

#63-F-210

W.P. # 5-60

Hwy. # 18

CANARD RIVER

CROSSING

Materials and Research Division

August 28, 1963

**William T. Irow & Associates, Ltd.,
1350 Jane Street,
Weston, Ontario.**

Attention: Mr. W. T. Irow

**To: R.F. 5-60, Hwy. #13, Canard River Bridge,
District #1, Chatham, 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 August 26, 1963.

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 October 2, 1963. 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 E.H.C. standards. To enable you to do this, we are enclosing a sample drawing with all the necessary explanations, together with a linen sheet for your drawing. You are also requested to provide the E.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.

**WJF/40ef
Encls.(2)**

Yours very truly,

AL

**cc: D. McCombie
A. Gater
G. U. Howell
J. Roy**

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

**L. D. Smith (2)
Mrs. T. Tate (Accounts)**

**Foundations Office
Gen. Files (2)**

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

Attention: Mr. S. McCombie

Mr. A. G. Stermac,
Principal Foundation Engr.,
Materials & Research Division,
Foundation Section.
October 9, 1963.

FOUNDATION INVESTIGATION REPORT BY -
William A. Trow & Associates Ltd. -
Proposed Crossing over Canard River,
Highway No. 18, Amherstburg, Ontario.
- W.P. 5-60 -- District No. 1 -

Attached we are forwarding to you the above mentioned report submitted by W. A. Trow and Associates Ltd. of Toronto. We have reviewed the report and found the finding well presented and are in agreement with the recommendations contained in the report.

Should there be any additional questions that you would like to discuss please feel free to contact this Office.

AGS/tt
Attach.

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. H. A. Tregaskes
H. D. McMillan
A. Gater
G. U. Howell
J. R. Roy
A. Watt

Foundations Office
Gen. Files

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

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

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

Project: J 1215

October 3, 1963.

Mr. A. Rutka, P. Eng.,
Materials and Soils Engineer,
Dept. of Highways of Ontario
Parliament Buildings,
Toronto, Ontario

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

Foundation Investigation
Proposed Crossing over Canard River,
Highway No.18, Amherstburg, Ontario.
W.P. 5-60

Dear Sirs:

In conformance with your authorization, dated August 28, we have made a foundation investigation at the site of a three span bridge to be built downstream of the existing five span Hwy. 18 crossing of the Canard River, north of Amherstburg, Ont.

We have found that the silty clay till underlying the river bed is not strong enough to support the weight of the bridge and therefore that end-bearing piles must be used. Limestone bedrock, in a porous state with some honeycombing, is located at a depth of approximately 46 feet below the river surface, or about 35 feet below the river bed. Cylindrical piles will encounter refusal at its surface, but H piles may penetrate

into it to some extent. The safe loading per pile, when driven to refusal, should equal its permissible capacity as a short column. The piles must be designed to support the negative load resulting from settlement of the clay under the embankment weight. If the fill is placed before bridge construction is begun, this drag force will not develop.

No embankment stability problem exists with this construction. The organic mud in the river bed will displace as fill is end-dumped from shore onto the route of the approaches to the bridge. The settlement of the fill at the bridge location is estimated to be in the order of $3\frac{1}{2}$ inches.

If you have any queries after you have reviewed the contents of this report we shall be pleased to discuss them with you.

Yours very truly,

W. A. Trow

William A. Trow (P. Eng.)

WAT/lt

DEPARTMENT OF HIGHWAYS OF ONTARIO
PARLIAMENT BUILDINGS,
TORONTO, ONTARIO.

FOUNDATION INVESTIGATION
PROPOSED CROSSING OVER CANARD RIVER
HIGHWAY NO.18, AMHERSTBURG, ONTARIO.
W.P. - 5 - 60

Project:J1215

October 3, 1963

William A. Trow and Associates Ltd.

TABLE OF CONTENTS

Project	Page	1
Site Description		1
Field Work		2
Subsoil Conditions		3
Foundations		6
Embankments		
a) Stability		7
b) Settlement		9
Conclusions and Recommendations		9

ENCLOSURES

Photographs of Site		
Field Vane Shear Strength Measurements in Peat	Table	I
Settlement Calculations	"	II
Borehole Logs, Holes 1 - 6	Dwgs.	1 - 6
Undrained Triaxial Test Results	"	7 - 8
Consolidation Test Results	Dwg.	9
Summary of Field Vane & Laboratory		
Shear Strength Measurements		
in Clay	"	10
Results of Stability Analyses (Total Stress)	"	11
Borehole Location Plan & Estimated Stratigraphy		

FOUNDATION INVESTIGATION
PROPOSED CROSSING OVER CANARD RIVER
HIGHWAY NO. 18, AMHERSTBURG, ONTARIO
W.P. 5 - 60

PROJECT

In order to improve traffic facilities along Hwy. 18 at this river crossing, a new three span structure, about 270 feet long, is to be constructed as a replacement of the existing bridge, which is located about 80 feet to the east. The bridge deck and the approaches to it will be at Elev. 583 feet, or about 12 feet above water level and approximately 18 feet above the deepest part of the river bed.

The purpose of the investigation was to determine the sub-soil conditions at the site and, with this information, to make positive recommendations regarding the type of foundation required for this bridge. The stability of the approaches to the bridge was also to be examined.

SITE DESCRIPTION

As indicated in the opening paragraphs, the site investigated is located approximately 80 feet to the west, or downstream, of the existing bridge which is the present Highway No. 18 crossing of the Canard River, about $4\frac{1}{2}$ miles north of Amherstburg, Ont. The general terrain at the crossing is flat, low-lying and swampy. There are no distinct river banks and the flow in the river is very sluggish. The existing bridge, a 5-span concrete structure 40 feet wide and 250 feet long, is supported on concrete abutments and on four intermediate rows of timber piles. The deck level is approximately at

elevation 578 feet, which is about 13 feet above the river bed. The average water level in the river at the time of the investigation was at elevation 571.5 feet, although it fluctuated somewhat due to rainfall.

A concrete abutment is situated approximately 150 feet east of the south end of the existing bridge and there are also rows of old timber piles standing 1 to 2 feet above water level, immediately along the west side of the bridge. These remnants of former crossing structures indicate that this new bridge will be the fourth structure to be erected at the site.

FIELD WORK

A total of 6 borings was made at this site, using conventional wet sampling methods. The holes were cased to full depth. Four of these boreholes were carried down to bedrock at the proposed bridge location, and the remaining two holes - located along the route of the approach embankments, - were terminated within the upper subsoil strata before reaching bedrock. Because the area to be investigated is under water, 5 of the holes were made from a raft.

Most of the clay samples were obtained by pressing, or driving when necessary, 2-inch I.D. Shelby tubes into the soil ahead of the borings. Upon recovery, the tubes were immediately sealed with low melting point wax to avoid losing moisture. Field vane measurements were made between sampling intervals and also to obtain an indication of the strength of the upper peat and organic muck lying in the river bed.

In order to determine the thickness of the peat stratum at the site, probing was done at 3 locations at right angles to the road centre line at the positions shown on the site plan.

Bedrock was cored in AX core size in two of the holes to a maximum depth of 10 feet, and the recovery of core was approximately 75 percent.

The locations of the boreholes and probings in relation to the existing and the proposed structures, together with the interpreted subsoil stratigraphy, are shown on the site plan drawing located in the pocket at the end of this report.

The elevations of the boreholes are referred to Geodetic Datum. The Geodetic bench mark, No.3021, is located on the south-east face of the retaining wall at the south-east end of the existing bridge. The elevation of the bench mark is given as 577.972 feet, as shown on D.H.O. Drawing No. E-4302-1, dated May, 1963.

SUBSOIL CONDITIONS

The results of the borings are described in detail on the borehole logs and are summarized as follows:

(a) River Mud

The river bed at the proposed crossing is generally covered by a layer of organic mud having a thickness ranging from about 7 feet in the mid-section of the river to 4 and 2 feet near the north and the south shore, respectively. This material was generally found to consist of completely decomposed organic material, with a trace of sand and silt. Field vane tests, performed in this material, indicated in situ shear strengths ranging from 20 to 80 p.s.f. The results of these vane tests are given in

Table 1. Occasional thin layers of fibrous peat were encountered at the bottom of the stratum, and the insitu shear strengths of this portion of the stratum were found to be in the order of 380 to 480 p.s.f. In view of the silty fibrous nature of the peat, these tests merely serve to indicate that it is soft and displaceable.

(b) Silty Clay Till

Underlying the soft mud is a stratum of silty clay till which represents the main body of soil underlying this site. This stratum is desiccated by weathering in the shallower sections of the river and this hardened condition extends, in diminishing form, to approximate Elev. 550 feet, or about 20 feet below the river bed in the vicinity of the north and south approaches to the bridge.

Atterberg limits were performed on representative samples from the clay, and the liquid and the plastic limit values obtained were 28 and 15% approximately. These limit values indicate that the clay is inorganic and of low to medium plasticity. Numerous fine gravel sizes up to $\frac{1}{2}$ inch thick were encountered in this clay mass.

Undrained triaxial compression tests were carried out on undisturbed samples from some of the borings and the results are plotted on Dwg. 10 together with the field vane test measurements. Based on these measured strength values, the consistency of the clay is indicated to be very stiff for the upper crustal portion, decreasing to firm below approximate Elev. 550 feet.

One consolidation test was carried out on a sample from the mid portion of the clay stratum and the pressure-void ratio curve for this test is shown on Dwg. 9. From this curve, it is estimated that the clay had been pre-consolidated in the past with

a loading of about 1500 p.s.f. in excess of the existing overburden pressure. Because of the need to remove fine gravel particles during the preparation of the sample, the results obtained undoubtedly indicate a more compressible condition than actually prevails in situ.

(c) Silty Sand

A layer of silty sand with occasional gravel was encountered underlying the till in Holes 3, 4 and 5, near the south end of the bridge. The thickness of the sand was found to be 8 feet in Hole 4 and 4 feet in Hole 3, whereas Hole 5 was terminated in the sand. The results of penetration tests indicate that the sand exists in a medium dense condition.

In borehole 1, a thin layer of sand $3\frac{1}{2}$ inches thick was found between the till and the underlying clayey silt. It is expected that this sand is a continuation of the granular deposits encountered at the south end of the bridge.

(d) Clayey Silt

Underlying this sand is a layer of clayey silt which has a thickness ranging from 7 feet in Hole 4 to 16 feet in Hole 2. From visual examination, the clayey silt contains some sand and gravel. The sand and gravel content appears to increase with depth. This stratum was generally found to be in a medium dense condition for the upper portion, increasing to dense with depth.

