

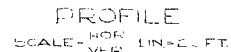
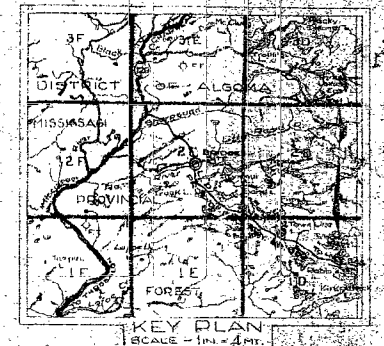
#65-F-259

W.P. # 19-65

Hwy. # 639  
RAPID RIVER  
PROP. CROSSING

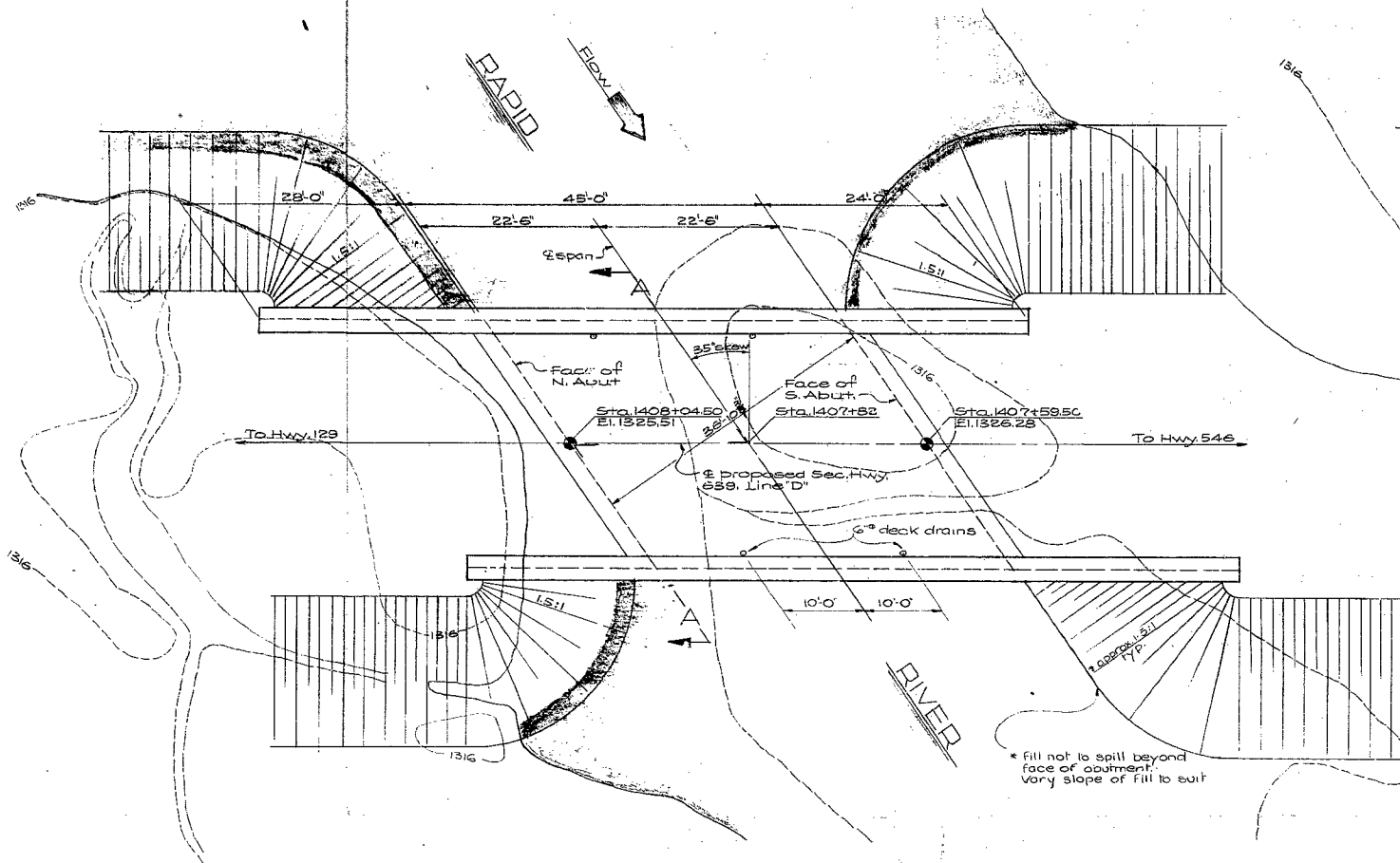


TO CONDITION OF ORIGINAL DOCUMENTS



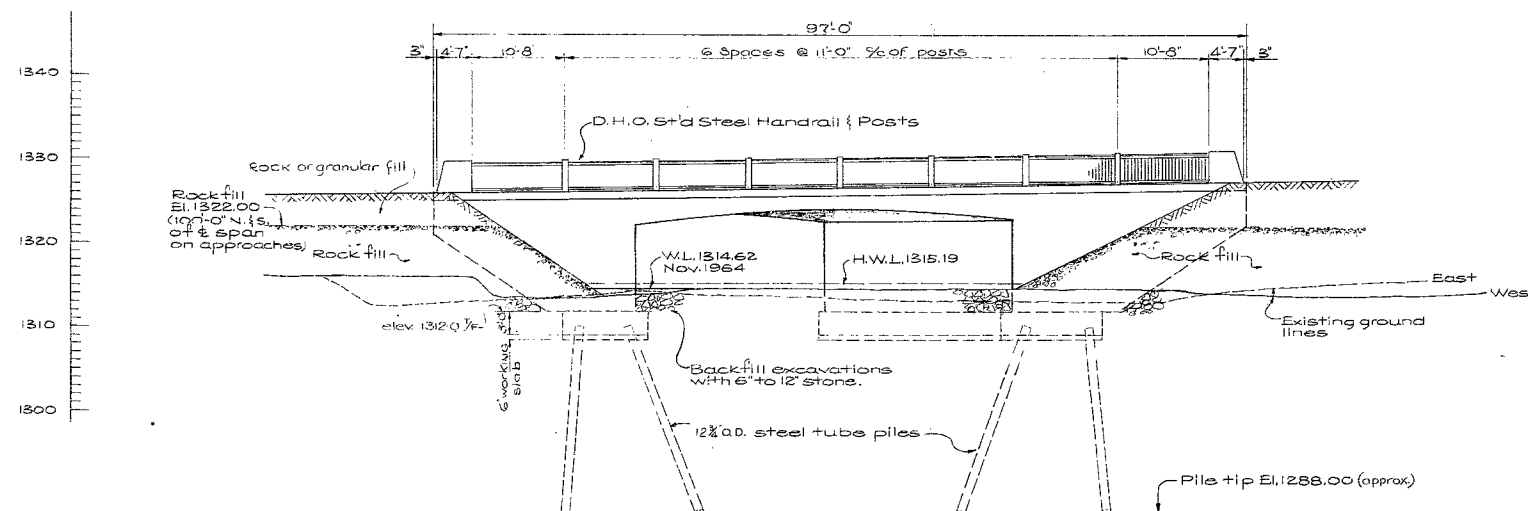
DATE	REVISIONS	ADDITIONS	BY	CHECKED

SCALE	DISTRICT N 215	REGION
AS SHOWN	EAST STE MARIE	NORTH-WEST (21)
DATE FORWARDED	DATE - OCT 1964	CITENS.
	DATE NOV 1964	
SURVEY BY		DIVISION BY
Chief of Party - <u>W. J. GIERZ</u>	Co-Chief - <u>W. G. GIERZ</u>	
Co-Chief - <u>P. A. GIERZ</u>	Co-Chief - <u>W. G. GIERZ</u>	
CHECKED BY	PLAN	
Definition	N E - 4530-1	



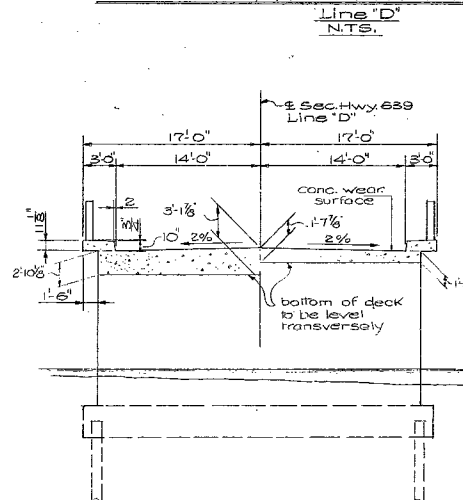
PLAN  
1"=10'-0"

Functions of 35° Skew	
Sin.	0.57358
Cos.	0.81915
Tan.	0.70021
Sec.	1.22077



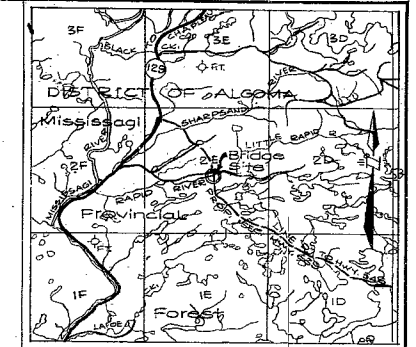
ELEVATION  
1"=10'-0"

PROFILE AT CROWN OF HWY. 639



A-A  
1"=10'-0"

Reference Plans	
Bridge Site Plan	E-4530-1
Location Plan	B-959-2
Profile	C-959-3
Soils	BA-2091



