

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS  
LABORATORY TESTING  
SOIL MECHANICS CONSULTATION

23-66-267  
BA1817

W. A. TROW, M.A.Sc., M.E.I.C., P.ENG.

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

Project: J1355

April 10, 1964

Mr. A. Rutka,  
Materials and Research Engineer,  
Materials and Research Section,  
Department of Highways of Ontario,  
Parliament Buildings, Ontario

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

Re:

Foundation Conditions  
Proposed Crossing Over Bonnechere River  
Highway 521, Bonnechere, Ontario

W.P. 254 - 62

Dear Sirs:

In conformance with your letter of authorization, dated February 17, 1964, we have completed a foundation investigation at the above bridge site. The field work, consisting of five borings and six dynamic cone penetration tests, was carried out between February 25th and March 7th. Our findings and conclusions arising out of this work are as follows:

- 1) The site is covered by a 4 to 8 feet thick layer of loose sandy silt which is underlain by a layer of loose sand extending to a general elevation of 515 feet, which is approximately 40 feet below the river bed. Beneath these loose granular deposits, there is a stratum of dense to very dense stratified sandy silt with occasional clayey silt seams. Boulders and cobbles were encountered in the stratum below depths of 65 feet to the end of boring at 99 feet.

- 2) In view of the loose nature of the upper granular deposits at the site, the use of footings placed at shallow depths for the new bridge is not considered as feasible. It is recommended that the proposed structure be founded on end-bearing piles, such as timber piles, driven into the dense to very dense sandy silt stratum at El 515 feet. Assuming that the base of the pile cap for the new bridge is to be at the river bed or at approximate El 555 feet, it is estimated that a timber pile of 8 inches in diameter at the tip, driven to El 510 feet or about 5 feet into the dense sand-silt stratum will have an ultimate bearing capacity of 47 tons.
- 3) If pile caps were to be situated level with the river bed, the construction work could be carried out inside braced sheeting driven a few feet below the base of the pile cap. The excavation to the desired level, as well as the casting of the concrete structure, can be made below water level. The sheeting that surrounds each pile-cap could be left in place for scour protection.
- 4) Because of the granular nature of the subsoils at the site, embankment stability as well as long term settlement is not considered as a problem in this project.

The foregoing comments are enlarged upon in the following sections.

#### SITE

The site is located immediately to the north of the existing Highway 521 crossing of Bonnechere River, approximately 3 miles east of Bonnechere, Ontario.

The existing bridge at the crossing is a timber structure, approximately 70 feet long and 20 feet wide, and is supported on timber cribs with twin openings of 23 feet. This bridge is topped

by a Bailey structure, 12 feet wide and 100 feet long, with a deck level at El 571 feet. At the time of the investigation, the ice level in the river was at El 561, which is about 5 feet above the river bed. The general area around the crossing is fairly flat at approximate El 564 feet, and is covered by numerous trees up to 2 feet in diameter.

#### FIELD WORK

A total of five cased borings and six dynamic cone penetration tests were put down using a conventional diamond drill rig equipped for soil sampling purposes. Each boring was advanced by driving BX pipe into the ground to the desired sampling depth, and then by cleaning the hole with water. Because of the granular nature of the subsoils at the site, sampling was carried out by means of a 2 inch O.D. split spoon which was driven into the soil under an energy of 350 ft. lbs. per blow. Thin wall tube samples were obtained in the lower sandy silt stratum in order to determine the approximate in-place density of the soil.

The locations of boreholes, together with the interpreted subsoil stratigraphy, are shown on the site plan drawing.

The elevations of the holes are referred to the top of a nail driven into the root of a 1.0 foot diameter Red Pine tree at 92 feet to the right of Sta. 552 + 41. The Geodetic elevation of this reference point was given to be 566.26 feet, as indicated on the D.H.O. site plan No. E - 4245 - 1.

#### SUBSOIL CONDITIONS

The results of the borings are described in detail on the borehole logs, Dwg. 1 to 5, of this report, and only a brief summary of these findings is given as follows:

a) Sandy Silt

The site is generally covered by a 4 to 8 feet thick layer of sand-silt with cone penetration resistances ranging from 1 to 5 blows per foot. These deposits are generally yellowish brown in colour and contain some grass roots in the upper portion.

b) Sand

Beneath the surfacial silt layer, or at a level approximately coinciding with the river bed, there is a stratum of sand with occasional seams of fine gravel extending to about 40 feet below the bottom of the river, or to El 515 feet. From visual inspection on samples obtained, this stratum consists predominantly of medium sand sizes for the upper 30 feet. The lower portion of the sand becomes finer in size, and silt was brought up during the washing process. Five grain size analyses were carried out on samples from the stratum, and the resulting curves are shown on Dwg. 7.

Standard penetration tests were performed on the sand, and the average 'N' values were found to be 5 and 13 blows per foot from the upper medium sand and the lower fine sand respectively.

SANDY SILT

Below the layer of silty fine sand or the transition zone, a stratum of sandy silt with occasional clayey silt seams was encountered to the sampled depth. The 'N' values obtained in the silt were found to range from 31 to 45 blows per foot, with an average value of 38 blows per foot.

Six grain size analyses were performed on the silt sample and these resulting curves are shown on Dwg. 8.

Natural unit weight as well as maximum and minimum density determinations were carried out on selected silt samples, and the results are given on the borehole log and on Table 1. It can be seen that the sandy silt has an in-place unit weight ranging from 120 to 127 pcf, a porosity of 44 to 40, and a relative density of 72 to 73.5 percent. These values, together with the results of penetration tests, indicate that the sandy silt exists in a dense to very dense state.

### FOUNDATIONS

It is understood that the replacement for the existing bridge will incorporate an increase in width, from 20 feet to 40 feet, and in grade, from El 571 feet to El 575 feet, while the length of the bridge will remain unchanged. The centre line location of the proposed structure is to be about 50 feet to the north of the existing bridge.

Because the upper sandy silt and medium sand strata are loose in relative density, the use of footings to support the new bridge is not considered as feasible in this project. It is recommended that the proposed structure be founded on displacement type piles driven to end-bearing in the dense stratified silt-sand stratum at approximately 40 feet below the river bed. The ultimate bearing capacity,  $Q_u$ , of a displacement pile driven into granular material can be estimated by the following expression:

$$Q_u = A_b \cdot \gamma \cdot D \cdot N_q$$

where:

- $A_b$  is the cross section area at the base of pile
- $D$  is the penetration depth of pile into soil
- $\gamma = 50$  pcf is the submerged unit weight, estimated for the medium sand
- $N_q$  is a bearing capacity factor depending on the friction angle of the load-bearing soil.

In the selection of friction angle,  $\phi$ , for the dense stratified sandy silt, the lowest 'N' value of 31 blows per foot and the highest measured porosity of 44.3 are considered. According to Peck\* and Bjerrum\*\*, the  $\phi$  angle is estimated to be  $35^\circ$  and  $35^\circ$  respectively. It is recommended that a  $\phi$  angle of  $35^\circ$  be used in design. The corresponding  $N_q$  for a  $60^\circ$  pointed pile driven approximately 5 feet into the load-bearing stratum is 120.\*\*\*

For a pile of 8 inches diameter at the tip, driven approximately 5 feet into the dense sandy silt or a penetration length of 45 feet, the ultimate bearing capacity will be:

$$Q_u = 0.349 \times 50 \times 45 \times 120 = 94,000 \text{ lbs. or } 47 \text{ tons.}$$

Depending on the loading conditions of the new bridge, the inclined loads due to lateral earth pressure and other forces may necessitate the use of batter piles.