One Atterberg limits test was performed on a sample from the mid portion of the stratum; the liquid and the plastic limit values were found to be 17.8 and 11.1. These values indicate that the material of this stratum is essentially granular in nature and has a very low plasticity.

(e) Bedrock

Arenaceous limestone bedrock was cored in AX core size to a depth of 10 feet in Hole 3 and 5 feet in Hole 4. The recovery was about 75%. No bedrock was cored in Holes 1 and 2, although the fragments of rock obtained in the spoon sampler indicate that the surface of bedrock has been reached at these locations. The limestone contains numerous tiny leached cavities at some levels and it is quite porous. It was possible to drive the split spoon into rock in three of the borings.

FOUNDATIONS

As indicated in the opening paragraphs of this report, the existing bridge, comprising the Highway No. 18 crossing of the Canard River, will be replaced with a 3-span structure approximately 40 feet wide and 270 feet long. The new bridge will be located about 80 feet to the west of the existing structure and the proposed deck level is to be at Elev. 583 feet.

According to the results of this investigation, silty clay till is the first natural stratum, of any competence, to be encountered below river bed level. In the shallow sections of the river, north and south of the bridge, this clay till is fortified by a desiccated crust. In the immediate bridge area, however, most of this crustal material has been eroded away, leaving relatively weak soil of low bearing capacity below it.

In view of the low supporting capacity and the compressible nature of the clay till, the support of this structure on end-bearing piles, driven to refusal in bedrock, would seem to be the only

reasonable foundation scheme. Any type of end-bearing steel or concrete pile should be suitable for this application, and the permissible loading per pile, if driven to refusal, should be equal to its safe structural capacity when considered as a short column. Cylindrical piles are preferred to H piles, however, since they will reach refusal in or just above bedrock. H piles may penetrate into this rock to some extent, and, in view of the high concentration of loading, they may settle an additional amount when full load is applied, if they bear in honeycombed material. Piles driven to rock and cut off below the lowest channel level will be about 35 feet long.

It is expected that the settlement of the clay, resulting from the application of approach embankment fill, will exert a drag on the piles supporting the abutments of the bridge. For preliminary estimating purposes, it is assumed that the adhesion in the upper clay crust has the same magnitude as the undrained shear strength of the underlying firm clay and, therefore, that an adhesion force of approximately 600 p.s.f. will develop around each pile for the full embedded length of 27 feet in the clay. Using these values, it can be shown that a pile of 12 inch diameter will carry a dragging force in the order of 25 tons per pile. Even though this estimate is considered to be quite conservative, particularly for a steel pile, an additional load of this magnitude certainly eliminates timber piles as a supporting medium for the abutments.

EMBANKMENTS

(a) Stability

Since the river mud was revealed by vane tests and probings to be in a very soft condition, it should be easily displaced by the weight of the embankment fill. The use of additional fill, to ensure displacement, will not be necessary.

After the organic mud is displaced, the embankment fill at final grade level will reach a maximum height of 24 feet above the clay till surface at the north end of the bridge. Taking a unit weight of 130 p.c.f. for the fill, the corresponding pressure on top of the clay will be about 2300 p.s.f. at this location.

Stability analyses, in terms of total stresses, were carried out for the northern approach embankment. In the analyses, shown on Dwg. 11, both the "closed" and the "spill-through" type of abutments have been considered. Taking an average undrained shear strength of 650 p.s.f. for the clay in the vicinity of Borehole 1, the factors of safety of the embankment with respect to failure toward the south into the river were computed to be 1.22 and 1.37 respectively, for the types of abutments referred to above. Because of three dimensional resistance to failure, it is believed that the factor of safety for both embankment situations will be higher than these values.

In the analyses for the "spill through" type of abutment, a slope of 2 horizontal to 1 vertical was used for the fill placed in front of the abutment. Since the sides of the embankment will have a similar slope, the analysis is applicable for movements at right angles to the road centre line as well.

From these calculations, it is apparent that a "spill through" type of embankment will be safer. However, if this method of construction is used, provision must be made to protect the fill from erosion and in a manner that does not restrict the river flow. In view of these limiting requirements, the temporarily lower factor of safety, associated with the "closed" abutments, may appear less unacceptable.

(b) Settlement

According to the computations shown in Table 2, the estimated consolidation settlement of the clay underlying the embankment fill is in the order of 3 inches at the north end, and 4 inches at the south end of the bridge. No computation has been made for elastic compression or for the compression of the clayey silt stratum below the clay. Considering that disturbances were incurred in the preparation of this very gritty material for test, the neglect of these additional movements probably compensates, in part, for the over estimation of settlement obtained by using the test results in Dwg. 9. Since the bridge itself must be carried on end-bearing piles, a small amount of settlement of the adjacent embankments is not a matter meriting serious attention.

Taking the coefficient of consolidation value, $C_v = 0.114$ sq. ft./day indicated in Dwg. 9 for the pressure range applying, and assuming drainage at two boundaries, it can be shown theoretically that 50% consolidation of the thicker clay stratum at the south end of the bridge will be complete after a period of 10 months. This estimate is believed to be unduly conservative and a duration of 6 months or less is considered to be more appropriate.

CONCLUSIONS AND RECOMMENDATIONS

The foregoing observations and comments can be summarized briefly as follows:

1. The river bed at this proposed crossing site is generally covered by a thin layer of very soft organic mud. This mud is underlain by a stratum of firm silty clay till which has a very stiff desiccated crust under the north and south embankment approaches. At the actual bridge location most of the crust has been eroded away by the stream. The clay is in turn underlain by medium dense granular or semi-granular deposits which extend to limestone bedrock at approximately 40 feet below the river bed.

2. Because of the low shear strength and compressible nature of the clay, it is recommended that the proposed bridge be supported on end-bearing piles driven to refusal depth in the bedrock.
3. Due to the consolidation of the clay underlying the approach fill, each pile supporting the abutment structure should also be designed to carry the dragging force or the negative friction exerted by the surrounding consolidating clay. This requirement is not necessary if the approach fill is installed about 6 months prior to the driving of piles.
4. The mud, that overlies the site, is very soft and it should easily be displaced by the weight of the embankment fill.
5. No embankment stability problem exists if the fill grade level is maintained at or below Elev. 583 feet.
6. The consolidation settlement in the clay underlying the embankment fill is estimated to be equal to 3 inches approximately, at the north side of the bridge and about 4 inches at the south side. It is further estimated that 50% of the settlement will be completed within 6 months of load application.

JW/lt

J1215

Oct. 3/63



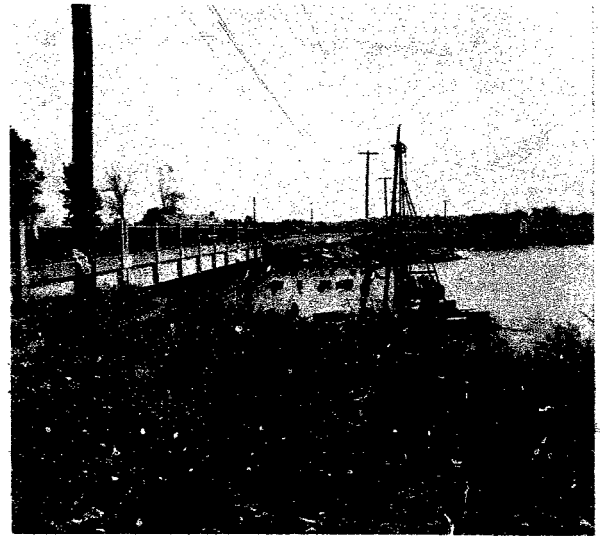
J. Wong, P. Eng.



Existing Bridge Viewing
From the South West
Drill Rig Near B.H. 4



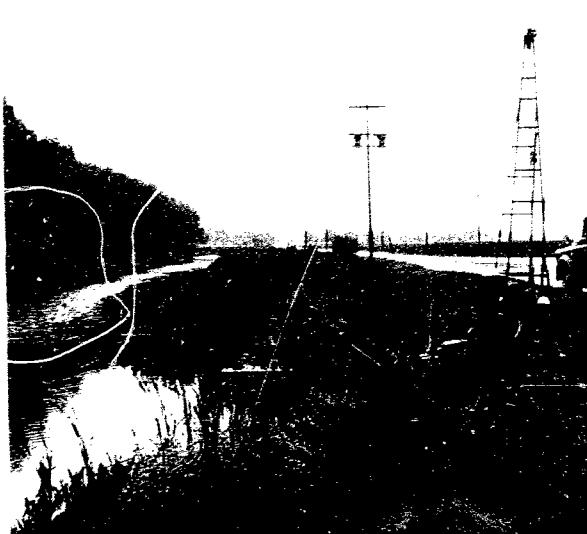
View Looking North
Drill Rig on B.H. 6



View Looking South
Drill Rig on B.H. 1



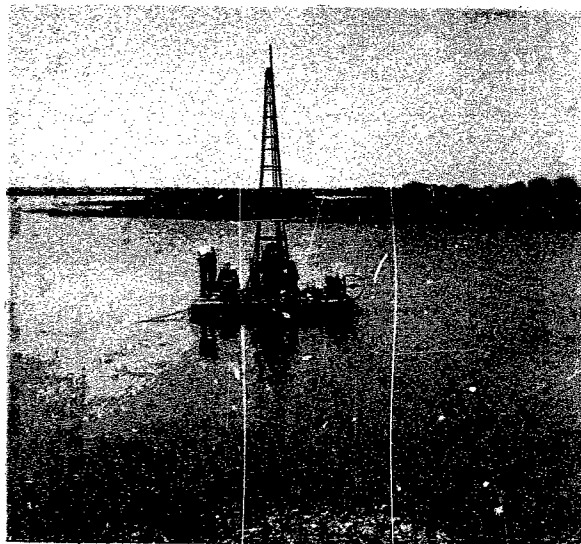
existing bridge Viewings
From the South West
Drill Rig Near B.H. 4