KEY PLAN  
1"=4 MI.

B.M. El. 1330.05  
Geodetic Datum  
N. & W. in top of 0.5' spruce stp.  
105.0' Lt. of Sta. 1405+02.

Found.

DEPARTMENT OF HIGHWAYS, ONTARIO BRIDGE DIVISION			
RAPID RIVER BRIDGE 4.0 Miles East of Hwy. 129			
Sec. 116'S HIGHWAY No. 639	DIST. No. 18		
Dist. of Algoma	TWP.	LOT	CON.
PRELIMINARY			
APPROVED	BRIDGE ENGINEER	SITE No. 385-278	W.P. No. 18-65
DESIGN	CHECK	CONTRACT	
DRAWING	CHECK	No.	
DATE	LOADING	No.	
OCT. 1965	H20S16		D-5752-PI



MR. A. RUTKA, P.ENG.  
CHIEF MATERIALS AND TESTING ENGINEER  
DEPARTMENT OF HIGHWAYS OF ONTARIO  
PARLIAMENT BUILDINGS, TORONTO, ONTARIO

FOUNDATION INVESTIGATION  
PROPOSED CROSSING RAPID RIVER  
HWY. NO. 639 W.P. 19-65

Project: J1758

William Trow Associates Limited

June, 1965

90 Milvan Drive  
Weston, Ontario  
749-1260

**William Trow**

Project: J1758

Soil Mechanics  
Consultants  
W. A. Trow  
MSc. MEIC. P. Eng.  
K. Peaker  
PhD. MEIC. P. Eng.  
D. H. Shields  
PhD. MEIC. P. Eng.