If the pile cap is to be situated below water level in the river, the construction work can be carried out inside braced sheeting driven a few feet below the pile base. The excavation to the desired level as well as the casting of the concrete structure can be carried out below water.

In order to maintain the bearing capacity of the piled foundations, scour protection, in the form of a layer of rip rap in front of each abutment wall, should be provided. The sheeting for the pile cap construction can also be left in place for the same purpose.

\* Peck, Hanson & Thornburn - "Foundation Engineering" P.222, Wiley, N.Y.

\*\* Bjerrum, Kringstad & Kummeneje, 1961 - "The Shear Strength of a Fine Sand", 5th Int. Conf. Soil Mechanics & Foundation Engineering, Vol. I, P.29

\*\*\* G.G. Meyerhof, 1963 - "Some Recent Research on the Bearing Capacity of Foundations", Canadian Geotechnical Journal, Vol. I, P. 16

EMBANKMENT

It is understood that the proposed grade of embankment at the crossing will be at El 575 feet, which is approximately 10 feet above the existing ground surface. The granular subsoils at the site will adjust as soon as the weight of fill is applied, and therefore, long term settlement is not a problem in this case. Although the upper silt and sand strata are loose in relative density, it is believed that the light embankment loading will not endanger the stability of the soil.

EARTH PRESSURE

The abutment walls of the new bridge should be designed to withstand the lateral earth pressure exerted by the retained soil. Assuming that free-draining, granular material is to be placed behind these walls, the magnitude of lateral earth pressure will depend on the degree of compactness of the backfill, and the flexibility of the retaining structure. Since the piled structure is fairly rigid, the earth pressure will approach the 'at rest' state, in a short period of time. It is therefore recommended that the walls be designed for a pressure only slightly less than the at rest condition. If the compacting effect produced a somewhat greater lateral earth pressure than the condition assumed, the normal factor of safety used in structural design should be sufficient to compensate for this situation.

The horizontal force,  $P$ , at any depth,  $h$ , can be estimated by the following expression:

$$P = K ( \gamma (h-h_1) + \gamma' h_1 + q )$$

where:  $K = 0.35$  is the earth pressure coefficient  
estimated for well compacted granular  
backfill

$\gamma = 115$  pcf is the estimated unit weight of  
retained soil above water table

$\gamma' = 50$  pcf is the estimated submerged unit  
weight of the retained soil

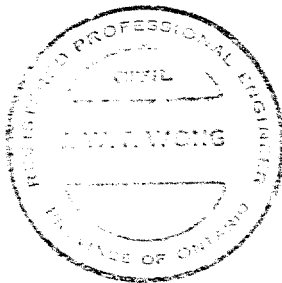
$h_1$  is the height of water table above the  
point being considered

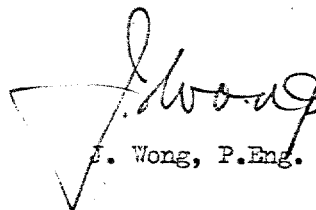
$q$  is the magnitude of surcharge, if any,  
acting near the top of abutment wall.

Depending on the magnitude of the lateral earth pressure,  
batter piles may be required to maintain the stability of the  
abutment structure.

We shall be pleased to discuss any question you may have  
after reviewing the contents of this report.

Yours very truly,



  
E. Wong, P.Eng.

JW/gc  
Encls.



TABLE 1

## RELATIVE DENSITY DETERMINATIONS

&amp;

Hole	Depth Ft.	Moisture Content	Natural Unit Wt. pcf By Measurement*	Unit Wt. pcf Indirectly From W**	Average Dry Unit Wt. pcf	Relative Density % +	Porosity	Ø
3	46	27.8	127.3	123	98.3		.418	38
	51	27.1		123.8	97.3	73.5		
	56	26.0	121.2	124.9	97.8		.420	38
	61	24.5	119.7	126.2	98.8		.413	38.5
5	56	29.4	121.3	121.5	94.0	72	.442	35
	61	25.3	125.8	125.6	100.2		.403	39.5
	71	26.6	122.5	124.4	97.6		.421	38

\* Volume and weight determination

\*\* For Saturated Soil  $\gamma = \frac{(62.4S)}{(1+SxW)}$ 

where: S = Specific gravity assumed = 2.70

W is moisture content expressed as a decimal

$$+ \text{ Relative Density } = \frac{\gamma_{\max}(\gamma - \gamma_{\min})}{\gamma(\gamma_{\max} - \gamma_{\min})} \times 100\%$$

Maximum and minimum density of the sandy silt, as determined in the laboratory, were found to be 115 and 68 pcf respectively.



North Side of Existing Bridge  
Rig on B.H. 2



View Looking East



North Side of Existing Bridge  
Rig on B.H. 2



View Looking East



View Looking West  
Rig on B.H. 2



View Looking West From East Bank, Rig on B.H. 5  
Stakes Are Along Prop. C.L.



View Looking West  
Rig on B.H. 2



View Looking West From East Bank, Rig on B.H. 5  
Stakes / re Along Prop. C.L.

## LEGEND

BOREHOLE NO. 1.  
 PROJECT Proposed Crossing Over Bonnechere River.  
 LOCATION Rwy. 521, Bonnechere, Ontario.  
 HOLE LOCATION See Dwg. 1.  
 HOLE ELEVATION 564.0 ft.  
 DATUM Geodetic.