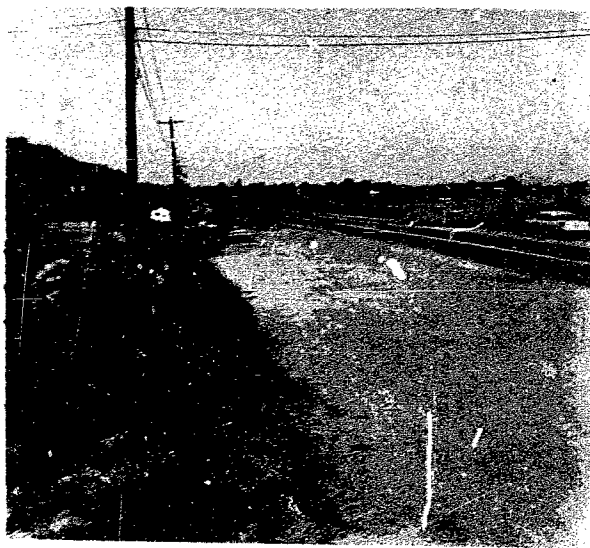
View Looking North
Drill Rig on B.H. 4



View Looking South
Drill Rig on B.H. 1



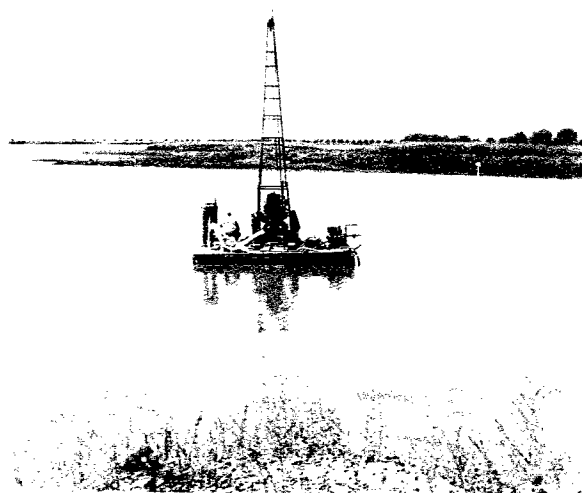
View Looking West
Drill Rig on B.H. 4



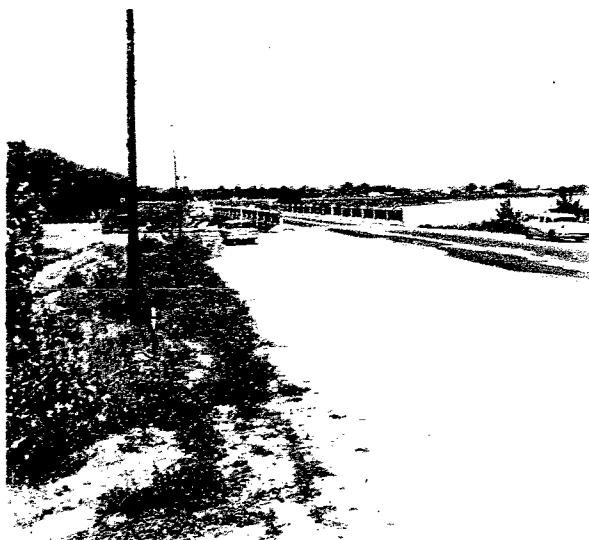
View Looking North
From the South End of Bridge



Viewing Upstream From
The North End of Bridge



new Lockhart est
drill rig on B.R. 4



Lockhart est
view from the north



view from the south
the north end of the est

TABLE 1

FIELD VANE SHEAR STRENGTH MEASUREMENTS
SILTY MUD IN RIVER BED

<u>Location of test</u>	<u>Depth below W.L. (or Elev. 571.5)</u>	<u>In-Situ Vane Shear Str. p.s.f.</u>	<u>Remarks</u>
8 ft. south of Hole 1	5.5 ft.	20	
	7.5 ft.	70	
Adjacent to Hole 2	8.5 ft.	30	
	10.0 ft.	60	
	12.0 ft.	380	Fibrous Peat
5 ft. south of Hole 5	2.5 ft.	480	Fibrous Peat.

TABLE 2CALCULATION FOR SETTLEMENT OF PROPOSED APPROACH EMBANKMENTS

The consolidation settlement occurring under an increase in pressure, Δp , is given by the expression:

$$S_c = \mu M_v \Delta p \Delta h$$

where:

μ is a factor to allow for over-consolidation
= 0.7 (estimated) for this soil **.

Δh is the thickness of soil acted upon by Δp .

M_v is the modulus of compressibility of the soil
equal to 0.01 sq.ft./Kip, as determined by the
consolidation test result, Dwg. 9.

A. APPROACH EMBANKMENT ON NORTH SIDE OF BRIDGE (Near BH 1).

Thickness of Clay = 17 feet.
Embankment Load on top of Clay = 2.34 K.s.f.

Depth Ft.	Depth Increment Δh ins.	Pressure Δp^* under centre of embankment Ksf	Settlement $S_c = 1.007 \Delta h \Delta p$ ins.
0		2.34	
3	36	2.34	0.588
6	36	2.34	0.588
9	36	2.33	0.585
12	36	2.32	0.582
15	36	2.29	0.575
17	24	2.26	0.378

Total Consolidation Settlement = 3.296 inches

3.3 inches approx.

* - Osterberg, J.O. (1957) "Influence Values for Vertical Stresses in a Semi-Infinite Mass due to an Embankment Loading" - Proc. 4th Inter. Conf. on Soil Mechanics and Foundation Engineering Vol.1, P.393

** - "A Contribution to Settlement Analysis of Foundations on Clay" - Skempton & Bjerrum, Geotechnique, Dec., 1957

Table 2, Cont.B) APPROACH EMBANKMENT ON SOUTH SIDE OF BRIDGE (Near BH 4)

Thickness of Clay = 27 ft.
 Embankment Load on Top of Clay = 1.8 K.s.f.

Depth Ft.	Depth Increment Δh ins.	Pressure, Δp , under centre of embankment K.s.f.	Settlement $S_c = .007 \Delta h \Delta p$ ins.
0		1.8	
3	36	1.8	0.452
6	36	1.8	0.452
9	36	1.79	0.450
12	36	1.77	0.444
15	36	1.75	0.440
18	36	1.72	0.430
21	36	1.68	0.422
24	36	1.64	0.412
27	36	1.60	0.402
Total Settlement			3.904 inches

4 inches approx.

The foregoing computations apply for pressure conditions existing under the centre of a continuous embankment 40 feet wide shoulder to shoulder and with side slopes of 2 horizontal to 1 vertical. The settlement under the end of the fill at a closed abutment will be about one half these values and for a "spill-through" arrangement the settlement will be somewhat more. Since the computed movements are not large and the abutments will be on piles, an exact determination of settlement at various locations is not considered to be necessary.

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 —■—

X LI

—○—

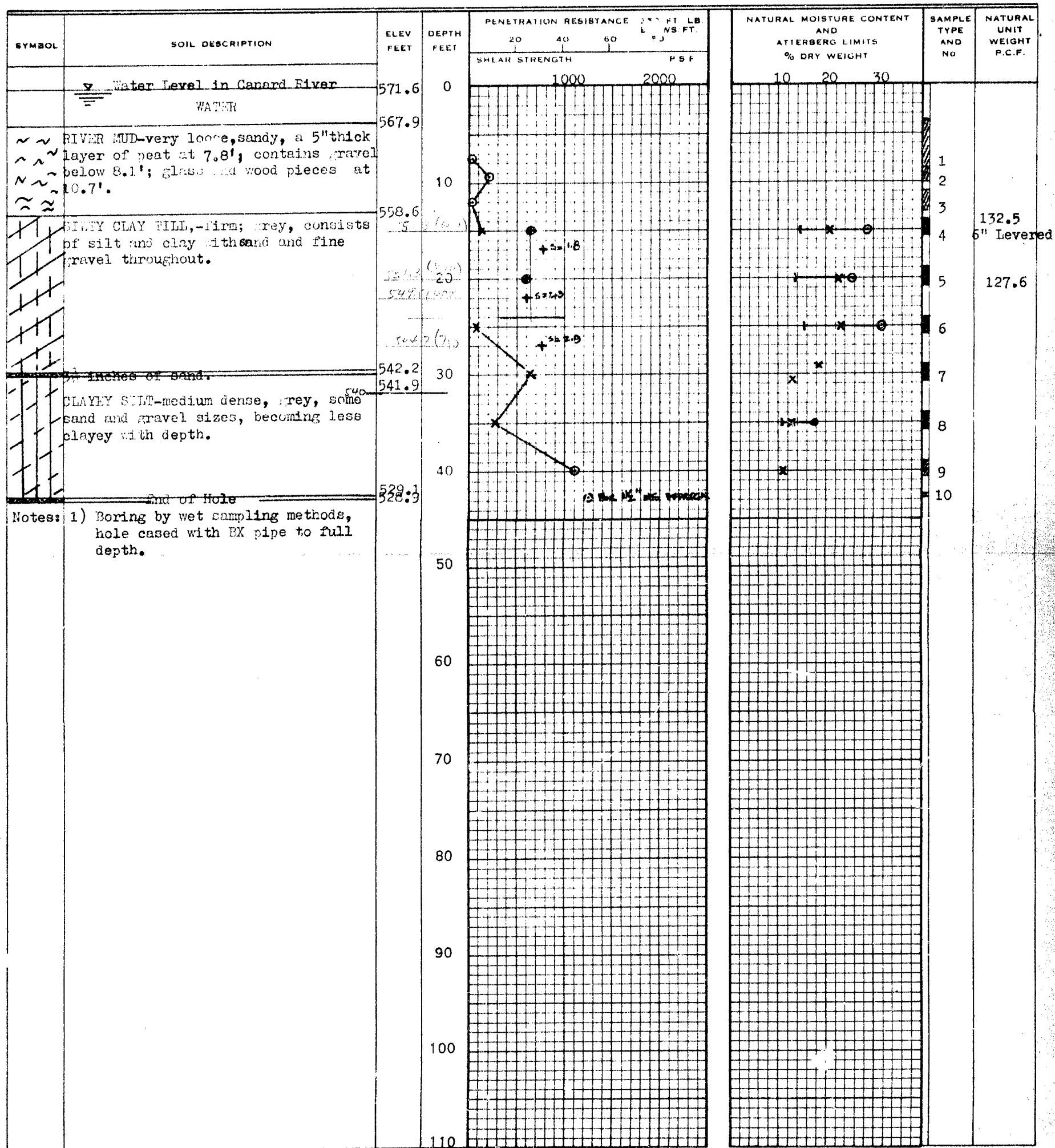
———

■

■

■

BOREHOLE NO. 1
 PROJECT Proposed Canard Bridge, W.P. 5-60
 LOCATION Amherstberg, Ontario
 HOLE LOCATION See Site Plan Dwg.
 HOLE ELEVATION 571.6 ft.
 DATUM Geodetic