**Associates Ltd.**

Mr. A. Rutka, P.Eng.,  
Chief Materials and Testing Engineer,  
Materials and Testing Section,  
Department of Highways of Ontario,  
Parliament Buildings,  
Toronto, Ontario.

May 31, 1965

Attention: Mr. K.Y. Lo, P.Eng.

Foundation Investigation  
Proposed Crossing Rapid River  
Hwy. No. 639 W.P. 19-65

Dear Sirs:

Following your authorization of December 17th, 1964,  
we have completed a foundation study at the above site.  
The field work associated with this project was carried out  
during the period April 2 to 25.

Because of adverse weather and river conditions the  
field work was, of necessity, reduced to that outlined in  
the report. Final borehole elevations were obtained on  
May 15 permitting completion of this report.

Our findings and recommendations are as follows.

SUMMARY

1) The subsoil at this site consists of approximately  
45 feet of sand and gravel over a variable depth of sand gravel  
and boulders followed by bedrock.



2) Foundations for the proposed bridge can be simple spread footings designed using a safe net bearing value of  $2\frac{1}{2}$  tsf and founded at El  $\pm$  1309 feet. A piled foundation using displacement piles as outlined in the body of the report is an alternate type of foundation.

3) Excavations for spread footings will be well below river level and must follow one of the procedures outlined in the body of the report. Comments on scour protection and earth pressures have been included.

#### FIELD WORK AND SUBSOIL STRATIGRAPHY

The proposed field program at this site consisted of two boreholes at each abutment location. Because of rising water this program was reduced to two borings and two dynamic cone penetration probes. Site conditions during the period of the field work were extremely poor thus the relatively long period of time required to complete this project.

The location of the boreholes and dynamic cones are shown on the site plan drawing. A detailed description of the subsoil is included on the borehole logs, Dwgs. 1 to 3 and in summary form on the site plan drawing. The elevations of the boreholes are referenced to the D.H.O. bench mark shown on this site plan drawing.



At this site the subsoil was found to consist of about 45 feet of sand and gravel followed by sand gravel and boulders and bedrock. The sand and gravel contains some organic material in the upper four feet and exists in a dense state. The stratum of sand gravel and boulders consists essentially of a boulder bed with a sand and gravel matrix. Bedrock has been identified as a grey granite gneiss. This material is identical to the bedrock outcrops noted near the site. Core from the boulders immediately above the bedrock identify the boulders as a pink granite thus the identification of bedrock was easily established from the colour of the rock as well as from the performance of the drill and the rock recovery.

#### FOUNDATIONS

Two alternative foundation types - spread footings or piled foundation, are possible at this site. A discussion of each type follows.

a) Spread Footings: The proposed structure may be founded on spread footings placed below river scour level. For footings placed at sufficient depth to provide scour protection a safe net bearing value of  $2\frac{1}{2}$  tsf may be used for design purposes. This design value of  $2\frac{1}{2}$  tsf has been obtained after considering the relationship between the subsoil density measured by the resistance to penetration of the split spoon sampler and the dynamic cone penetration values.



Foundations should be founded at or below El 1309 or some 6 to 7 feet from present ground surface at the borehole location. The maximum depth of scour at this site has been estimated as 6 feet from ground surface near the boreholes. River bed level was El 1314 feet approximately at this location.

Settlements resulting from the suggested designed loading will be negligible. The minor settlement that does occur will be elastic in nature and will be complete at the end of construction.

Foundation excavations below the water table in porous sand and gravel will be necessary. Before any attempt is made to dewater the area, the footing excavation including perimeter drainage ditches should be made. The base of the excavation should be crowned, with the footing area being the highest point of the cut and with the ground sloped down beyond the edge of the footing at 3 horizontal to 1 vertical. The material which is removed should be placed on the river side of the excavation to form a dyke. In this manner the creek can be diverted around the excavation. The excavation can then be pumped out gradually. Continuous pumping will be required to keep the water table depressed in the working area.

The foregoing proposal will require an excavation which is much larger than that which would normally be needed for a bridge footing. In addition, because the soil is quite permeable, a considerable inflow of water can be expected. Provided the pumping equipment is adequate, the volume of seepage should gradually reduce with continued pumping. The flow can be minimized by directing the river



well clear of the work. Since the sand and gravel is quite well graded (Dwg. 4), it is expected to remain stable as the dewatering program progresses, provided that the perimeter walls of the excavation are sloped at approximately 2 horizontal to 1 vertical. A sketch of the suggested shape of the excavation and an estimate of seepage flow is given in Dwg. 5.

An alternative to this dewatering method is one incorporating the use of well points and a sheeted excavation. If well points are impractical in this isolated area, however, another alternative would be to drive water tight sheeting well below footing elevation. The depth to which sheeting should be driven to obtain a factor of safety of 2 against piping can be obtained from the following table.\*\*

$W/H$	D	F.S. = 2.0
0.5	1.75 H	
1.0	1.4 H	
2.0	1.0 H	
3.0	0.8 H	
4.0	0.75 H	
5.0	0.7 H	
10.0	0.6 H	
2.0	0.52 H	

The sheeting, if left in place, will provide excellent protection against scour.