## 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  $c_u$ 

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

## ATTERBERG LIMITS

LIQUID LIMIT

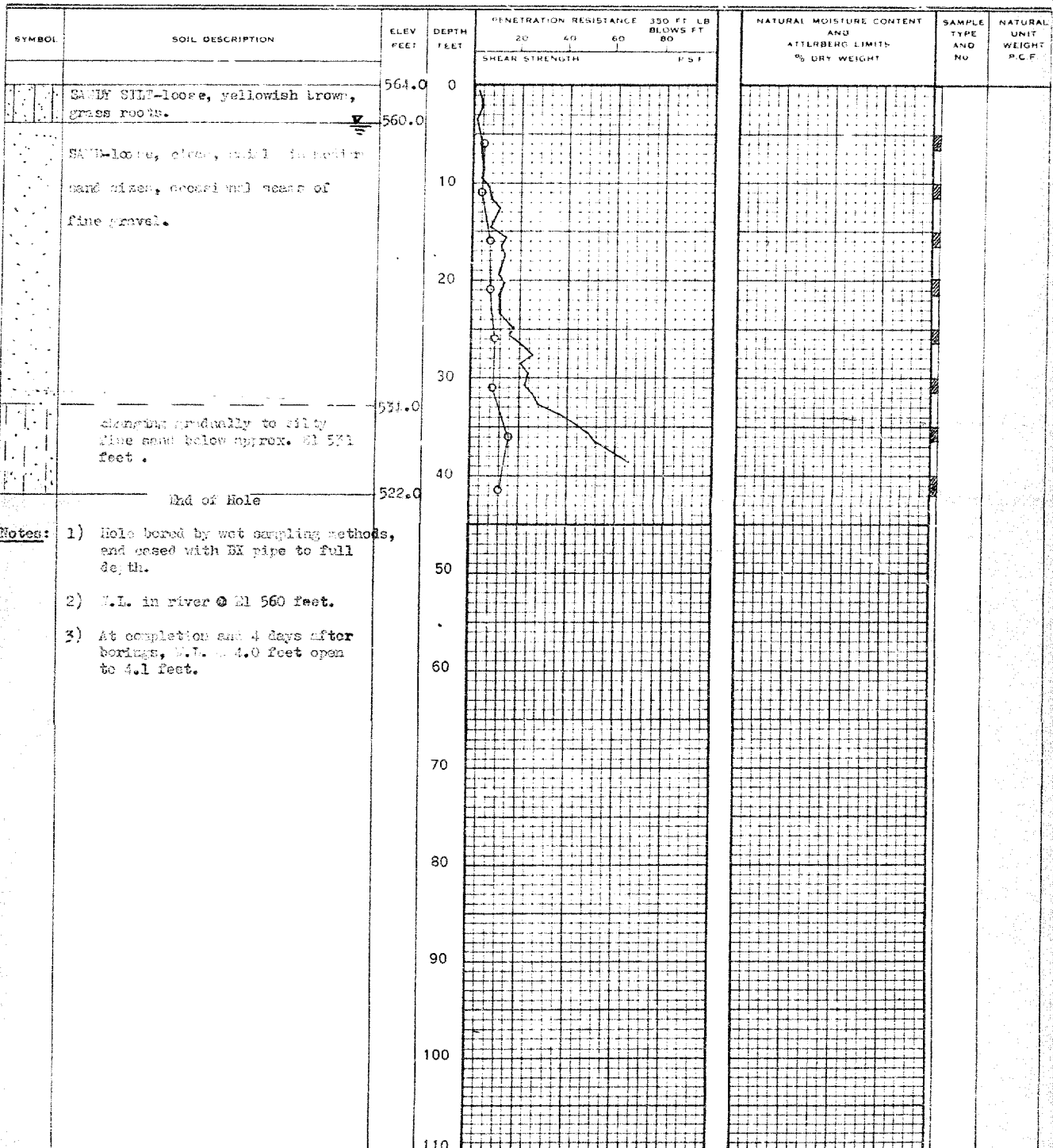
PLASTIC LIMIT

## SAMPLE TYPE

7" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

3" O.D. SHELBY TUBE



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<sub>u</sub>) ± ‡

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

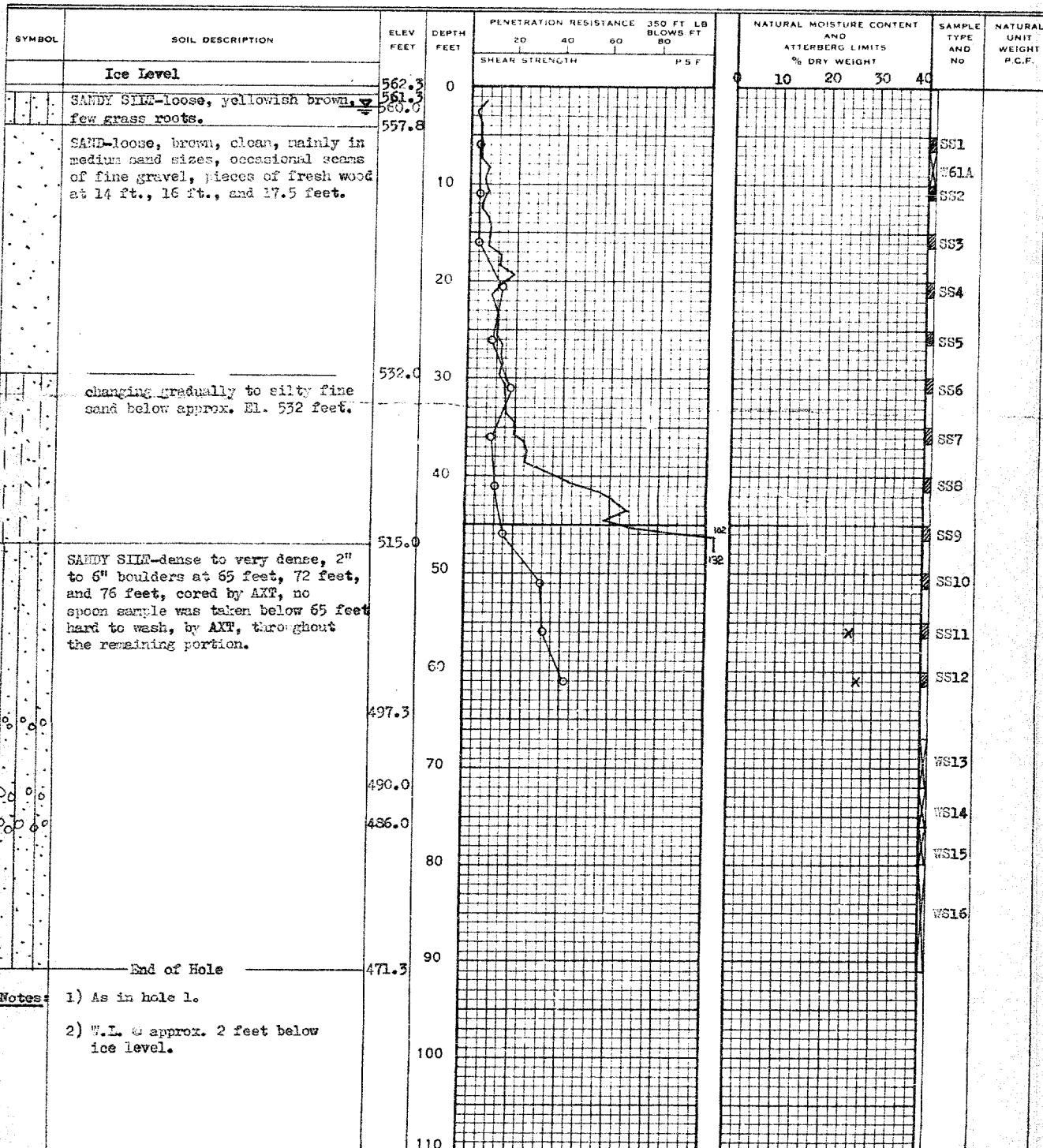
LIQUID LIMIT —○—

PLASTIC LIMIT ———

SAMPLE TYPE







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

BOREHOLE No. 2.  
PROJECT Proposed Crossing Over Bonnechere River.  
LOCATION Hwy. 521, Bonnechere, Ontario.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 562.3 ft.  
DATUM








BOREHOLE NO. 3.  
PROJECT: Proposed Crossing Over Bonnechere River.  
LOCATION: Hwy. 521, Bonnechere, Ontario  
HOLE LOCATION: See Dwg. 1.  
HOLE ELEVATION: 563.6 ft.  
DATUM: Geodetic.