LEGEND

DATUM Geodetic

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

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

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE		350 FT. LB BLOWS/FT 80	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT			SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
				20	40		60	10	20		
	W.L. in Canard River	571.5	0	1000		2000					
	WATER										
~ ~ ~ ~ ~ ~ ~ ~ ~	RIVER MUD-very soft, black, contains some gravel and wood pieces for the lower 6".	565.5	10								
	SILTY CLAY TILL-firm, grey, sconsists of silt and clay with sand and fine gravel throughout.	558.0	20							1	
		549.0	20							2	
		543.0	20							3	L = 12"
		540.0	20							4	L
		542.0	30							5	L
	CLAYEY SILT-medium dense, grey, some sand and gravel sizes, becoming less clayey with depth.	542.0	40							6	
		525.5	40							7	
	BEDROCK LIMESTONE	525.0	40							8	
End of Hole											
Notes: 1) As in hole 1.			50								
			60								
			70								
			80								
			90								
			100								
			110								

BOREHOLE NO. 3, 3A & 3B
 PROJECT Proposed Canard Bridge, W.P. 5-60
 LOCATION Amherstburg, Ontario
 HOLE LOCATION See Site Plan Dwg.
 HOLE ELEVATION 571.7 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 (S) ⊕^s

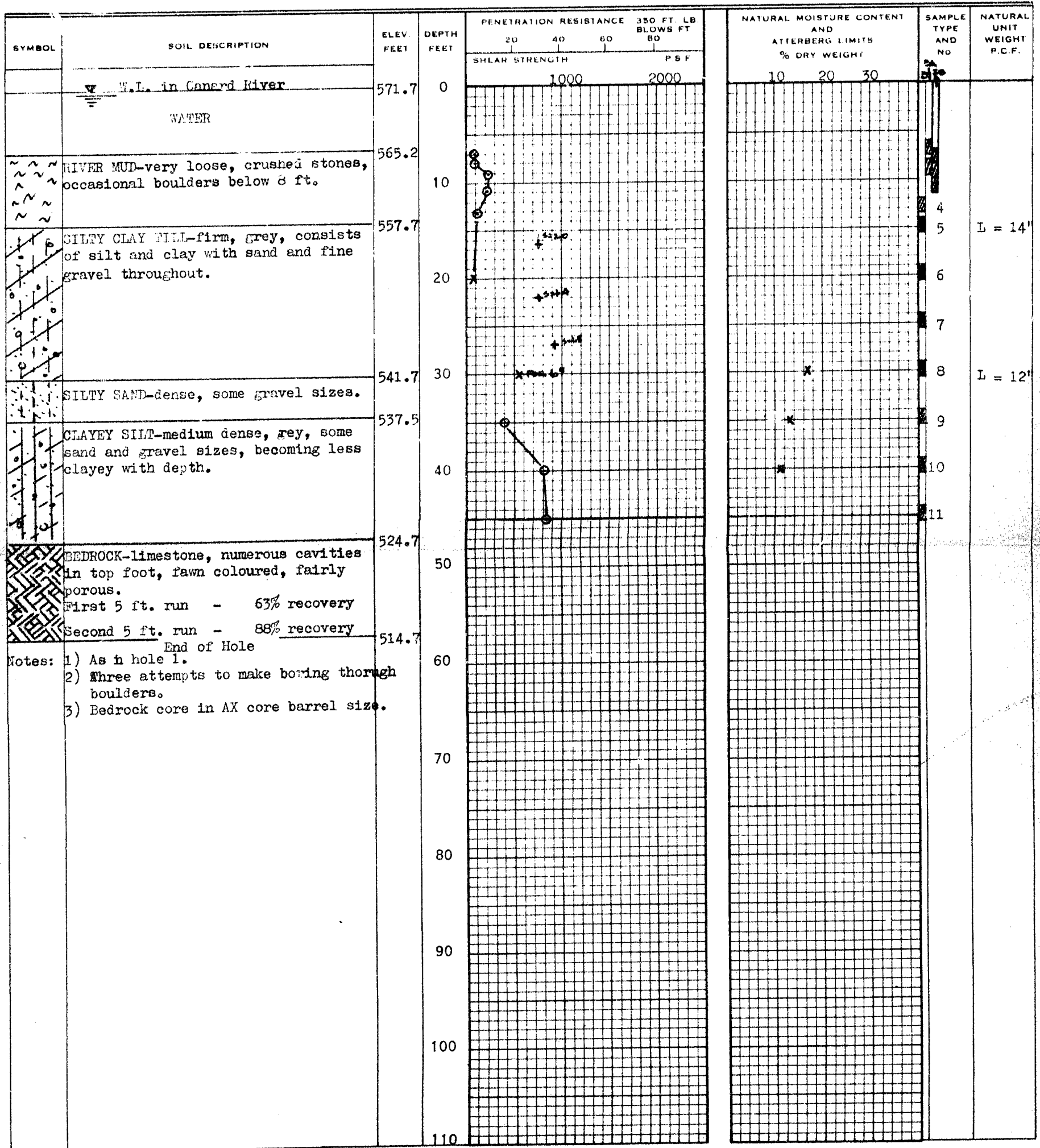
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS

LIQUID LIMIT —○—
 PLASTIC LIMIT ———

SAMPLE TYPE

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



BOREHOLE NO. 4
PROJECT Proposed Canard Bridge, W.P. 5-60
LOCATION Amhersiburg, Ontario
HOLE LOCATION See Site Plan Dwg.
HOLE ELEVATION 571.7 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 (S_v) +

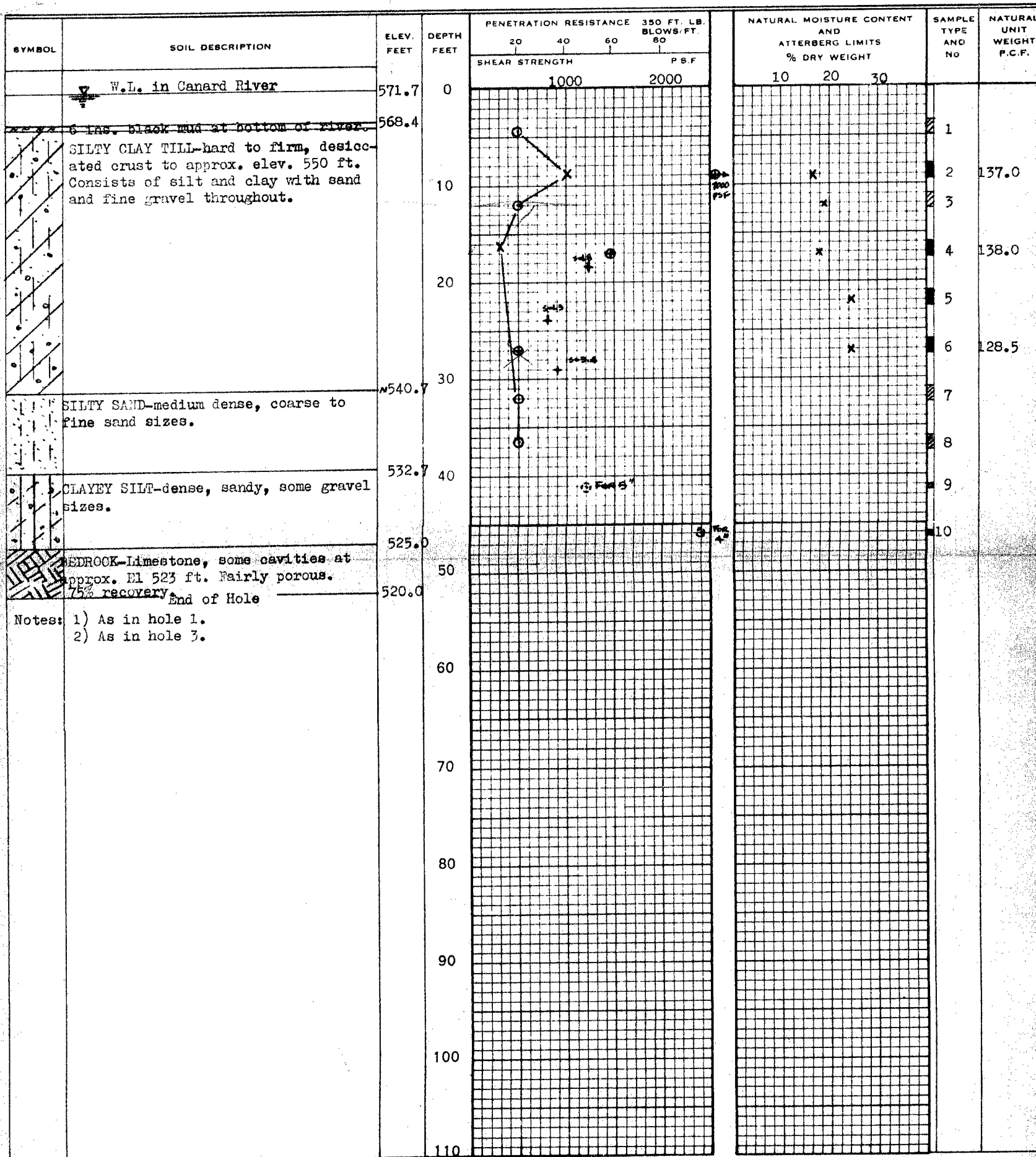
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT ———

SAMPLE TYPE

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



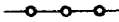
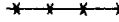