\*\*McNamee, J., - 'Seepage Into a Sheeted Excavation,' - Geotechnique, Volume III, 1952 - 13.



b) Pile Foundation: If piles are used to support the abutments it is recommended that treated wood or 12 inch diameter concrete filled tube type steel piles be selected. For this type of pile the following safe loading is suggested.

PILE	ELEVATION PILE TIP		SAFE LOAD
3" Diameter Tip (wood pile)	1298	15' pile length below stream bed level	20 tons
12" Diameter Steel Tube Type	1293	20' below stream bed level	60 tons

These suggested loadings and pile tip elevations have been estimated from the expression as follows, and consider the structural capacity of the piles.

$$Q = A\gamma^* D N + \frac{p k \gamma^* D^2}{2}$$

- where:
- Q = the required ultimate pile capacity
  - A = the area of the pile tip, assumed to be 0.35 square feet for an 8 inch diameter pile tip, and 0.8 for a 12" diameter pile
  - $\gamma^*$  = the submerged unit weight of the sand and gravel, assumed to be 65 pcf
  - N = the bearing capacity factor, estimated to be at least 300 for the soil density applicable
  - k = the resultant friction coefficient on the shaft, estimated to be 1.0
  - p = the perimeter of the pile, assumed to be 2.6 feet for a timber pile having an average diameter of 10 inches, and 3.14 for a 12" diameter tube type pile



D = the required embedded depth of pile  
below stream bed level which will  
give an ultimate bearing capacity  
of 60 tons for a wood pile and 180  
tons for a steel tube type pile

These piles will not meet refusal at the suggested elevations. Because of the fact that in this isolated location pile load tests are not an economical check on the theoretical pile lengths, it is considered prudent to drive the piles an extra five feet into the sand and gravel.

#### SCOUR PROTECTION

Regardless of the type of foundation selected suitable scour protection must be provided at ground surface level, up and down stream of the abutments. This protection must be placed over the area from high to low water level and 30 feet up and down stream of each abutment. Suitable scour protection will be achieved using hand placed rip-rap.

#### APPROACH EMBANKMENTS

No problem associated with the stability of approach embankments exists at this site.

#### EARTH PRESSURES

If abutments and wing walls are used on this project, i.e., the approach fill does not spill through the abutments, they must be designed to withstand the lateral earth pressure



exerted by the retained soils. The earth pressure that will act on the walls can be estimated using a value of the earth pressure coefficient equal to 0.35. The earth pressure,  $p$ , on the walls at any depth,  $h$ , can be found from the expression:

$$p = K \{ \gamma (h - h_1) + \gamma_s h_1 + q \}$$

where:  $K = 0.35$ , the recommended earth pressure coefficient assuming the walls to be rigid

$\gamma = 125$  pcf, the estimated unit weight of the retained soil

$\gamma_s = 60$  pcf, the estimated submerged weight of the retained soil

$h_1$  = height of water table above the point being considered

$q$  = surcharge, if any, acting at the top of the wall

The stability of the abutment and wing walls should be checked for horizontal sliding along the footing base. The resistance against the sliding is the frictional force acting along the footing base. The frictional force developed along the footing base can be calculated using a friction coefficient of 0.7 (concrete sliding on granular soils).

If the resisting force is less than  $1\frac{1}{2}$  times the estimated sliding force, the footing base can be extended under the fill to increase the weight of backfill carried



by it. In this manner, the resistance to sliding can be increased.

If we can be of any service in explaining or amplifying our comments please do not hesitate to contact this office.

Yours very truly,

*K. Peaker.*

K.R. Peaker, P.Eng.

KRP/bs.  
Encls.

# WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

LEGEND

DRAWING NO. 11750  
PROJECT NO. 11750

BOREHOLE NO. 1  
PROJECT: Proposed Bridge, off River  
LOCATION: Highway 10, 300 ft.  
HOLE LOCATION: See Site Plan  
HOLE ELEVATION: 1316.2 ft.  
DATUM: See Site Plan