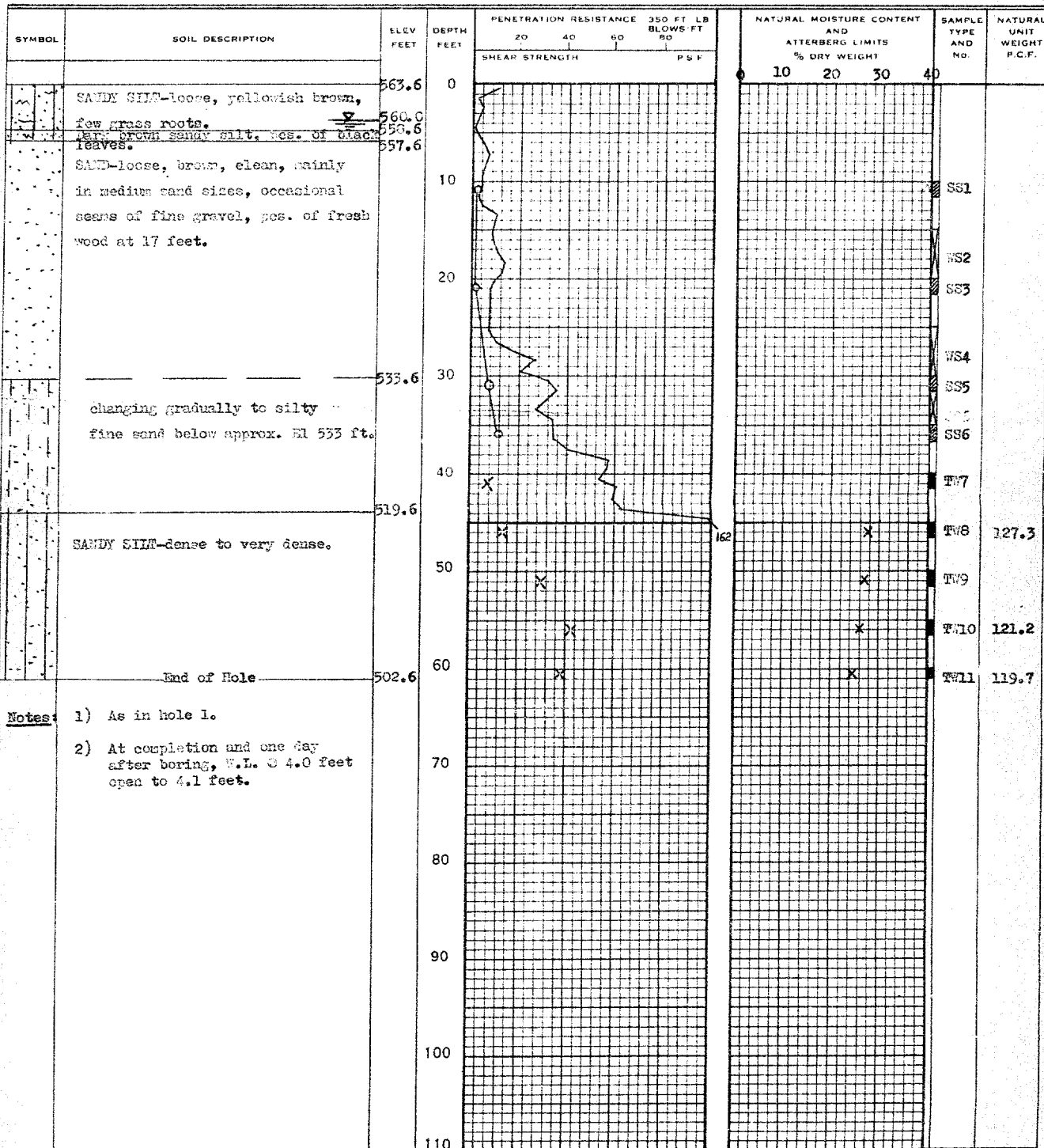
## PENETRATION RESISTANCE

2" O.D. SPLIT TUBE   
2" I.D. SHELBY TUBE   
2" DIA. CONE   
SHEAR STRENGTH  
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE   
UNCONFINED COMPRESSION   
VANE TEST AND SENSITIVITY  $15 \frac{1}{2}$  

## 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 



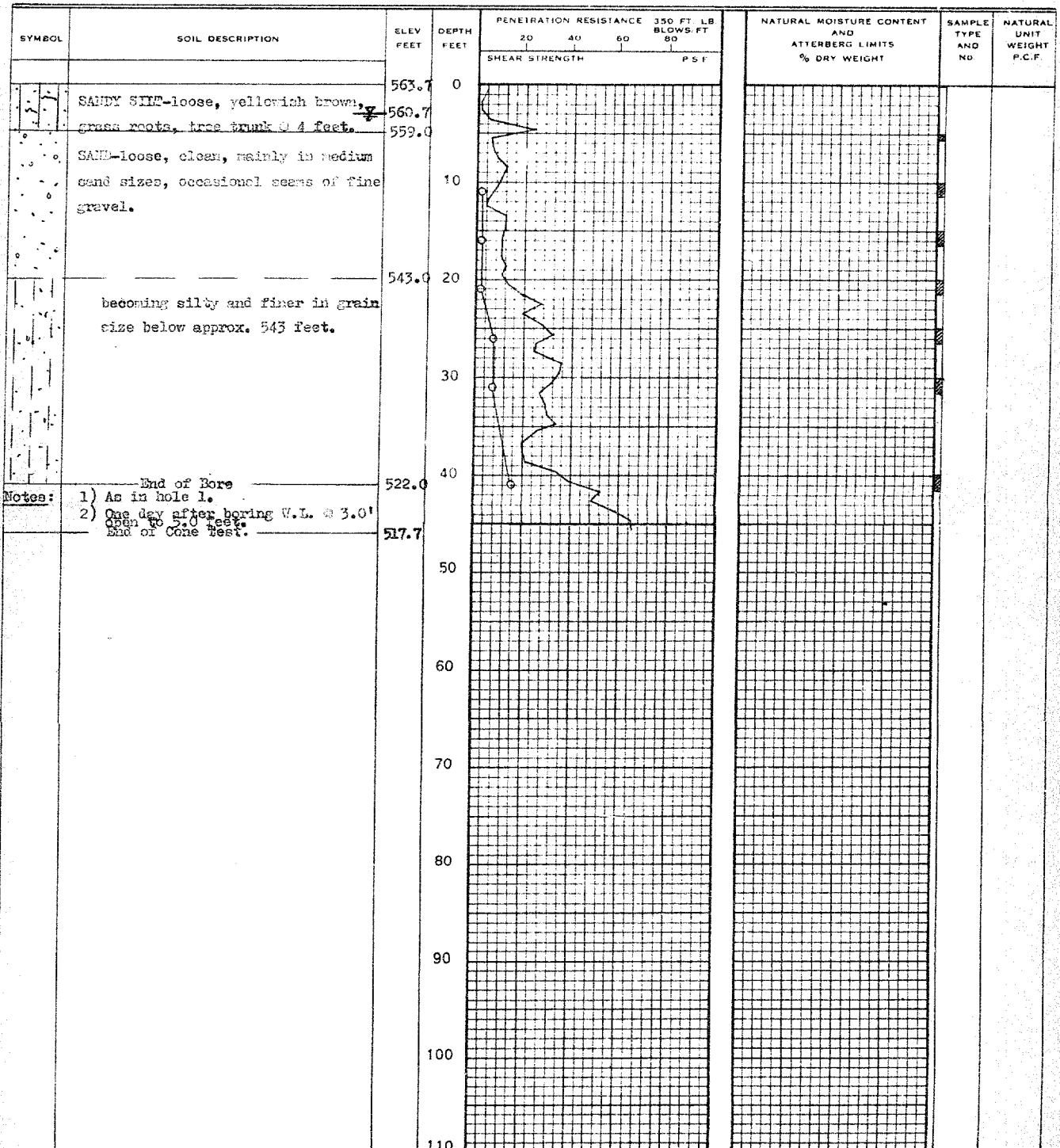


## LEGEND

BOREHOLE No. 4.  
 PROJECT Proposed Crossing Over Donnechere River.  
 LOCATION Hwy. 521, Donnechere, Ontario.  
 HOLE LOCATION See Dwg. 1.  
 HOLE ELEVATION 563.7 ft.  
 DATUM Geodetic.




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

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. 5.  
 PROJECT Proposed Filling Over Donnechere River.  
 LOCATION  Hwy. 521, Donnechere, Ontario.  
 HOLE LOCATION See Dwg. 1.  
 HOLE ELEVATION 566.0 ft.  
 DATUM Geodetic.


