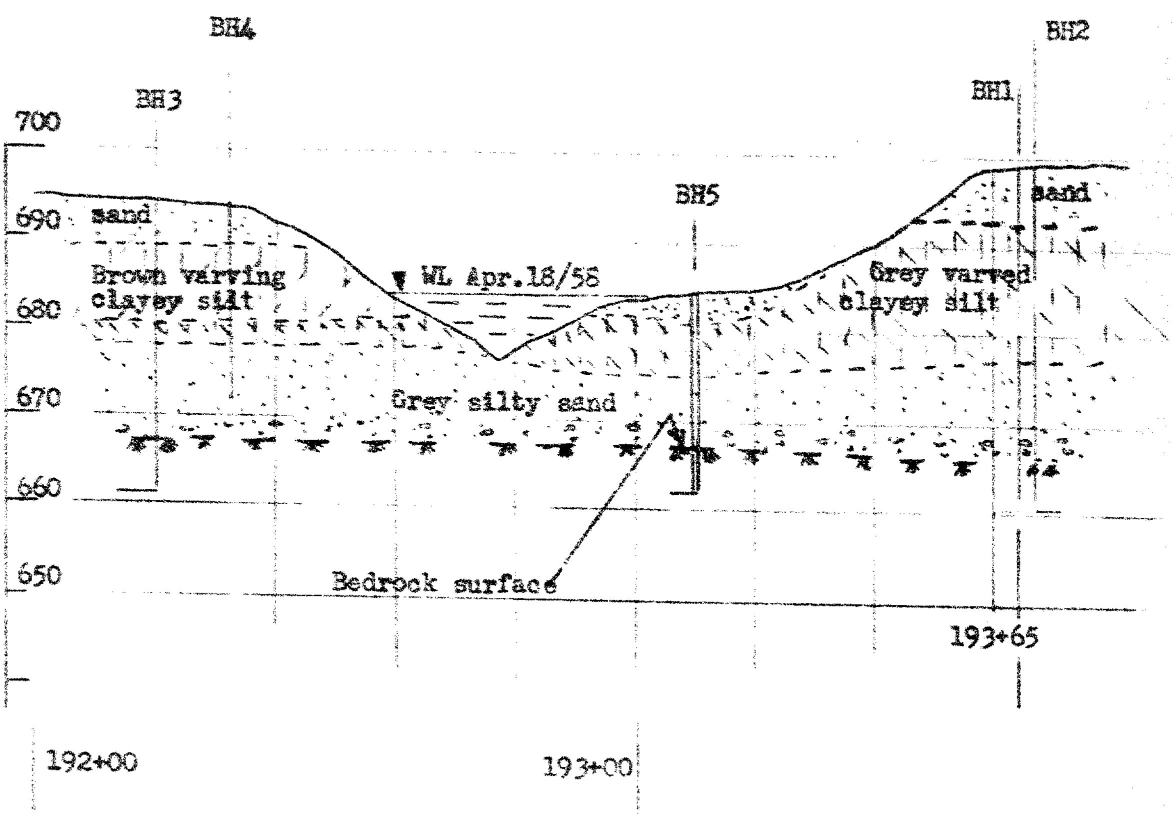
For clay fraction: ω = 52%; Liquid Limit = 75%; Plastic Limit = 21%

For silt fraction: ω = 35%; Liquid Limit = 34%; Plastic Limit = 22%

PROJECT NO. C108 J201 DRAWING NO. 1



BOREHOLE LOCATION PLAN
KAIBUSKONG RIVER



Scale: 1" = 30' horiz.
1" = 20' vert.