## 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)

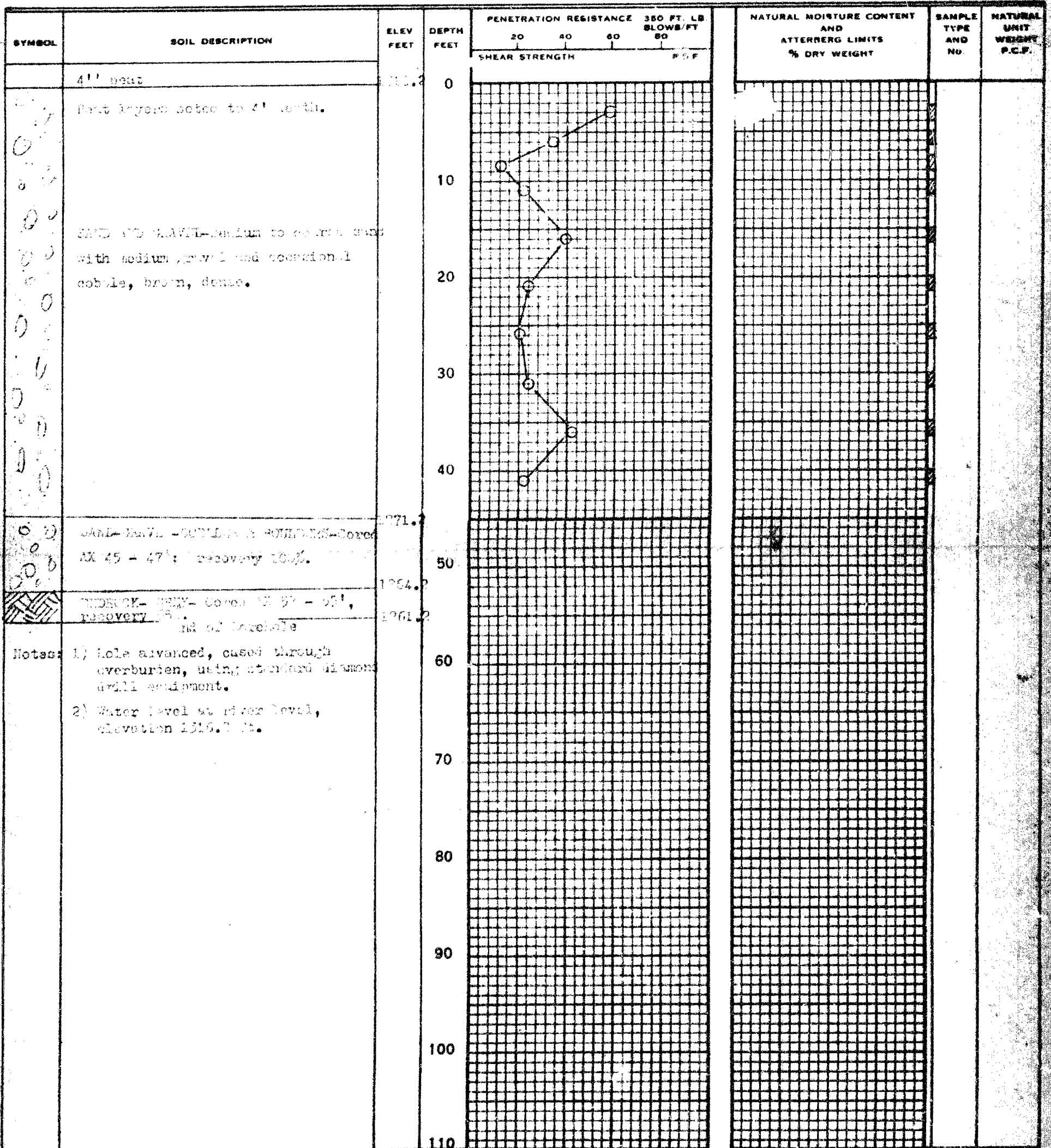
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

## ATTERBERG LIMITS

LIQUID LIMIT   
PLASTIC LIMIT

## SAMPLE TYPE

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



DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

# WILLIAM A. TROW & ASSOCIATES LTD.




SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

## LEGEND




DRAWING NO. 2  
PROJECT NO. 1070

BOREHOLE NO. Cone Test 2  
PROJECT Proposed Bridge, Rapid River  
LOCATION Highway No. 63  
HOLE LOCATION See Site Plan Map  
HOLE ELEVATION 1316.0 ft.  
DATUM See Site Plan Map

### 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) 

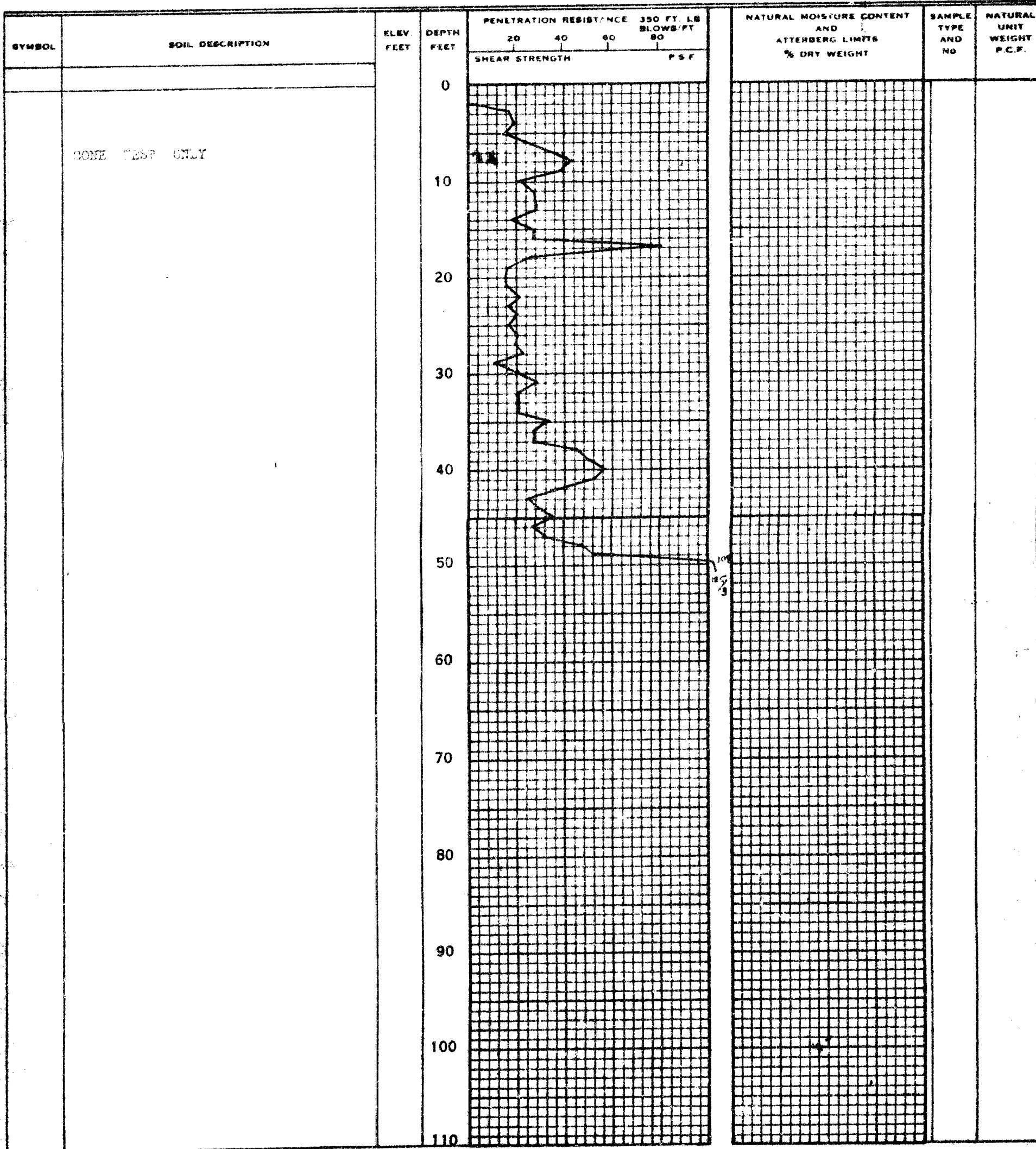
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX 