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION




DRAWING No. 5
PROJECT No. J1215


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 

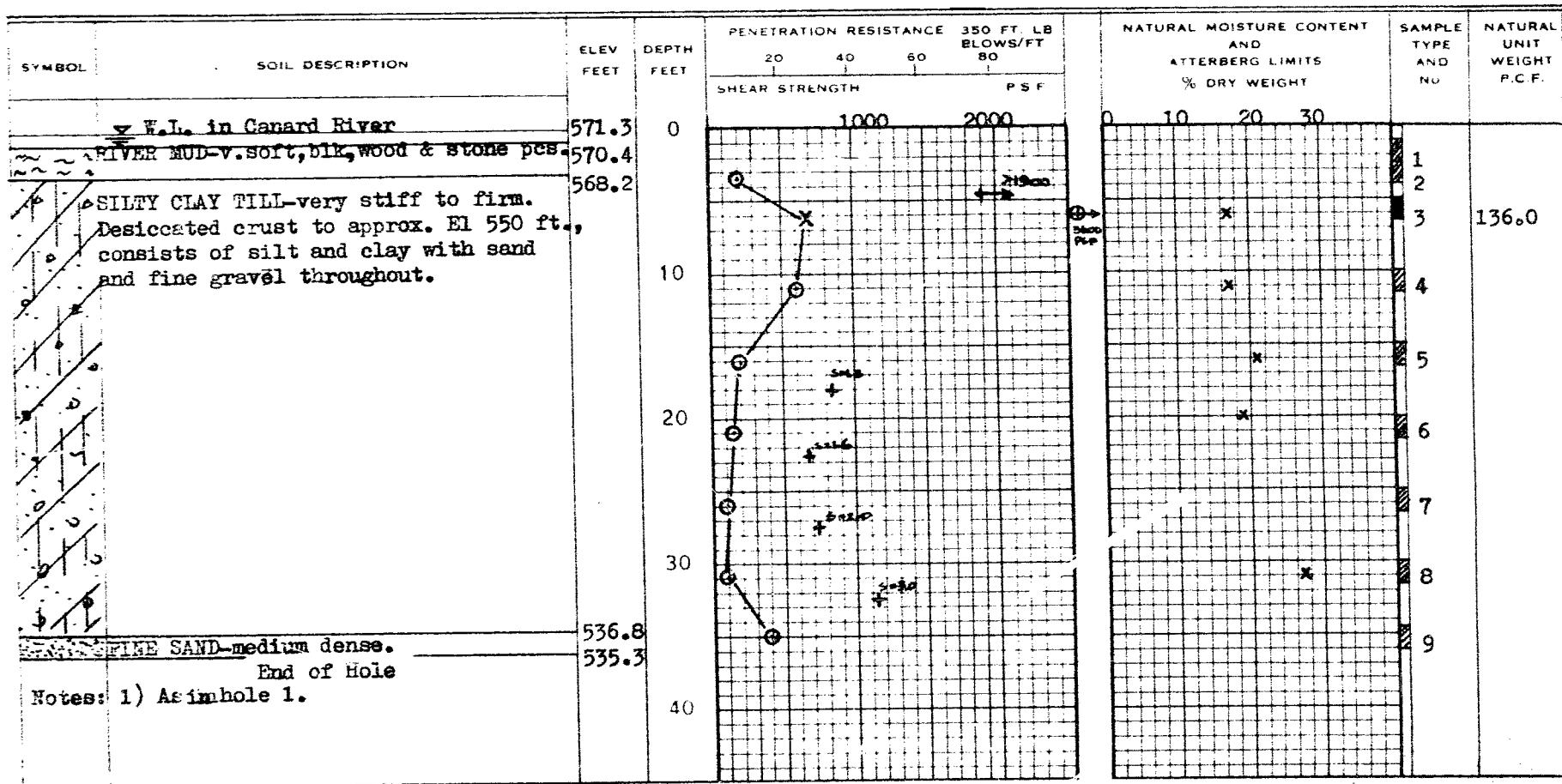
ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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

BOREHOLE No. 5
PROJECT Proposed Canard River Bridge
LOCATION Amherstburg, Ontario
HOLE LOCATION See Site Plan Dwg.
HOLE ELEVATION 571.3 ft.
DATUM Geodetic



WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 6
PROJECT No. J1215

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) +^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

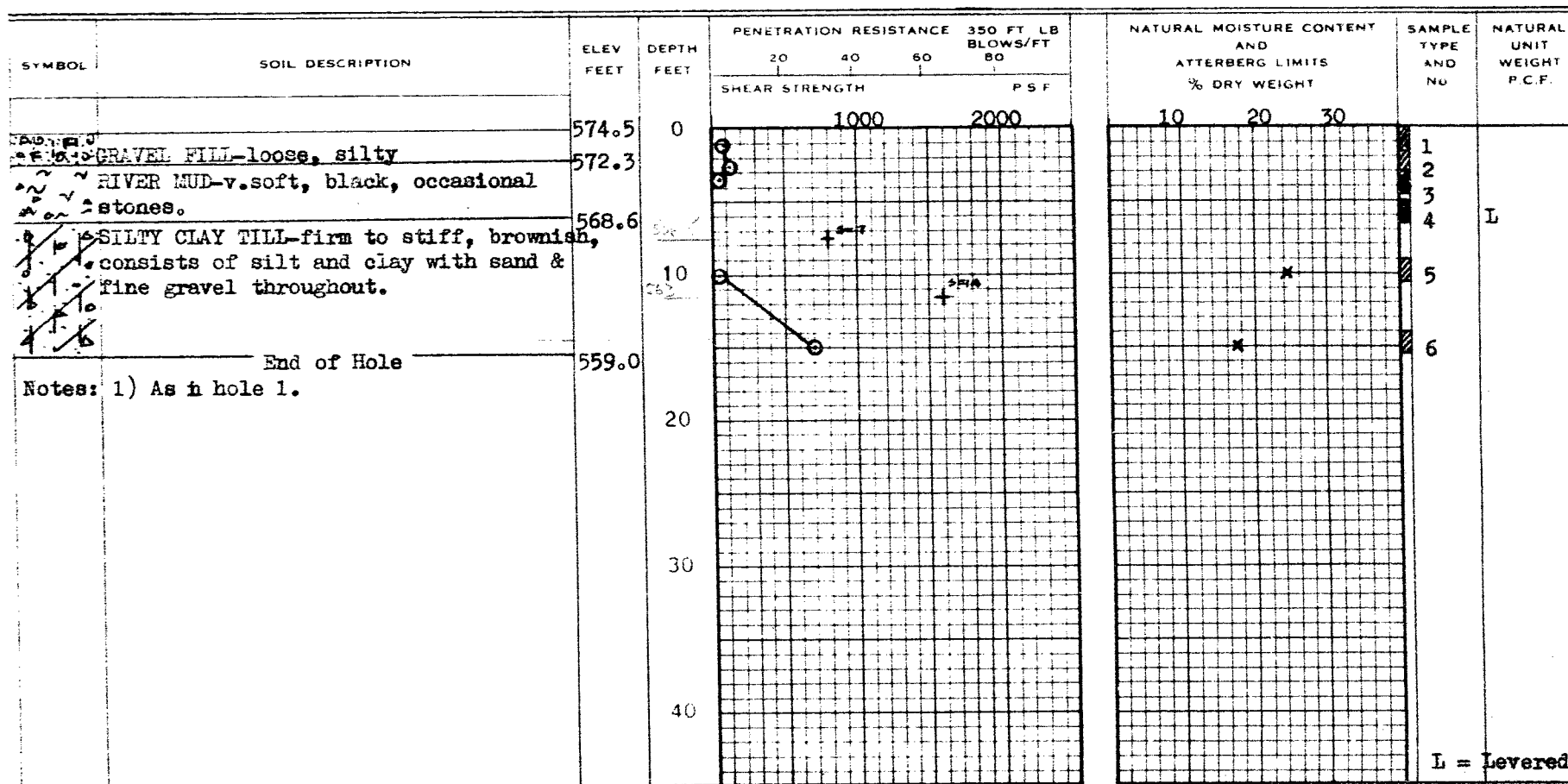
ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT ———