## 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  $\phi$  




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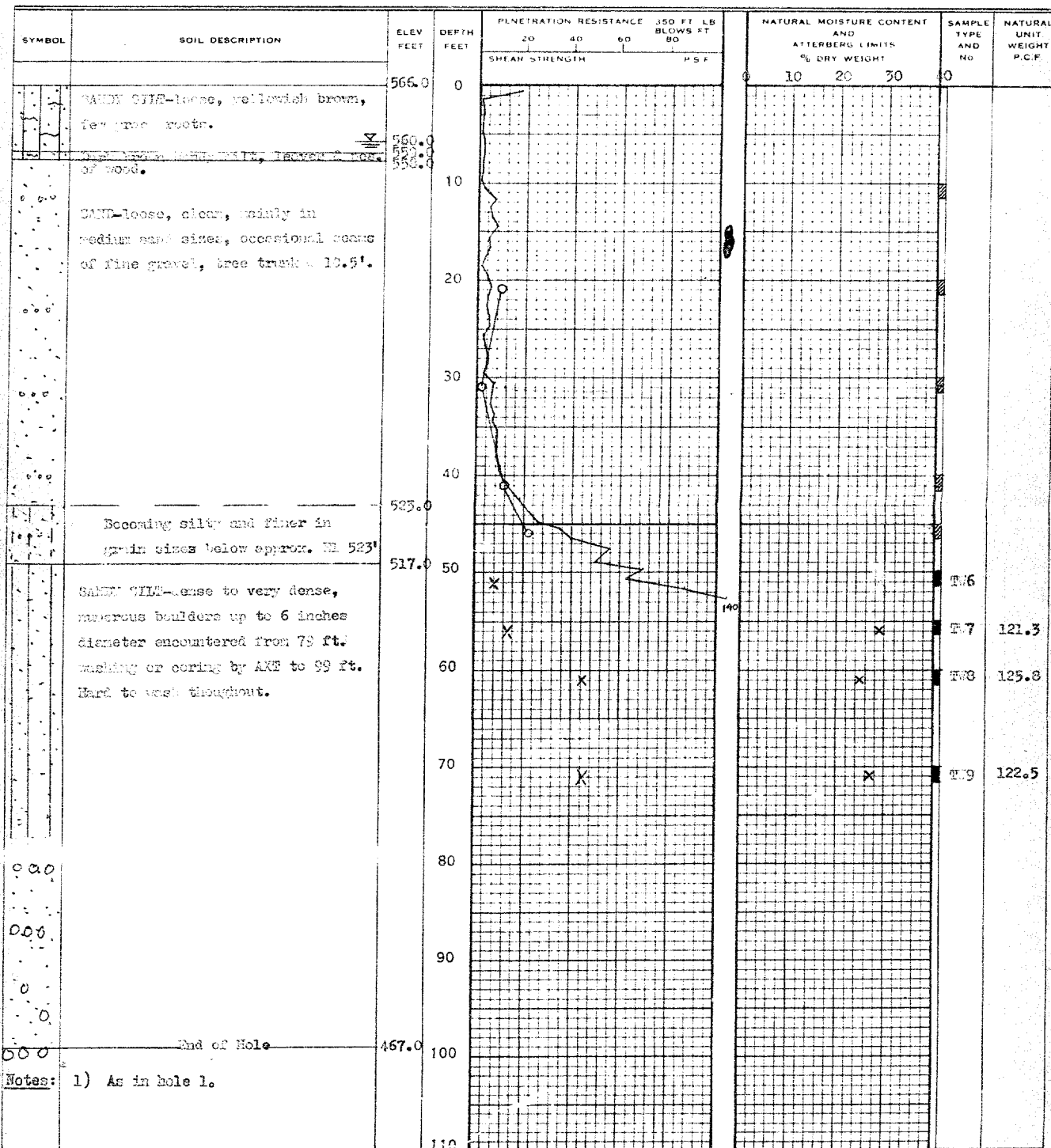
AND LIQUIDITY INDEX 

## ATTERBERG LIMITS

LIQUID LIMIT PLASTIC LIMIT 

## SAMPLE TYPE

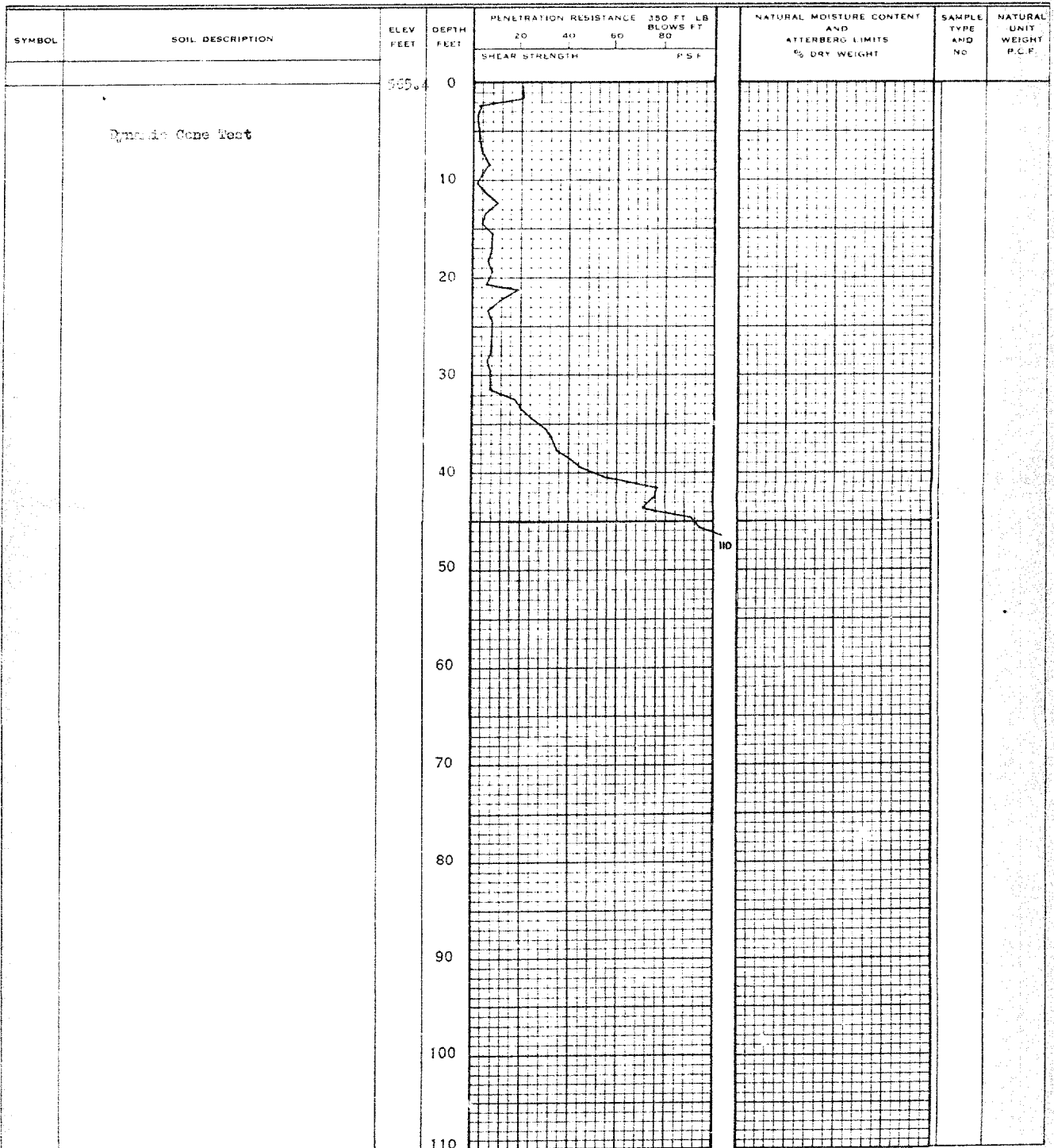
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 2 I.D. SHELBY TUBE   
 3 O.D. SHELBY TUBE 