### ATTERBERG LIMITS

LIQUID LIMIT   
PLASTIC LIMIT 

### SAMPLE TYPE

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





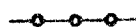

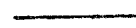
# WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION




DRAWING NO. 3  
PROJECT NO. J1750

## LEGEND

### PENETRATION RESISTANCE


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

### SHEAR STRENGTH




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

### NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

#### ATTERBERG LIMITS

LIQUID LIMIT   
PLASTIC LIMIT 

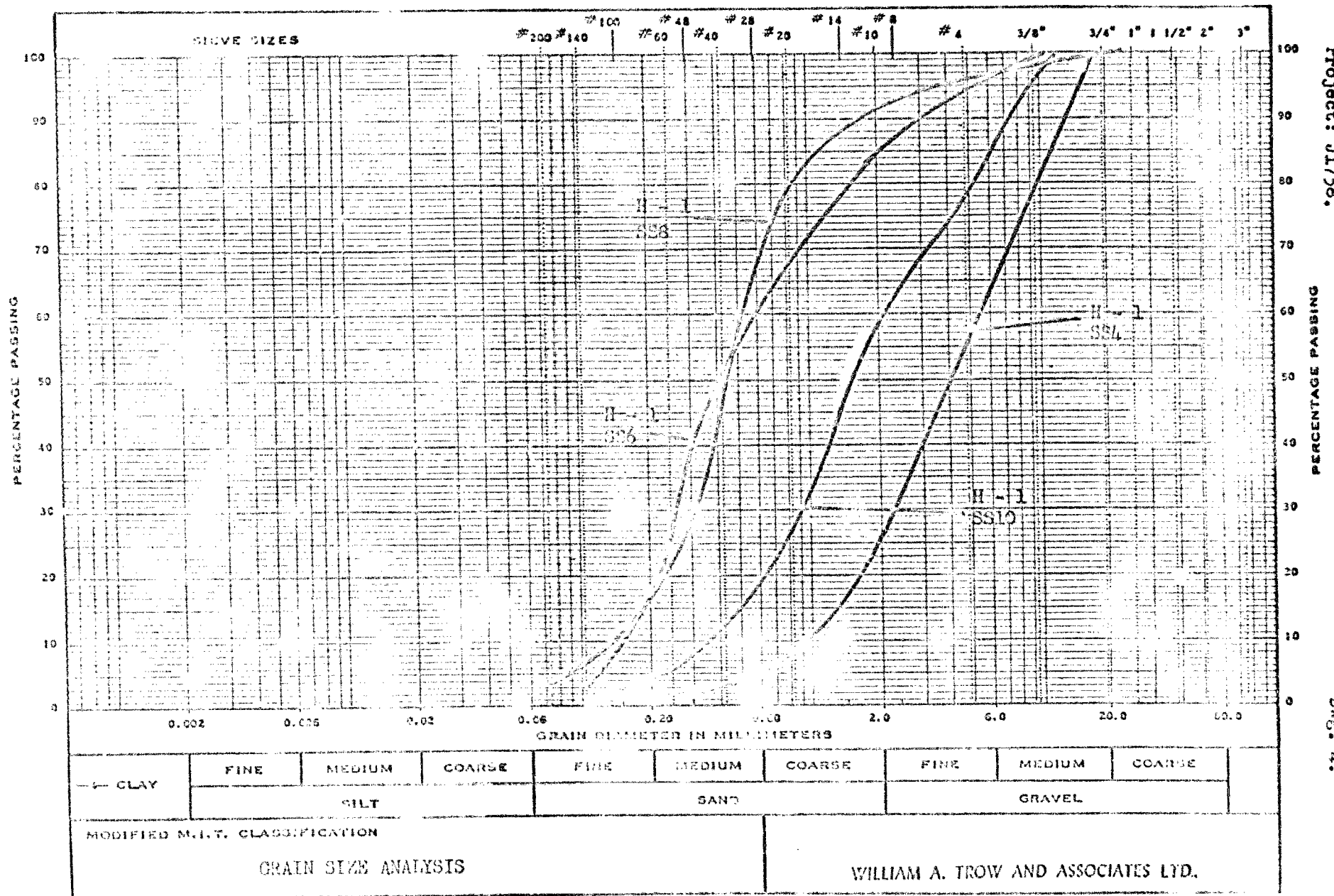
#### SAMPLE TYPE

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

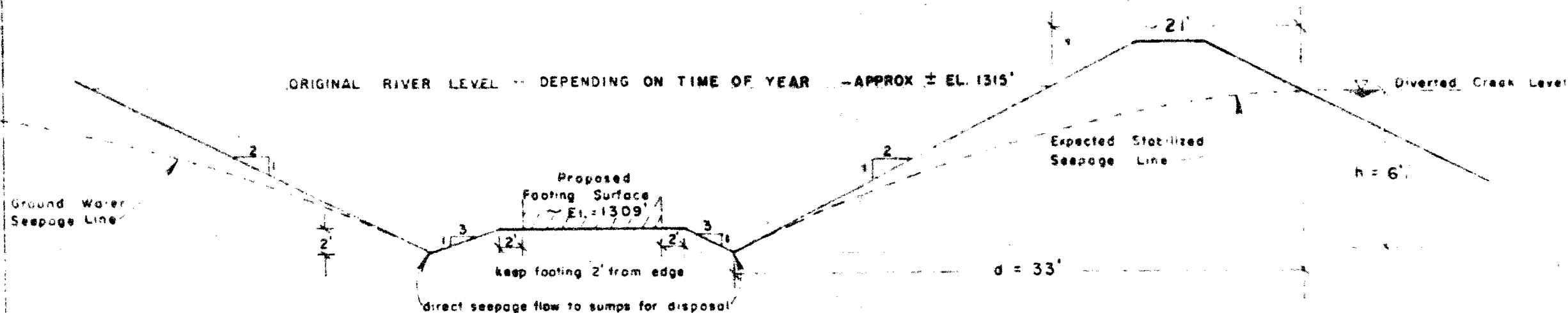
BOREHOLE No. 3  
PROJECT Proposed Bridge, La. River  
LOCATION Highway No. 50  
HOLE LOCATION See Site Plan Dwg.  
HOLE ELEVATION 1315.5 ft.  
DATUM See Site Plan Dwg.

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			
	6" water	310.5	0					
	Cobbles, wood chips to 5 ft.	310.5	0					
	SAND AND GRAVEL-fine to coarse sand and fine to medium gravel, brown, dense.		10					
			20					
			30					
			40					
		271.5	50					
	SAND, GRAVEL, COBBLES, BOULDER-Cored AX 45' to 45'7" (Cobble). Cored AX 52 to 56'. Boulder and sand. Recovered 13' of boulders. Cored AX 56 1/2 - 61', no recovery. Cored AX 63' - 64' no recovery.		60					
		257.5	70					
	BALFOUR-Cored AX 64 to 67 - 100% recovery. End of Borehole	249.5	70					