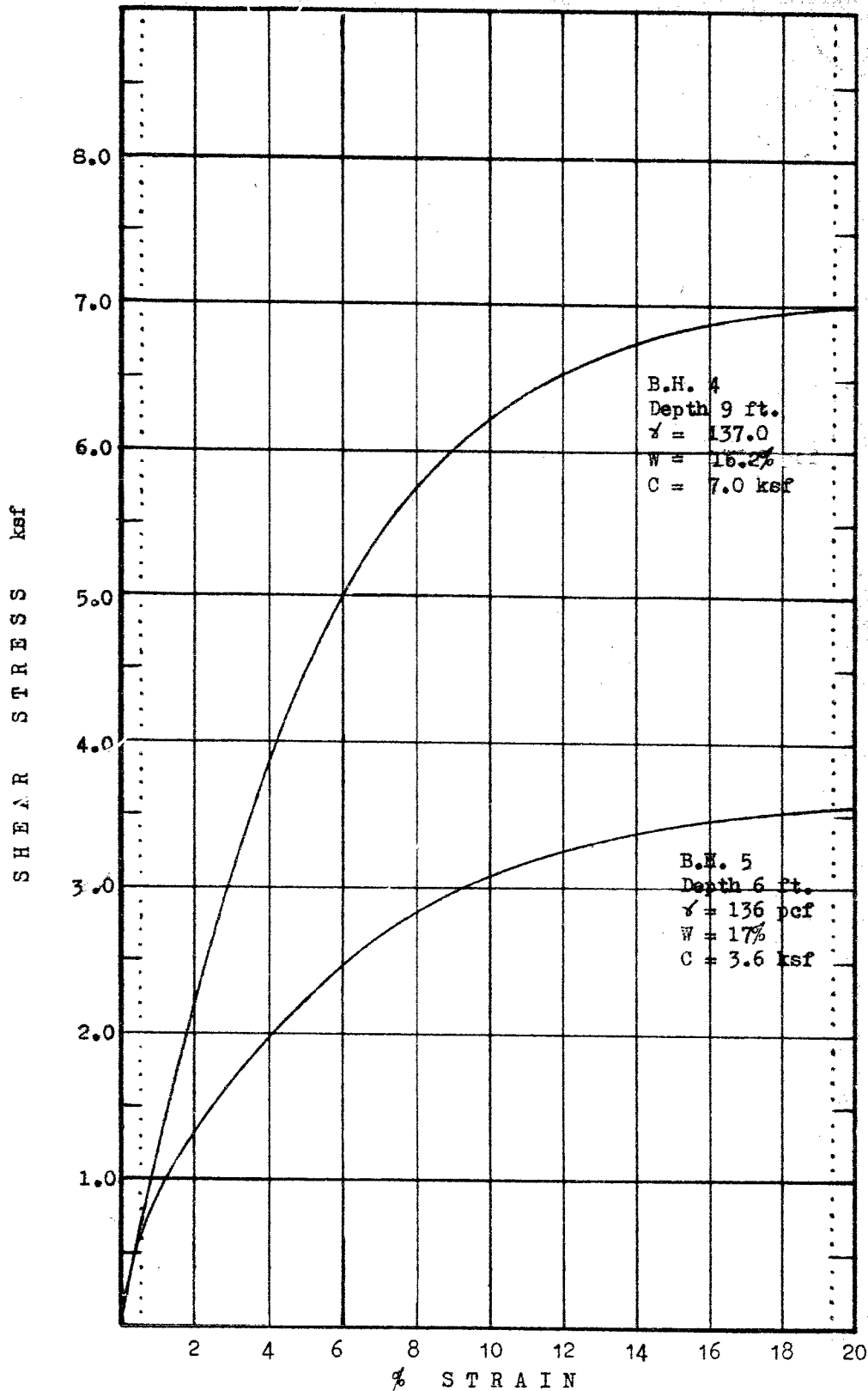
SAMPLE TYPE

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

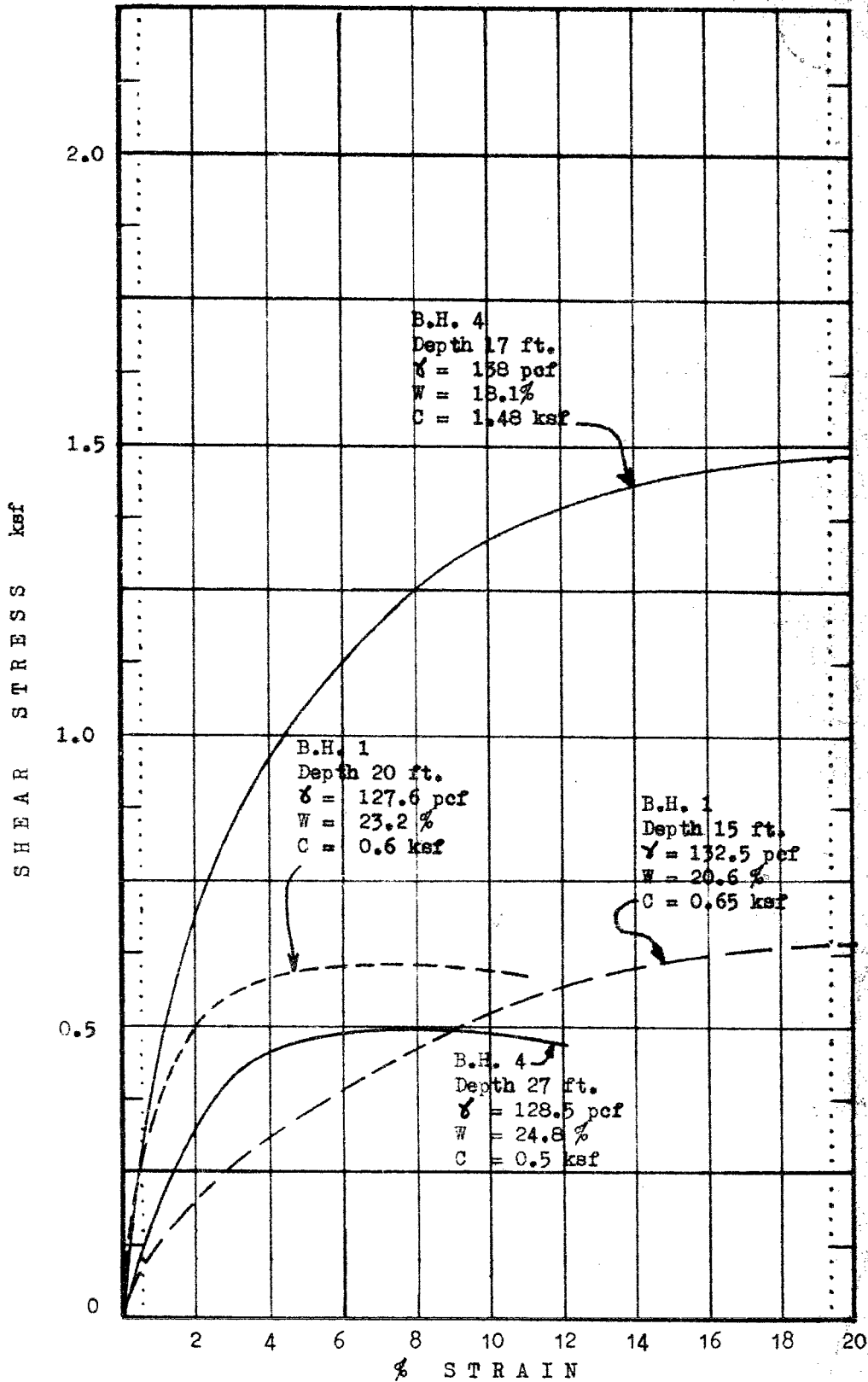
BOREHOLE NO. 6
PROJECT Proposed Canard River Bridge, W.P. 5-60
LOCATION Amherstburg, Ontario
HOLE LOCATION See Site Plan Dwg.
HOLE ELEVATION 574.5 ft.
DATUM Geodetic