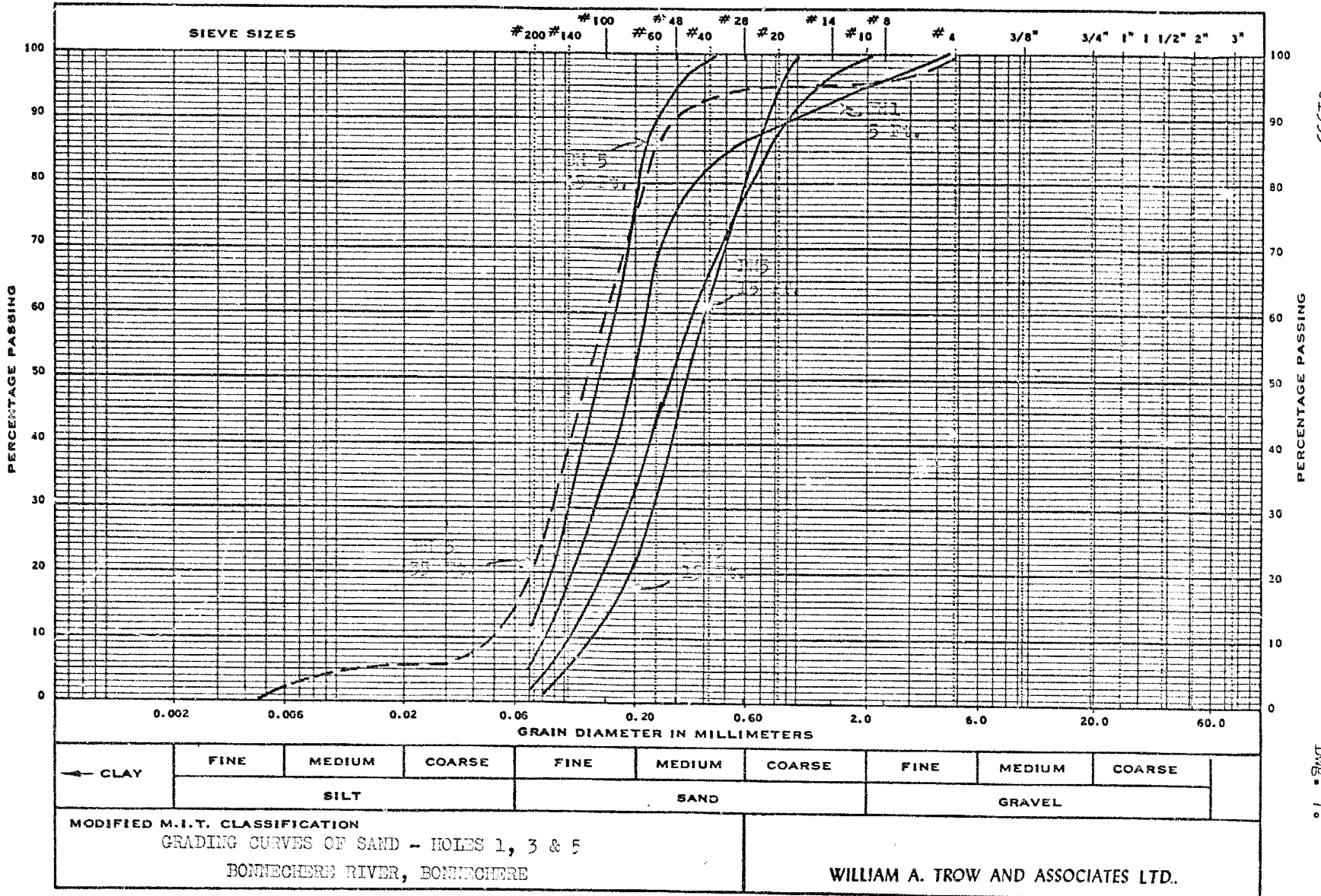
BOREHOLE NO. Cone 6.  
PROJECT Proposed Crossing Over Donnechere River.  
LOCATION Rty. 521, Donnechere, Ontario.  
HOLE LOCATION See Map. 1.  
HOLE ELEVATION 565.1 ft.  
DATUM Geodetic.

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

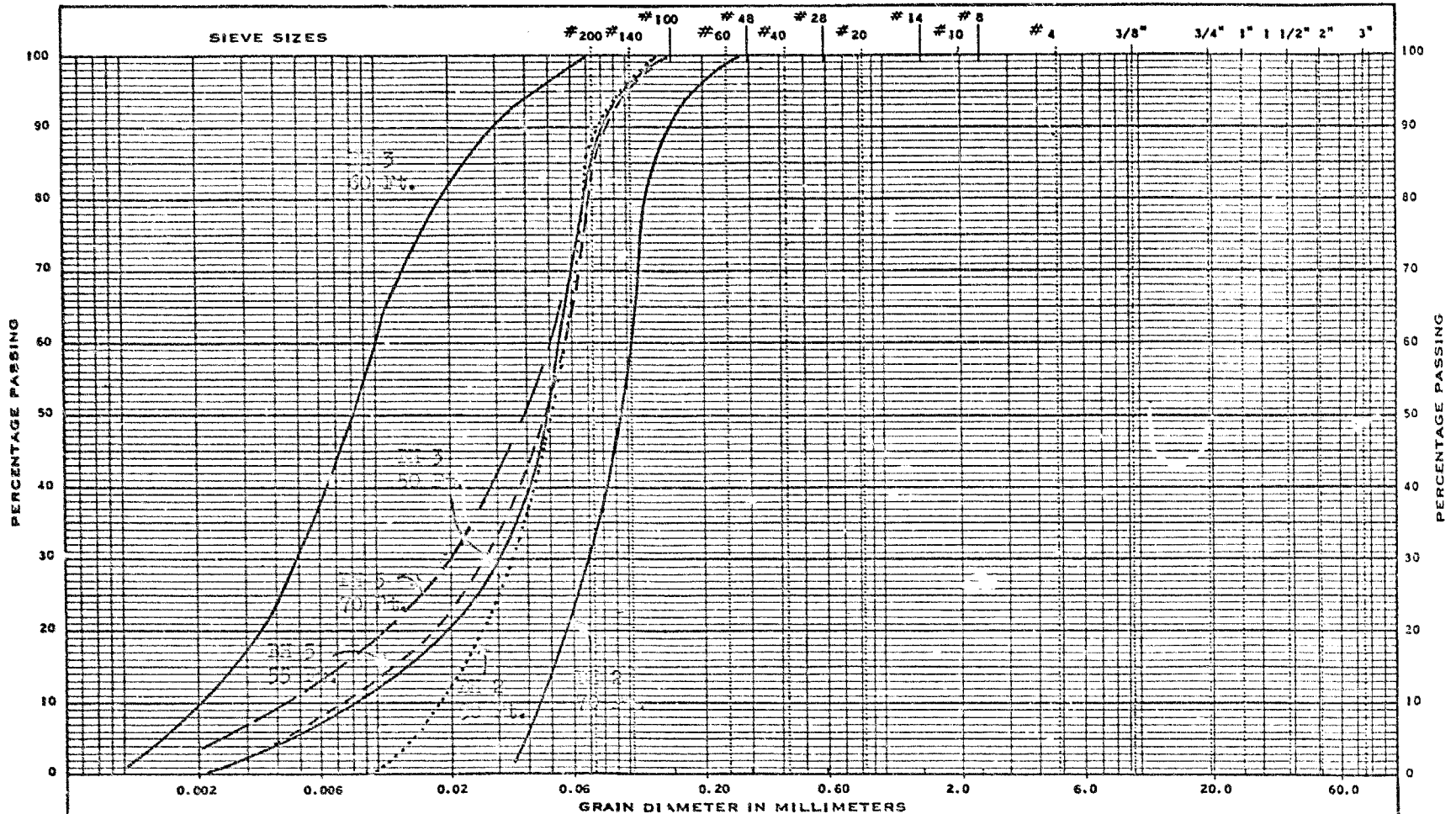
NATURAL MOISTURE CONTENT  
AND LIQUIDITY INDEX  
ATTERBERG LIMITS  
LIQUID LIMIT  
PLASTIC LIMIT  
SAMPLE TYPE  
2 O.D. SPLIT TUBE  
2 I.D. SHELBY TUBE  
3 O.D. SHELBY TUBE