Notes: 1) Hole advanced, cased through overburden using standard diamond drill equipment. 2) River level rising; water depth 1317 ft. approx. at end of hole; river bottom approx. at 1314.5 ft.								
			80					
			90					
			100					
			110					

# MECHANICAL ANALYSIS



# FOOTING EXCAVATION AND SEEPAGE CALCULATION



## PROCEDURE - 1) Divert creek

- 2) Excavate below water level to approximate dimensions above.
- 3) Pump out water
- 4) Prepare footing bed and install footing
- 5) Backfill to el ± 1314 with adjacent sand and gravel with a rip rap cover of coarser stone.

## ESTIMATE OF SEEPAGE FLOW

$$Q = \frac{P \cdot k \cdot d}{\sqrt{h^2 + d^2}}$$

Take coefficient of permeability  $k = 2 \text{ fpm}$  for sand and gravel\* (conservative estimate)  
 Perimeter of excavation (for base excavation 34' wide and 60' long - p. 188)

Solving  $Q = 300 \text{ gpm}$

It should be appreciated that the seepage line will become progressively flatter and therefore the value  $\sqrt{h^2 + d^2}$  and seepage will reduce in any case.  
 Seepage should not exceed creek flow

\* Seepage Through Dams, 1933 Casagrande

\* Soil Mechanics in Engineering Practice, Table 6, pg 48, Terzaghi and Peck

WILLIAM TROW ASSOCIATES LTD.

PROJ J-1758 DRAWING 5 MAY, 1965

DEFECTS IN NEGATIVE DUE TO  
 CONDITION OF ORIGINAL DOCUMENT

## MEMORANDUM

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

From: Bridge Division,  
Downsview, Ontario.

Date: December 11, 1964.

Our File Ref.