L = Levered



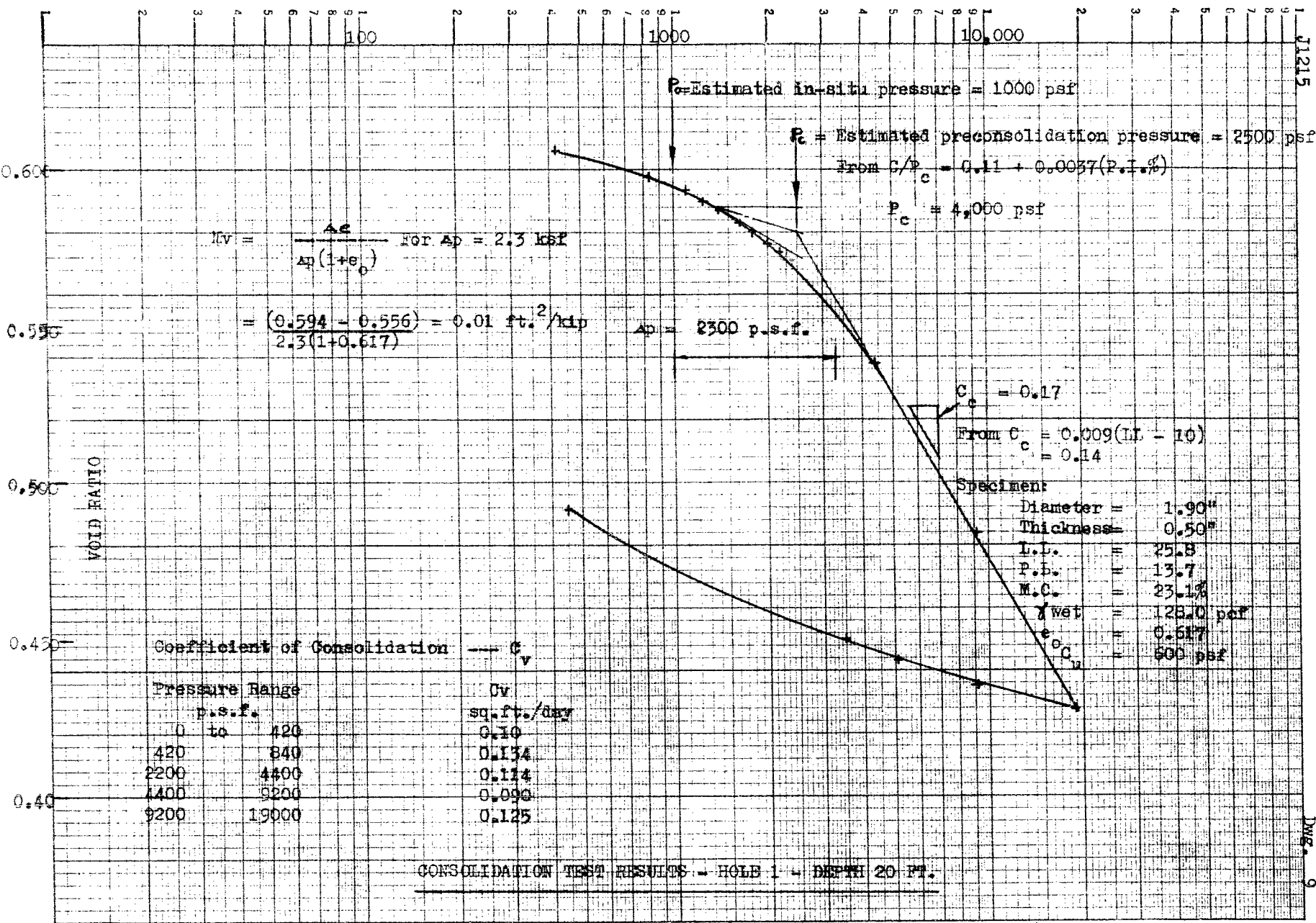
UNDRAINED TRIAXIAL TEST RESULTS

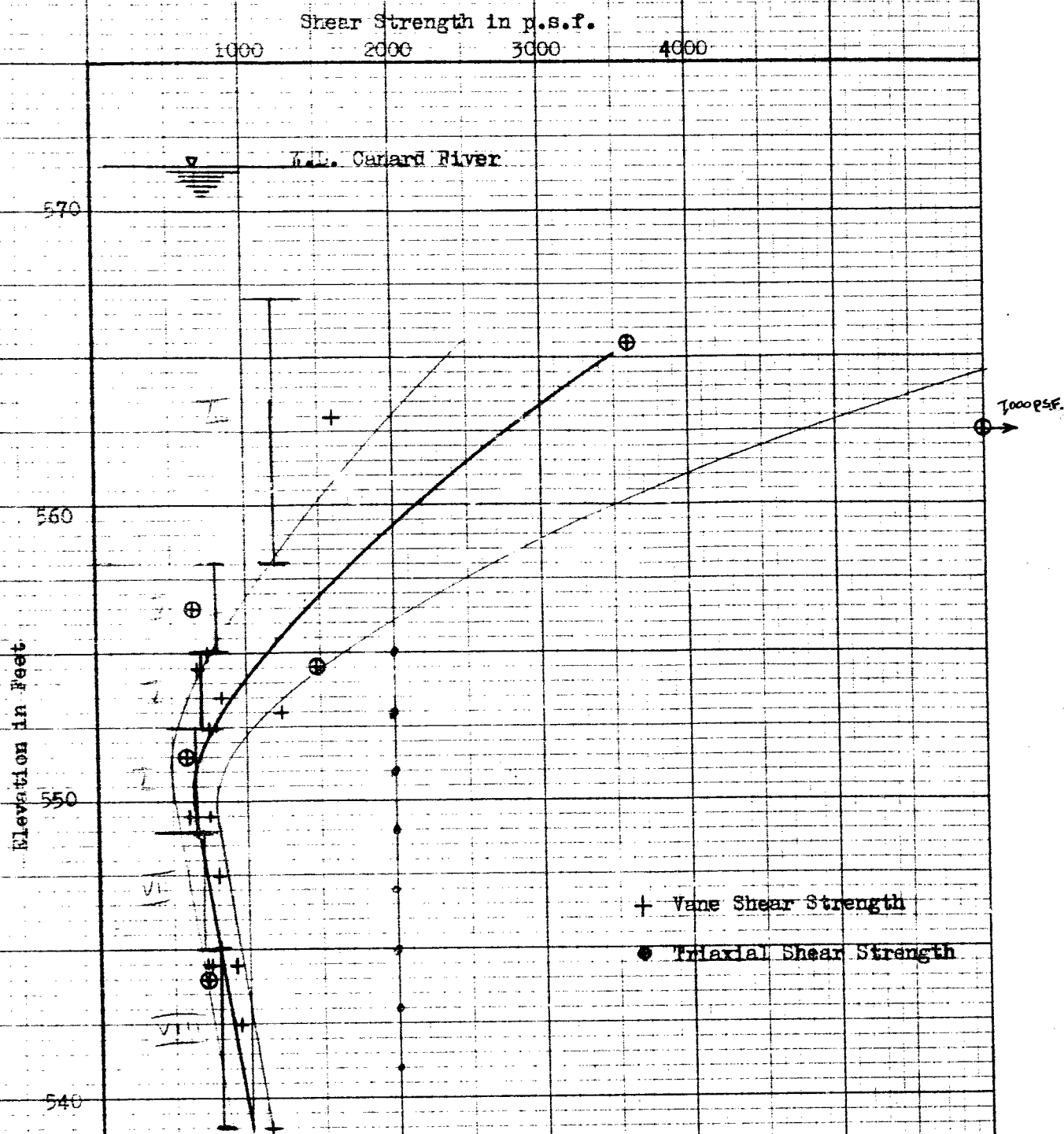


UNDRAINED TRIAXIAL TEST RESULTS

11215

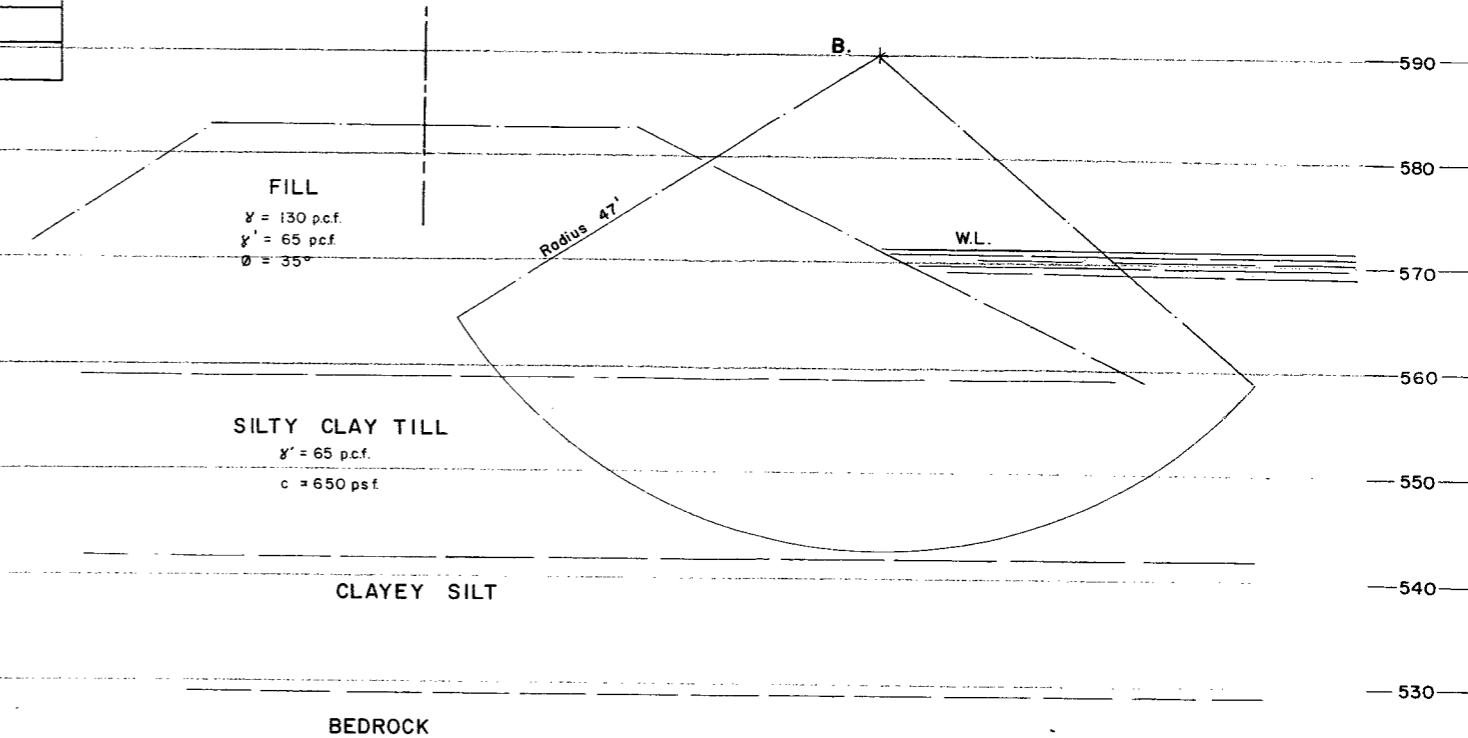
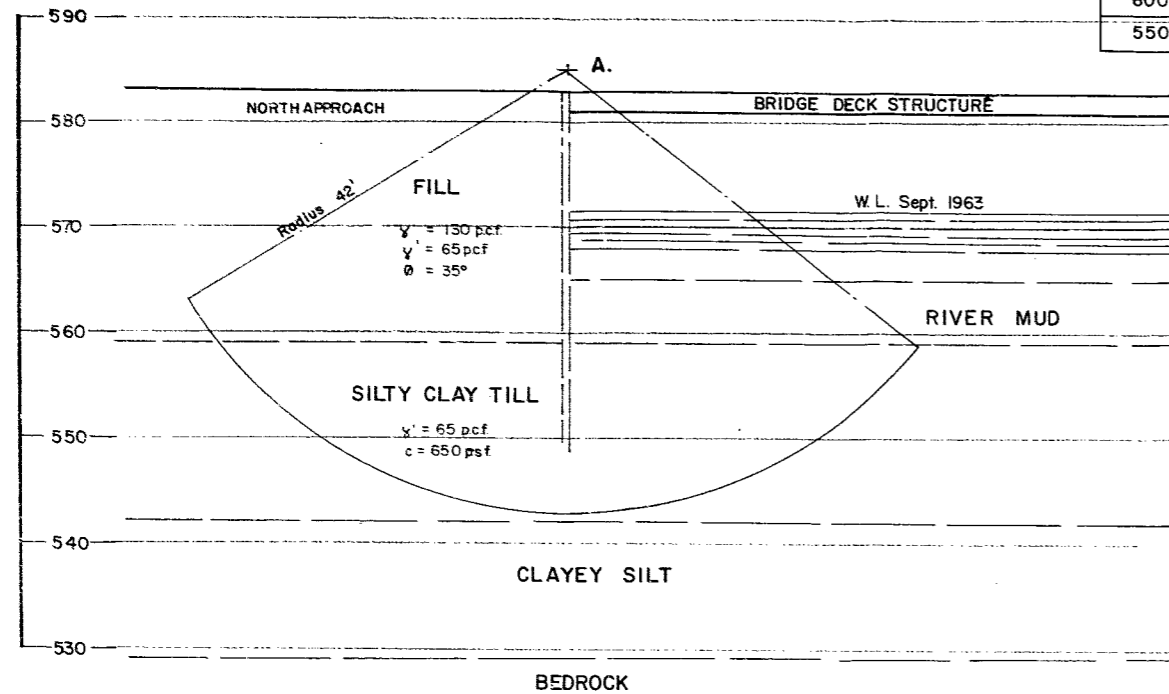
DMG 9





SUMMARY OF FIELD VANE & LABORATORY SHEAR STRENGTH MEASUREMENTS

C p s f	FACTOR OF SAFETY	
	A	B
650	1.22	1.37
600	1.14	1.28
550	1.03	1.18



SCALE : HOR. & VERT. 1 IN. = 10 FT.

STABILITY ANALYSES (TOTAL STRESS)
FOR
EMBANKMENT AND APPROACH
DWG. NO. 11

Foundation Investigation

Materials and Research Division

August 28, 1963

William A. Frowe Associates, Ltd.,
1350 Jane Street,
Brampton, Ontario.

Attention: Mr. W. A. Frowe

Re: W.P. 5-60, Hwy. #13, Canard River Bridge,
District #1, Chatham, 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 August 26, 1963.

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 October 2, 1963. 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 B.M.C. standards. To enable you to do this, we are enclosing a sample drawing with all the necessary explanations, together with a linen sheet for your drawing. You are also requested to provide the B.M.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.

RD/MSF

Encls. (2)

Yours very truly,

al

cc: J. McCosbie

J. Gater

G. E. Howell

J. Roy

H. D. Smith (2)

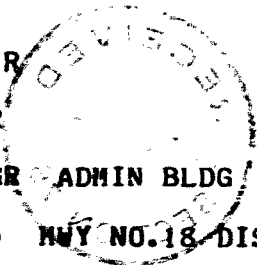
Mrs. T. Tate (Accounts)

J. Lutka,

MATERIALS AND RESEARCH ENGINEER

Foundations Office
Gen. Files (2)

RECEIVED
OCT 1 11:05



L

LOND DOWN 7 OCT 1/70 11.20A VR
A P WATT REG BRIDGE PLNG ENGR
CC F MCCOMBIE BRIDGE PLNG ENGR ADMIN BLDG
RE CANARD RIVER BRIDGE WP5-60 HWY NO.18 DISTRICT 1 CHATHAM

THIS IS TO CONFIRM THE CONTENTS OF OUR TELEPHONE
DISCUSSION OF WEDNESDAY SEPT 30/70 REGARDING THE ABOVE SUBJECT
THE STABILITY OF THE APPROACH EMBANKMENTS WITH A GRADE AT
ELEVATION 592.0 WAS CHECKED AND IT WAS FOUND THAT IN ORDER TO
ACHIEVE A SATISFACTORY FACTOR OF SAFETY A 10 FT BERM WILL HAVE
TO BE BUILT FOR THE NORTH APPROACH EMBANKMENT THE BERM TOP TO BE
AT ELEVATION 573.0 THE BERM SHOULD BE BUILT IN THE LONGITUDINAL
AS WELL AS PERPENDICULAR DIRECTION THE DETAILS OF THE EXTENT OF THE
BERM ON BOTH SIDES OF THE EMBANKMENT WILL DEPEND ON THE GRADE
ELEVATION THE VERY SOFT RIVER MUD IS TO BE DISPLACED OR REMOVED
BEFORE ANY FILL PLACEMENT

A G STERMAC MAT AND TEST OFFC

AW

MEMORANDUM

TO: Mr. A. G. Stermac
Principal Foundation Engineer
Downsview, Ontario

FROM: Bridge Planning
Southwestern Region

ATTENTION:

DATE: September 23, 1970

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 5-60-00, Bridge Site 6-120
Canard River Bridge
3.5 miles north of Amherstburg
Hwy. 18
District 1, Chatham

Attached please find one copy of the bridge site plan B-4302-1, one portion print of the plan B-244-5, and one portion print of the profile C-244-3 showing the recommended location of Highway 18 Line "D" (1963).