# MECHANICAL ANALYSIS



# MECHANICAL ANALYSIS



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	
	SILT			SAND			GRAVEL			
MODIFIED M.I.T. CLASSIFICATION										
GRADING CURVES OF SANDY SILT - HOLES 2, 3, & 5.										
BONNECHERE RIVER, BONNECHERE							WILLIAM A. TROW AND ASSOCIATES LTD.			

*Open file*

Materials and Research Division

February 17, 1964

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

Attention: Mr. Wm. A. Trow

Re: W.P. 254-62, Sec. Hwy. 521, Bonnechere River Bridge,  
3.6 mi. South of North Jctn. of Hwy. 62, District #10,  
Barnsby, Ontario.

Dear Sir:

Please consider this your authority to carry out a foundation investigation at the above site. Plans and profiles were provided to your representative on February 13, 1964.

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

Ten copies of the completed foundation report with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to May 1, 1964. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawing accompanying the foundation report, showing the location of borings, the inferred subsoil conditions, etc., is to become one of the contract drawings, you are requested to prepare it in accordance with the D.H.C. standards. To enable you to do this, we are supplying you with a sample drawing with all the necessary explanations, together with a linen sheet for your drawing. You are also requested to provide the D.H.C. with a Cronaflex copy of the drawing.

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

NDS/MJEF

Yours very truly,

cc: S. McCombie  
J. Ford  
J. E. Callaghan  
J. E. Gruspier  
H. D. Smith (2)  
H. Konings

*A. Ruths*  
A. Ruths,  
MATERIALS & RESEARCH DIVISION

Foundations Office -- Gen. Files

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

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

Attn: Mr. S. McCosbie

April 20, 1964

FOUNDATION INVESTIGATION REPORT BY:  
William A. Troy & Associates Ltd.,  
Bonnechere River Bridge, Hwy. No. 521,  
W.P. 254-62 -- District No. 10

Attached, we are forwarding to you the above-mentioned report and herewith, submit our comments for your consideration:

There is a definite change in materials around elevation 525.0. Even above this elevation there is a distinct increase in the density of the sand layer. It is between elevation 510 and 520 that the cones meet practical refusal.

In view of the above-mentioned facts, we feel that a safe load of 40 tons could be applied to a 12-inch steel tube pile driven to elevation  $\pm$  505.0. It is felt that a more rational design could be achieved in this manner.

Should there be any questions you would like to have clarified, please feel free to call on our Office.

AGS/MdeF  
Attach.

cc: Messrs. A. M. Toye (2)  
H. A. Tregaskes  
R. D. McMillan  
J. Ford  
J. E. Callaghan  
J. E. Graspier  
A. Watt

*A. C. Sternac*  
A. C. Sternac,  
PRINCIPAL FOUNDATION ENGINEER

Foundations Office ✓  
Gen. Files



## MEMORANDUM

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

From: Bridge Division,  
Downsview, Ontario.

Date: October 8, 1964.

CUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 254-62  
Bridge Site 30-14  
Bonnechere River Bridge  
3.6 miles south of north Jct. Hwy. 62  
Sec. Hwy. 521 - District 10

Enclosed please find one copy of the preliminary plan D 5498-P1 for the above noted structure.

Would you kindly review the bridge foundations proposed and inform me if they are satisfactory.

*Apwatt*

APW/im

A. P. Watt,  
Regional Bridge Location Engineer.

I formed Jim Keen the following information regarding the above mentioned job on 23rd Oct 1964.

The structure can be supported on end bearing piles driven to practical refusal. In such a case a safe load of 100 tons may be used for 12 1/2" O.D. tubular driven to the bearing stratum.

Initially it was recommended tubular piles (12 1/2" O.D.) driven to level  $\pm 98.0$  with a safe load of 40 tons per pile.

However the final choice should be decided upon economical considerations. Jim Keen on 26th Oct 1964 informed that cost will be same for both the recommendations and therefore the original recommendation will be adopted.