PROJECTED PROFILE ALONG CENTRE LINE

TROW SODERMAN & ASSOCIATES

C108/J201

DRAWING NO. 2

TROW SODERMAN AND ASSOCIATES

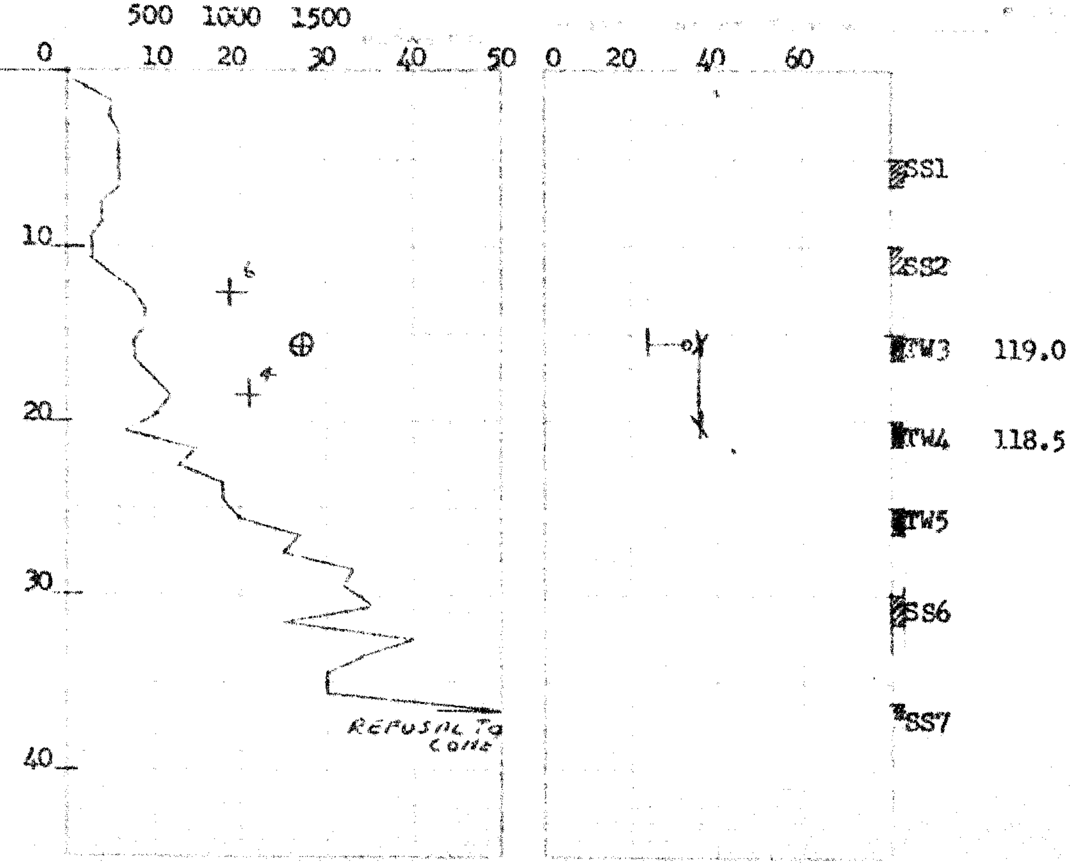
Kaibuskong River Crossing
Highway 17, District 13
As on plan

Elev. referenced to bridge deck 698.2

BOREHOLE NO. 1

KP
MG
LS

Ground surface	697.3
Loose fine to coarse sand with occasional stone. A little brown sandy clay throughout	690.3
Grey medium stiff varved clayey silt	W.T. 689.3
	671.3
Grey very fine sandy silt.	661.3
Br.fine to med.sand & gravel	660.5
Refusal to cone and sampling at 36.8, assumed bedrock contact.	



C108/J201

DRAWING NO 3

TROW SODERMAN AND ASSOCIATES

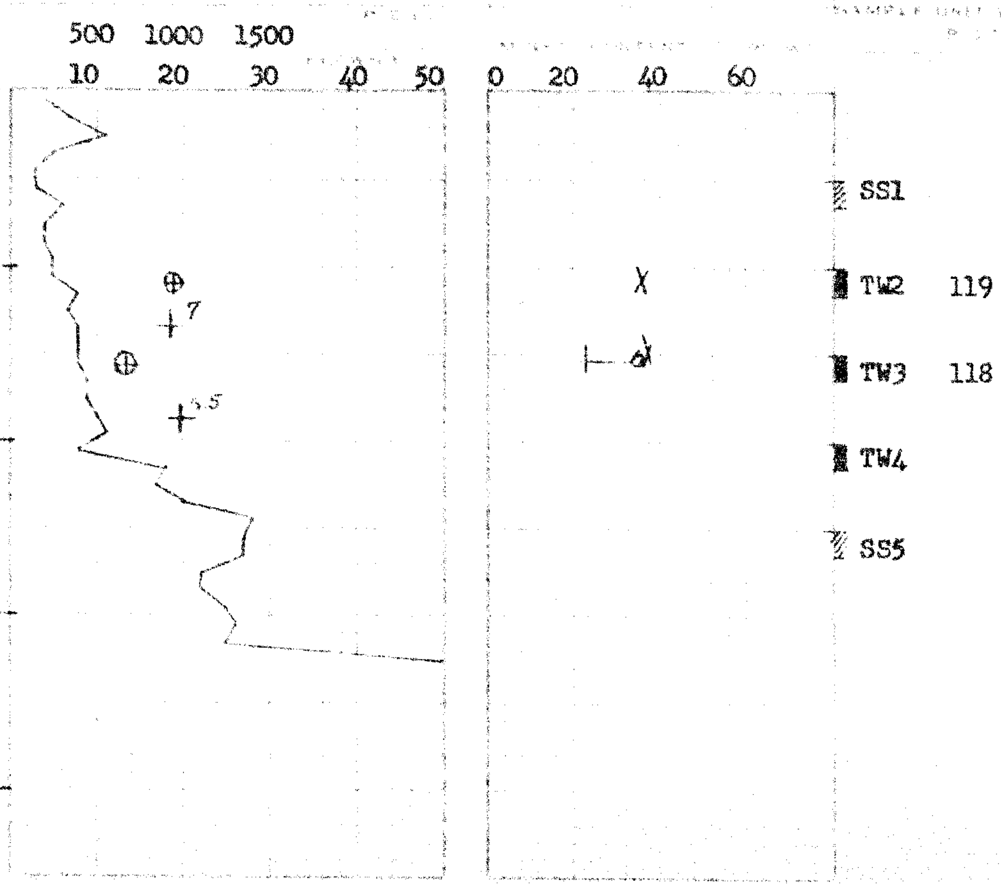
SITE INVESTIGATION AND SOIL MECHANICS CONSULTATION