IN REPLY TO

SUBJECT: Foundation Investigation  
District 10

Please make the necessary arrangements to have foundation investigations carried out at the following sites:

- |                 |                                  |
|-----------------|----------------------------------|
| 1) W.F.         | Kindiowami River (East Crossing) |
| 2) W.P. 16-65   | Kindiowami River (West Crossing) |
| 3) W.P. 17-65   | Dival Ck.                        |
| 4) W.P. 18-65   | West Little White River          |
| ✓ 5) W.P. 19-65 | Rapid River                      |
| 6) W.F.         | Sharnsand River                  |

Plans for the above crossings will be available to you by the end of this month.

SHC/sp  
cc. F. DeVisser

*S. McCombie*  
S. McCombie,  
Bridge Planning Engineer.

Hwy. 401 & Beale St.,  
Downsview, Ontario.

Materials and Testing Division

December 17, 1964

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

Attention: W. A. Trow

Re: W.P. - , Hwy. 639, Kindiogami River (East Crossing).  
W.P. 16-65, Hwy. 639, Kindiogami River (West Crossing).  
W.P. 17-65, Hwy. 639, Duval Creek.  
W.P. 18-65, Hwy. 639, West Little White River.  
W.P. 19-65, Hwy. 639, Rapid River.  
W.P. - , Hwy. 639, ~~Chapman~~ River. LITTLE WHITE RIVER  
--- District 15, Sault Ste. Marie ---

Dear Sir:

Please consider this your authority to carry out foundation investigations at the above sites. Plans and profiles were provided to your representative on December 11, 1964.

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

Ten copies of each completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to March 19, 1965. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawings accompanying the foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you are requested to prepare them in accordance with the D.M.C. standards. To enable you to do this, we are supplying you with sample drawings with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide the D.M.C. with Crenaflex copies of the drawings.

cont'd. /2 ...

December 17, 1964

Charges for the work performed will be in accordance with your schedule of Rates, dated November 19, 1962, and invoices to be addressed to the attention of the undersigned.

Yours very truly,



A. Lutka,

Materials & Testing Engineer

WAT/2027

cc: Messrs. S. McCombie  
H. McArthur  
A. A. Ward  
E. P. Saint  
Mrs. T. Tate  
H. D. Smith (2)  
Foundations Office ✓  
Gen. Files (2)

Mr. A. M. Teye,  
Bridge Engineer,  
Bridge Division.

Foundation Section,  
Materials & Testing Div.,  
Room 107, Lab. Bldg.

Attention: Mr. A. McElzbie

June 4, 1965

FOUNDATION INVESTIGATION REPORT BY:  
William A. Trow and Associates, Ltd.  
Proposed Crossing Rapid River, Hwy.  
No. 639, District 18 (Sault Ste. Marie)  
D.P. 19-65

The report on the foundation investigation at the above site submitted by the Consultant has been reviewed.

For the alternative of using pile foundations, we recommend either 20-ft. embedded length, timber piles, or 25-ft. 12-in. diameter steel tube piles should be used. The lengths are 5 ft. longer than those recommended in the report. The capacity of these piles will be the same as suggested by the Consultant, but they should be checked with the Hilley formula as per D.H.C. standards in the field.

In the profile of Drawing Number 1753, the borehole numbers 1 and 2 should be transposed on the prints. This correction has been made on the original copy.

Should additional information be required in connection with this project, please do not hesitate to contact our Office.

RYL/mst  
attach.

cc: Messrs. 1. A. Teye (2)  
2. A. Tregaskas  
3. P. McMillan  
4. W. Hurrell  
5. J. Ward  
6. R. Saint  
7. De Visser  
8. Watt

*A. V. Lo.*  
A. V. Lo.  
SUPERVISING FOUNDATION ENGINEER

Foundations Office  
Gen. Files

MEMORANDUM

To: Mr. A. Stermac,  
Principal Foundation Engineer,  
Downsview, Ontario.

FROM: Bridge Division,  
208 Simpson Street,  
Fort William, Ontario.

DATE: October 12, 1965.

OUR FILE REF.

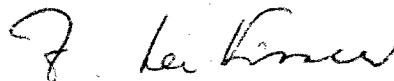
IN REPLY TO

SUBJECT:

Site 38S-278, W. P. 19-65  
Rapid River Bridge,  
4.0 Miles East of Hwy. 129,  
District 18.

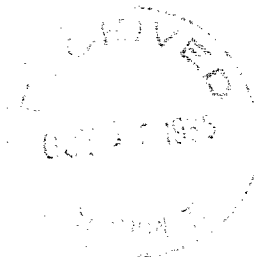
Enclosed please find one print of the  
Preliminary Plan for the Rapid River Bridge.

If you have any comments please let me  
know.



FDV/mcr  
Enc.

F. DeVISSER,  
Regional Bridge Location Engineer.





W.P. 19-65

Note:

Discussed foundations with Jim Keen.

Scour depth 9 ft. therefore piles are used. Sheet piling as a permanent feature is doubtful because adequate penetration may not be achieved.

Because of the foundation bottom elevation dewatering will be necessary. The subsoil is rather permeable and high capacity pumps may be needed. In case dewatering proves to be impractical a Tremie concrete slab can be poured and with the help of timber sheeting the rest of the footing can be poured in the dry.

The Contractor should be made fully aware that dewatering may be a problem.

The proposed pile tip elevation 1288.0 is only approximate and the final depth should be determined on the basis of the Hilley formula.

Afternoon

Oct. 18. 1965.