There is a Foundation Investigation conducted under the above work project by William A. Trow and Associates Ltd. with the report written on October 3, 1963. The conclusions and recommendations under item 5 suggest no embankment stability problem exists if the fill grade level is maintained at or below elevation 583.0 feet.

As the vertical clearance of the soffit required under the Navigable Water Protection Act could be (12 feet)⁺ above the summer water level of 572.5 geodetic, would you kindly let me know if there would be any embankment stability problems with a grade of approximately (589) in the vicinity of the proposed new structure. The tentative location of the structure is as per the attached Hydrology Report BW 625. (15 FT)
SEP 24/70
S. McCOMBIE

I would also like to know the stable height fill can be placed without creating embankment stability problems.



A. P. Watt
Regional Bridge Planning Engineer
Southwestern Region

APW/pm
Atch.

c.c. S. McCombie
A. Crowley
W. Zonnenberg
A. McConnell
J. D. Harris

MEMORANDUM

TO: Mr. G. Scott,
Bridge Location Engineer,
Bridge Division.

FROM: J. D. Harris

DATE: August 19, 1963.

OUR FILE NO.

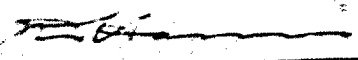
IN REPLY TO

SUBJECT: Canard River Crossing
Highway 13 District 1
M.P. 5-60 BW 625

Further to our memorandum of the 13 March 1962, recommendations for line 'D' are as follows:

1. Skew opening approximately 230' measured parallel to centre-line of new highway.
2. Angle of skew: 45° left-handed.
3. Location of bridge centre-line: Sta. 88+10 (line 'D').
4. Elevation of lowest point of soffit 577.0 or higher.
5. As mentioned previously, the number of piers should be kept to an absolute minimum. Upstream cutwaters on the piers should be sloped at 1:1 between elevations 570 and 578, and protected with steel angle nosings. The piers should be smooth-sided in order to reduce the possibility of ice jamming.
6. The two existing north abutments should be left in place, and the north river bank streamlined for about 250' upstream from the new abutment.
7. Both existing south abutments should be removed. The south approach fill at the existing bridge should be cut back 40' from the existing abutment face. The south approach fill at the disused road should be cut back 100' from the old abutment face.
8. All piles, including any which may be submerged, should be removed or cut off level with the river bed, to reduce ice jamming. All other obstructions in the channel, including fill, should be removed down to the level of the natural river bed.
9. On the assumption that bearing piles will be needed for the new structure, no special scour protection will be necessary.

JDH:go


J. D. Harris,
for B. Wilkie,
Bridge Hydrology Engineer.



ONTARIO

DEPARTMENT OF HIGHWAYS

Bridge Division.

Memo to Mr. S. MacIsaac,
Bridge Planning Engineer,
Administration Building,
TORONTO, Ontario.
From J. S. Harris

Date March 13, 1962.
Subject Canard River crossing
Hwy. #18 - District #1
H.P. 5-60 5W625

Attn: Mr. G. White

The area of watershed drained above the proposed crossing is approximately 120 square miles, consisting largely of fairly impervious clays and clay loam soils.

The river was originally used for commercial navigation in connection with the transport of lumber, but the headroom of about 2' at the existing bridge prevents navigation by all but the smallest boats. This clearance appears to be about 3' less than that of the original bridge.

The present bridge has a length of 250' between abutments, of which about 80' is ineffective due to the old fill immediately downstream.

Ice jamming is reported to occur occasionally at the existing bridge and at the old abutments just upstream. One such jam produced a backing up of nearly 2 1/2' at the bridge, and the pressure against the girders has moved part of the structure laterally by about 3". The upstream flood elevation was approximately 576.0, while that downstream was nearly 573.5. If ice jamming could be eliminated at the new bridge, the new H.W.L. would be around 573.5, but would depend to a large extent on conditions in the Detroit River, and on the direction of the wind.

It is understood that some doubt has arisen as to the accuracy of bench marks used in this area, and it should be noted that the elevations quoted above are based on profile PL157.

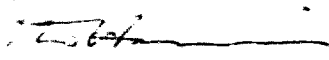
The following recommendations are tentative only, since the new line of the highway has not yet been fixed.

Canard River Crossing
W.P. 5-60

Tentative recommendations

- (1) Span, location and angle of skew to be determined when the line of the new highway has been fixed.
- (2) Minimum soffit elevation: 577.0 from purely hydrologic considerations. However, local residents indicate that they will request an elevation of between 577.0 and 587.0 for navigation purposes.
- (3) To minimize the possibility of ice jamming, the number of piers should be kept to a minimum and they should have smooth sides between N.W.L. and H.W.L.
- (4) No special scour protection will be required if the structure is founded on piles.

JDH/ea


J. D. Harris
for E. Wilkie,
Bridge Hydrology Engineer.

HWY. #18 and CANARD RIVER :

W.P. 5-60

RE: STABILITY ANALYSIS
NORTH APPROACH.

<u>HWY. #18 PROFILE GRADE</u>	<u>HT OF FILL ABOVE BOTTOM OF RIVER</u>	<u>LENGTH OF BERM REQUIRED AT ELEV 573</u>
592	33'	20 feet.
589.5	30.5'	10 feet
587	28'	5 feet

BTD.

M.R. STU MC COMBIE
3492

Message to Mr. J. J. ...

22 OCT 1970

1-2-3

NOTE:

ORIGINALLY A 10 FT BERM WAS GIVEN FOR APPROACH EMBANKMENT
ELEVATION 592.0. LATER IT WAS FELT THAT A SOMEWHAT HIGHER
FACTOR OF SAFETY WOULD BE DESIRABLE (APPROACHING 1.3) AND A 20 FT
BERM WAS SUGGESTED. CONSEQUENTLY FOR EL. 592.0 THE RECOMMENDATION
IS SOMEWHAT FLEXIBLE.

OCT 26, 1970

$$F.S. = \frac{\sum [c + (W \cos \alpha - \mu) \tan \phi]}{\sum W \sin \alpha}$$

CHECKED

LOCATION

MADE BY

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RADIUS

CIRCLE NO

Slice No.	OVERTURNING MOMENT $W \sin \alpha$							RESISTING MOMENT $W \cos \alpha \tan \phi$								
	γ or γ'	b	h	W Area	$S. \alpha$ W = Weight	α r	① $M_o = \sum \pm W_r$	c	L	R	② c L R	$\tan \phi$	β	$\cos \beta$	③ $\pm W R \cos \beta / \tan \phi$	$M_r = ② + ③$
1	15	3	5	5.25	0.743	70°	5.31					0.577	70.5	0.334	1.09	1.09
2	25	10	12.5	22.16	0.788	52°	22.21					0.577	52	0.616	10.00	10.00
3	25	10	10	47.5	0.515	31°	24.60									
4	25	10	8	32.25	0.165	95°	6.42									
								cul (c = 600 psf)								
								$= \frac{600 \times 654}{1000}$								
5	12	8	4	15.0	-0.996	-55°	-1.44									
6	65	10	20.5	13.45	-0.309	-18°	-4.13									
7	65	7	12.5	5.61	-0.405	-29°	-2.76									
8	65	4	6.5	1.6	-0.603	-35°	-1.05									
9	65	3	2.5	0.27	-0.602	-48°	-0.19									
								$c_u = 650 \text{ psf}$								
								$F.S. = \frac{53.71}{48.97} = 1.1$								
								$F = \frac{\sum ② + ③}{\sum ①} = \frac{50.41}{48.97} = 1.03$								

$$\sum ① = 48.97$$

$$c_u = 650 \text{ psf}$$

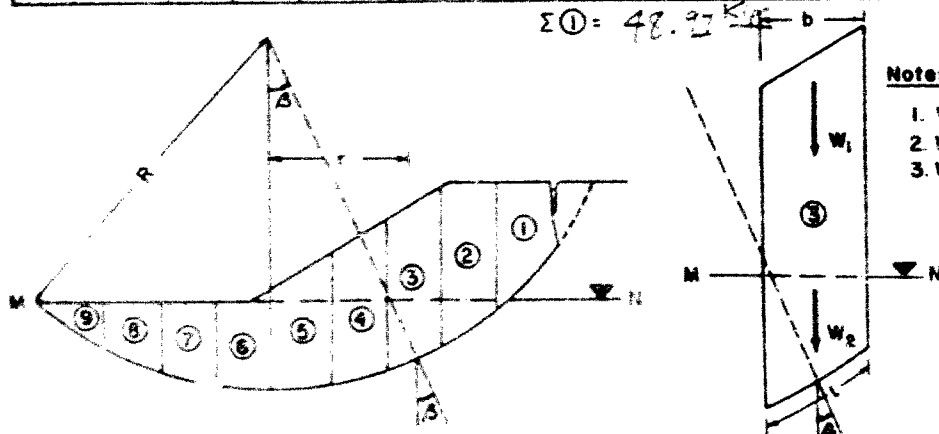
$$\sum ② + ③ = 50.41$$

$$F.S. = \frac{53.71}{48.97} = 1.1$$

$$F = \frac{\sum ② + ③}{\sum ①} = \frac{50.41}{48.97} = 1.03$$

- Notes:-
1. $W = W_1 + W_2$
 2. W_1 Weight of Slice above MN, computed from Bulk density
 3. W_2 Weight of Slice above MN, computed from Submerged density

SOIL PROPERTIES				
TYPE	γ	γ'	c	ϕ



STABILITY ANALYSIS NORTH APPENTH TO CANALS RIVER - HILL'S STRUCTURE

592
559
33

600
590
580
570
560
550
540