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

Kaibuskong River Crossing
Highway #17 District #13
As on plan
Referenced to deck of existing bridge at elev.698.2

BOREHOLE NO. 2
FIELD SUPERVISOR KP
DRILLER MG
PREP LS

Description	Elevation
Ground surface	697.0
Loose med. to coarse brown sand. Probable fill.	690.5
Grey medium stiff varved clayey silt	WT 688.0
	676.5
Medium de~~ grey silty fine sand. Sand becomes coarse at bedrock surface.	
Bedrock surface	666.0
Pink granite bedrock. Core recovery 100%	
End of hole	



PROJECT NO. C108/J201

DRAWING NO. 4

TROW SODERMAN AND ASSOCIATES

INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

LEGEND

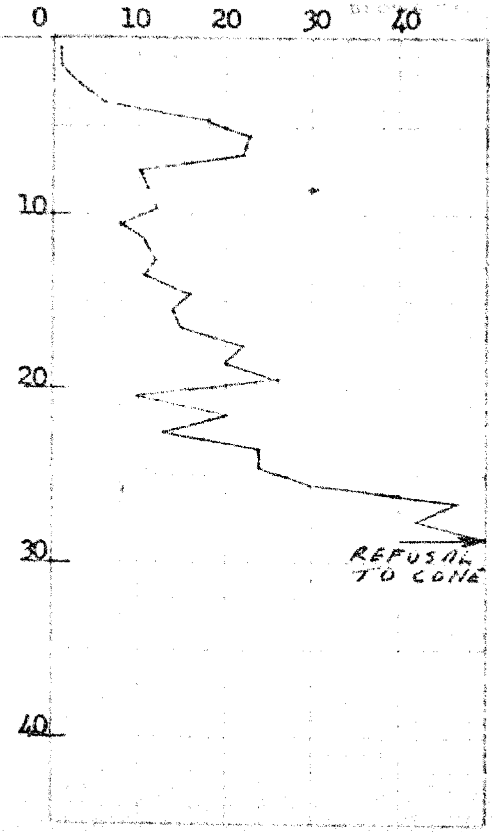
A. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

Kaibuskong River Crossing
Hwy. #17, District 13
As on plan
Referenced to deck of existing bridge at El.698.2

BOREHOLE NO. 3
FIELD SUPERVISOR KP
DRILLER MG
PREP. LS

	Elev.
Ground surface	696.4
Brown fine to coarse sand	
	690.0
Brown medium stiff varved clayey silt	
	682.0
Grey var... clayey silt	680.5
Grey very fine sandy silt	
	670.4
Fine to coarse brown sand & gravel	
Bedrock surface	667.4
Pink granite bedrock. Rock badly fractured for top 2 feet. Core recovery 40%	662.4
End of hole	