*APW*  
Oct 26th 1964



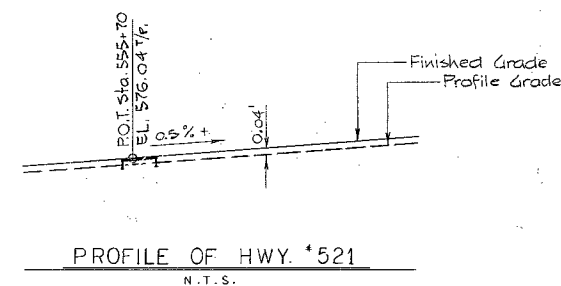
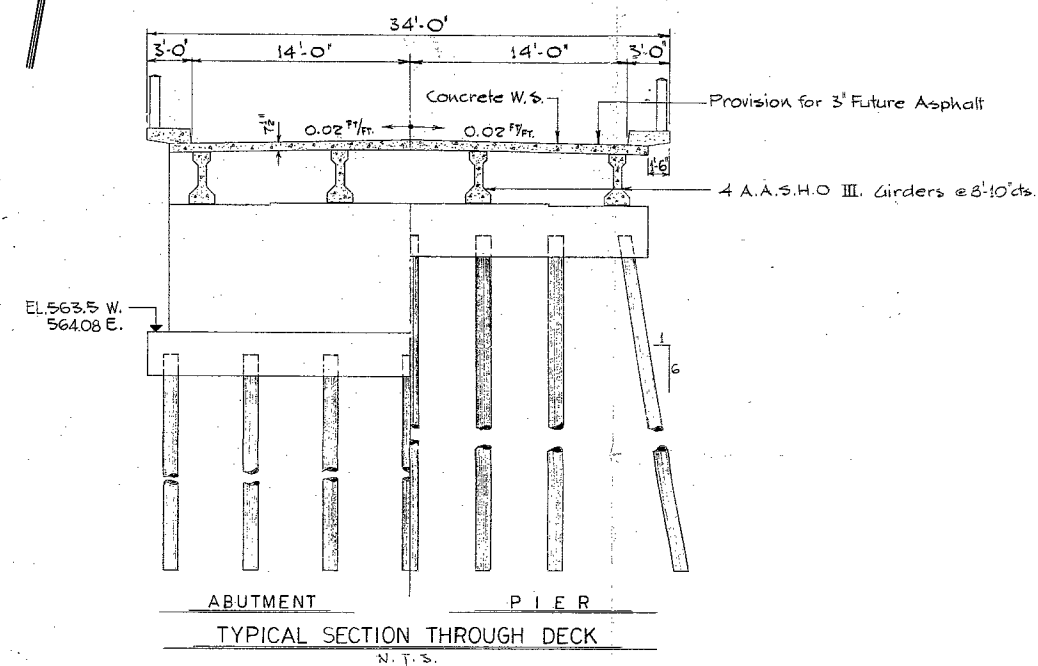
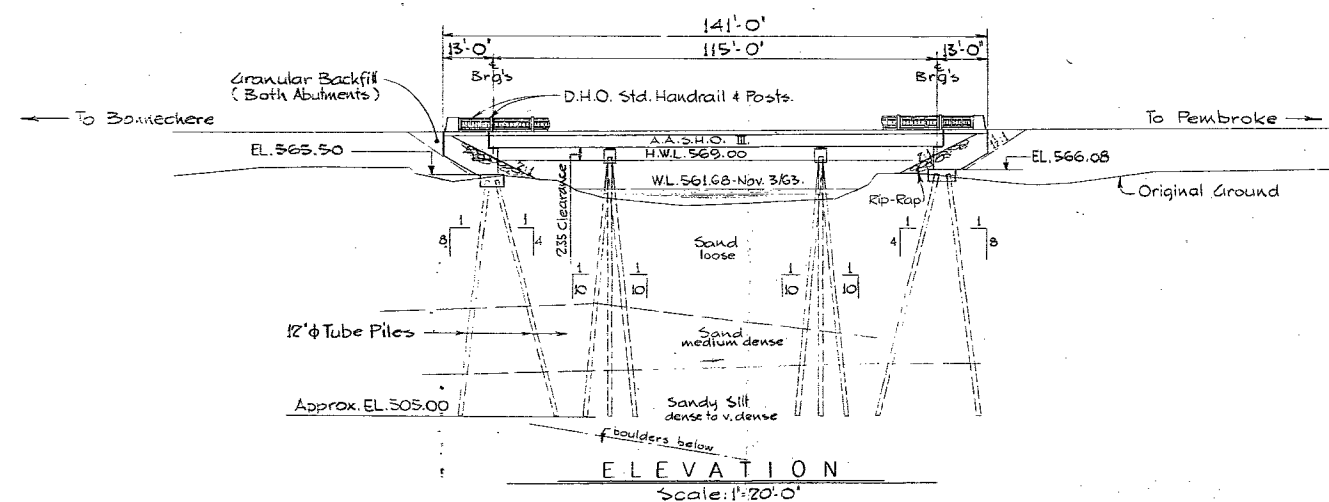
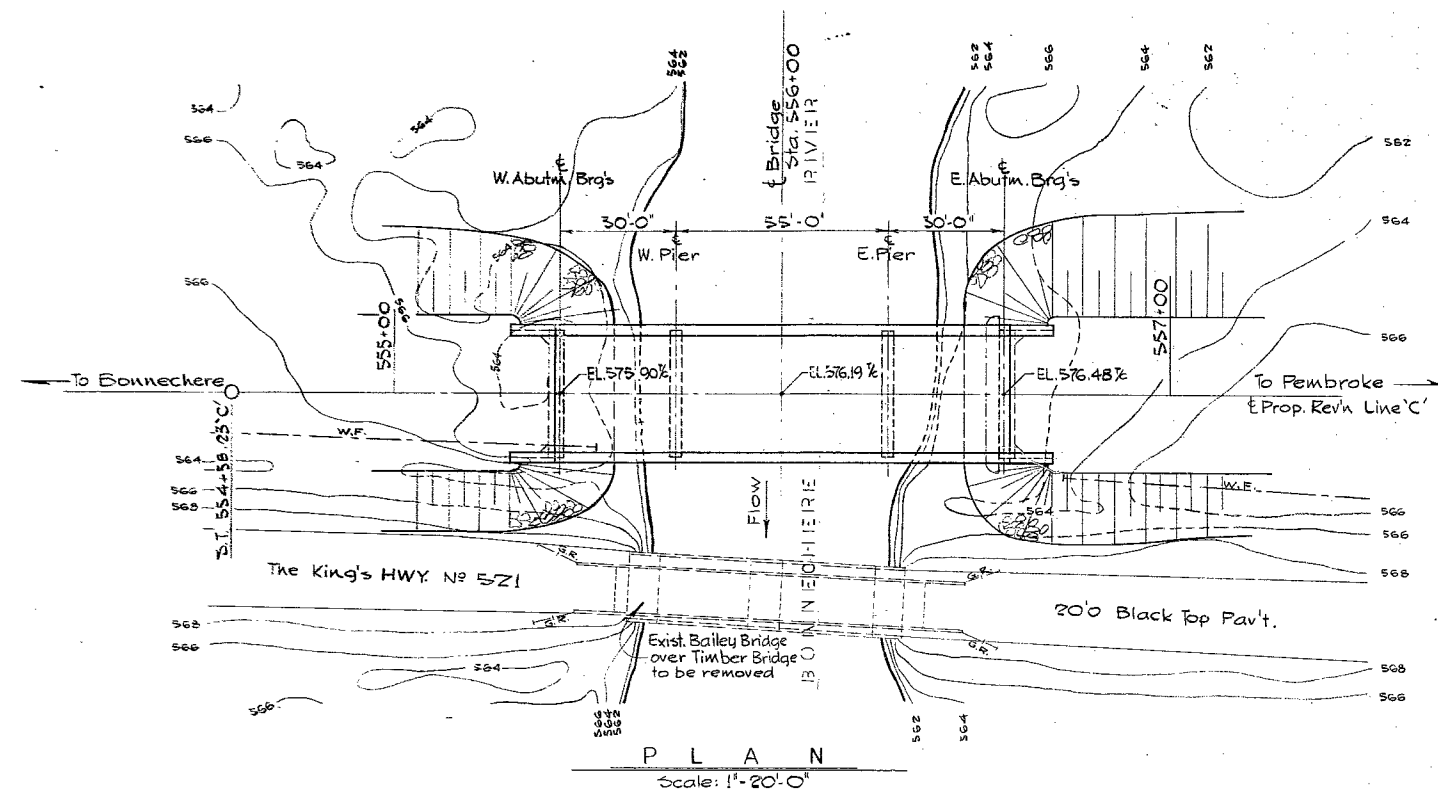
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W.P. 254-62

HWY. #521

BONNECHERE  
RIVER





REVISIONS			
DATE	BY	DESCRIPTION	

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
<h2 style="margin: 0;">BONNECHERE RIVER BRIDGE</h2>			
3.6 MI. SOUTH OF NORTH JCT. HWY. 62			
KING'S HIGHWAY No. <u>SEC. HWY. 521</u>		DIST. No. <u>10.</u>	
CO. <u>RENFREW</u>		TWP. <u>RICHARDS</u>	
LOT <u>23.</u>		CON. <u>VII.</u>	
<h2 style="margin: 0;">PRELIMINARY</h2>			
APPROVED _____		SITE No. <u>30-14</u>	
BRIDGE ENGINEER _____		W.P. No. <u>254-6.</u>	
DESIGN <u>F.G.</u>	CHECK <u>Jan 65</u>	CONTRACT No. _____	_____
DRAWING <u>T.T.B.</u>	CHECK <u>Jan 65</u>	DRAWING No. <u>D-5498-P1</u>	_____
DATE <u>Sept. 64</u>	LOADING <u>H20S16</u>	_____	