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY

NATURAL UNIT WT

SS1
TW2
TW#3
SS4
SS5

PROJECT NO. C108/J201

DRAWING NO. 5

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

LEGEND

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

PROJECT Kaibuskong River Crossing

LOCATION Highway #17, District 13

HOLE LOCATION As on plan

HOLE ELEVATION AND DATUM Referenced to deck of existing bridge at elev.698.2

BOREHOLE NO. 4

FIELD SUPERVISOR KP

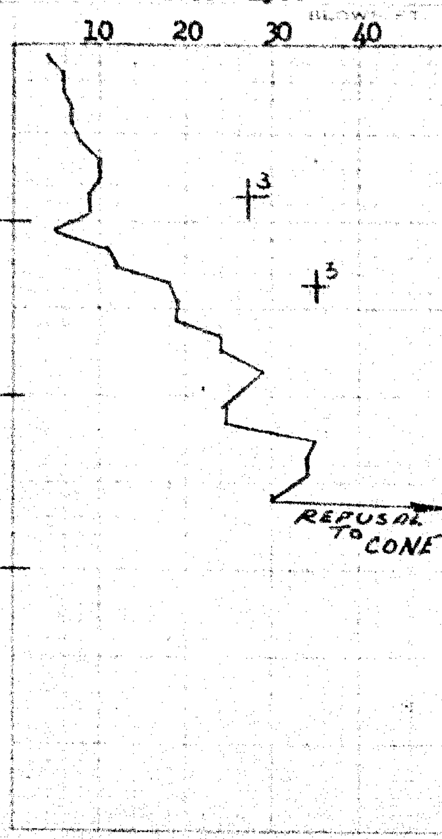
DRILLER MG

PREP LS

SAMPLE	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Ground surface	698.0	0
	Brown sand with some stiff silty clay	694.0	
	Medium stiff brown varved clayey silt		10
		684.0	
	Grey varved clayey silt	682.0	
	Grey very fine sandy silt. Gravel & stones at end of hole		20
	End of borehole	671.5	30
			40

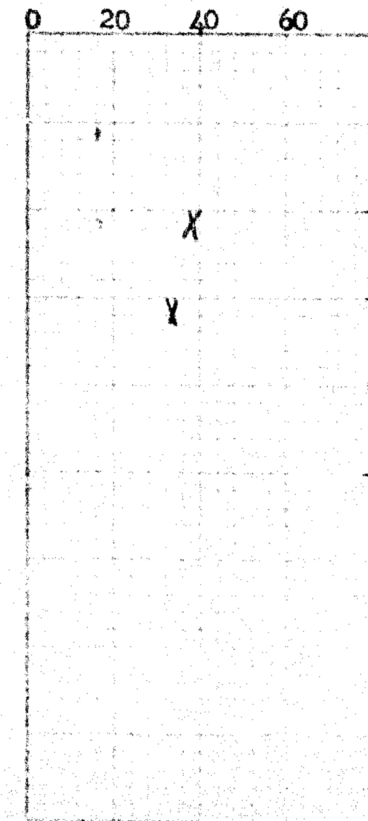
Refusal to cone penetration and sampling obtained at 26.5 feet below surface. Probable bedrock

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY

MOIST. CONTENT % DRY WT.



SAMPLE
TW1 lost
SS1A
TW2
TW3
SS4
SS5

NATURAL UNIT WT. P.C.F.

PROJECT NO. C108/J201

DRAWING NO. 6

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

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 Kaibuskong River Crossing
LOCATION Highway #17, Distr ct 13
HOLE LOCATION As on plan
HOLE ELEVATION AND DATUM Referenced to deck of existing bridge at elev.698.2

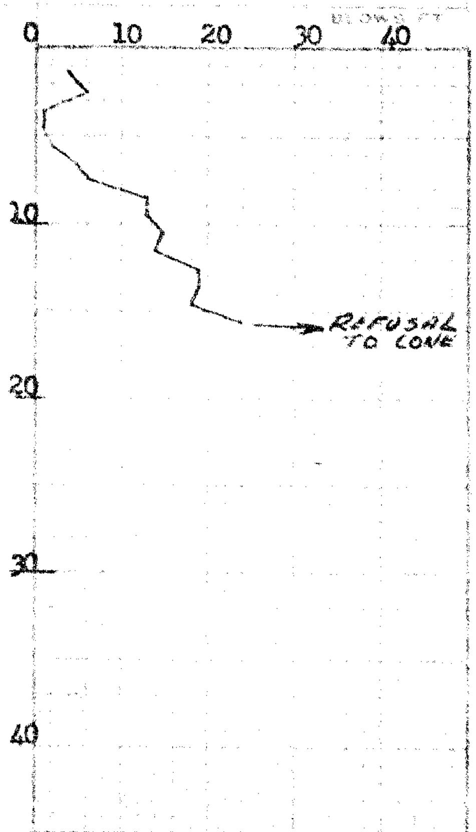
BOREHOLE NO. 5
FIELD SUPERVISOR KP
DRILLER MG
PREP. LS

DESCRIPTION	ELEV. FEET
Ground surface	685.0
Fine to med.grey sand with some clayey sand	WT 684.0 / 681.0
Grey varved clayey silt	675.0
Grey very fine sandy silt. Gravel noted above bedrock contact	
Bedrock contact	668.0
Pink granite. Core recovery 100%	
End of hole	663.0

Water rose to elev. 688.0 with casing at top of bedrock. Artesian condition in layer of sand at the contact zone. Hole sealed at depth of 10 ft. below surface.

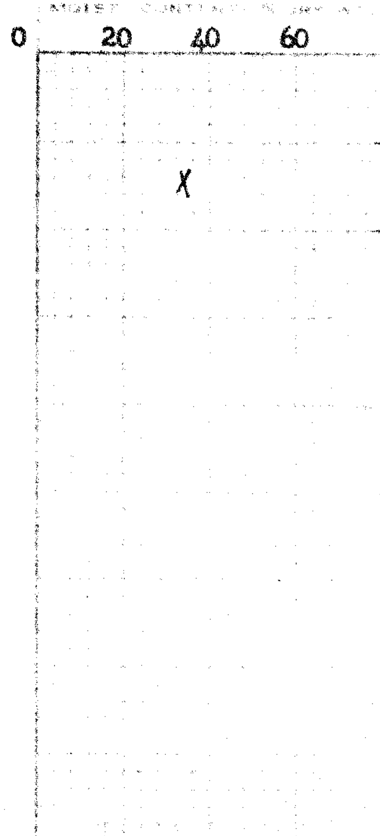
STRENGTH AND PENETRATION RESISTANCE

BLOWS/FT.



CONSISTENCY

MOIST. CONTENT [illegible]



SAMPLE	NATURAL UNIT WT.
TW1	
TW2	118.5
TW3	
SS4	

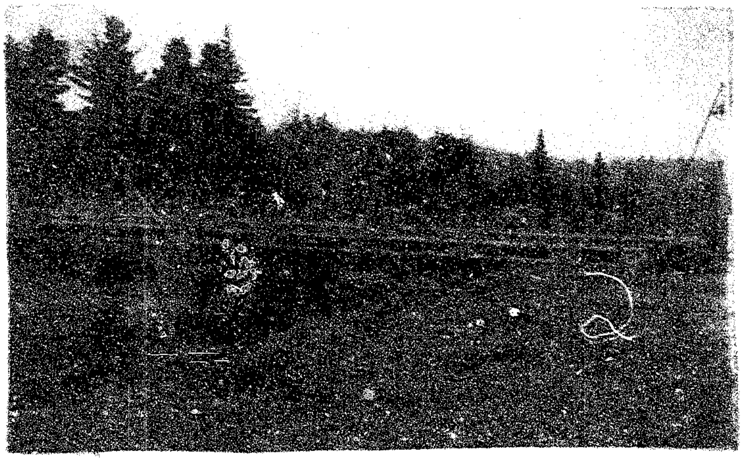


Photo taken facing upstream, showing existing structure. Drilling rig is set up over Hole No. 4

Photo taken standing on road centre line facing east abutment and pier location.

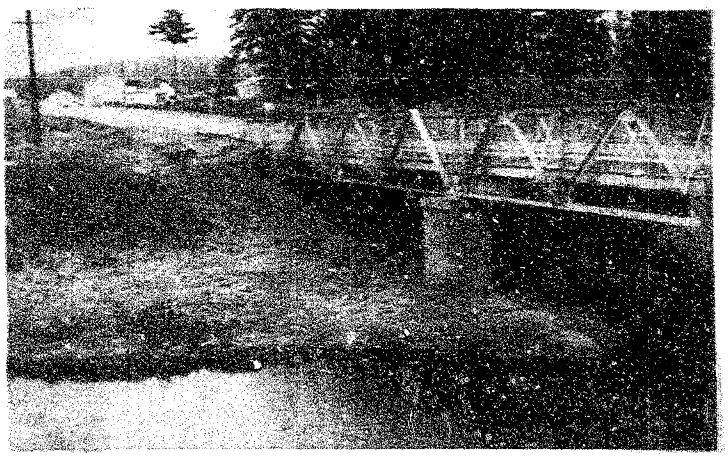


Photo taken facing upstream, showing existing structure. Drilling rig is set up over Hole No. 4

Photo taken standing on road centre line facing east abutment and pier location.