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Project: J1775

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Associates Ltd.

Norfolk County Road System,  
Superintendent's Office,  
Simcoe, Ontario

February 18, 1965

Attention: Mr. W.C. McDowell, O.L.S., P.Eng.,  
County Engineer

Subsoil Investigation  
Proposed Bridge Over Big Creek  
Near Pt. Rowan, Ontario

Dear Sirs:

In conformance with your authorization given in mid December, 1964 a subsoil investigation has been completed at the above site.

The field work which consisted of 4 sampled borings was carried out during the period January 6 to 9 inclusive. Two of the borings were made at the present bridge location and the remaining two shallow holes were put down on the east hillside and in a tributary ravine to the east in order to appraise subsoil stability under the high fill proposed for this area.

A summary of the conclusions and recommendations arising from this study is given below:

1) The subsoil at the bridge site consists of sand and clay fill to a depth of about 10 feet underlain by loose sand to a depth ranging from 30 on the east side to 40 feet on the west. Stiff clay underlies the sand to a depth of at least 100 feet (the maximum penetration of the boreholes).

**BA 2103**  
**SITE # 20-128**



2) Support for the structure must be provided by friction piles. One foot diameter piles driven 18 feet into the clay may be designed for allowable loads varying from 26 kips to 36 kips per pile depending on the pile material and the location. It is recommended that the piles be driven no deeper than 20 feet as the clay becomes less stiff at this depth. The settlement of the piles under the recommended loads should be within tolerable limits.

3) No stability problem should arise if the fill is placed directly on the existing surface. The settlement of the subsoil due to the addition of fill will also be within tolerable limits.

4) It is recommended that a spill-through design for the abutments be adopted, thus reducing the earth pressure.

These points are expanded in the sections which follow.

#### SITE DESCRIPTION

The site is located at County Road 9 and Big Creek, approximately 2 miles west of Highway 59, near Pt. Rowan, Ontario, (see Dwg. 1). A 72 foot long steel truss bridge presently provides access over the creek. This bridge appears to be in good condition but is too narrow for present day traffic.

Big Creek meanders through a flood plain which is about 200 feet wide. At this crossing location its course is along the base of the east slope. Its flow at the time of the field work was about 1.7 feet per second.

The surficial soil on the east hillside appears to be in a state of long term creep as evidenced by the humicky valley slopes shown in the accompanying photographs.

#### PROJECT

It is proposed to replace the existing bridge with a larger two span bridge. The proposed road grade is such that fill up to a thickness of about 18 feet will be required.

#### FIELD WORK AND SUBSOIL CONDITIONS

Four boreholes were put down at the site using wash boring techniques. The holes were cased for a portion of their depth to prevent caving. Samples were obtained in a disturbed state with a split spoon and in an undisturbed condition in Shelby tube samplers. The split spoons and Shelby tubes were driven into the soil with an energy conforming to the requirements of the Standard Penetration Test. Cone penetration tests were performed beside 3 boreholes to obtain a general indication of the soil conditions before boring progressed. In-situ vane tests were made to determine the shear strength properties of the soil.

The soil encountered at each borehole location is shown in detail on the borehole logs, (Dwgs. 2 to 5 inclusive). The subsoil stratigraphy, as interpreted from the borehole data, is shown on Dwg. 1. At the proposed bridge site (boreholes 1 and 2) fill exists to a depth of about 9 feet. Underlying the fill is a deposit of loose alluvial sand. The thickness of this stratum is between 20 and 30 feet, depending on the location. The alluvial sand deposit is underlain by



clay, a deposit of glacial Lake Warren. The lower limit of the clay was not precisely determined but at borehole 1 it was inferred to extend at least to a depth of 100 feet below the borehole elevation, indicating a probable thickness of at least 60 feet. The clay is stiff for the first 20 feet of depth but becomes less stiff below this depth.

About 4 feet of stiff clay was found underlying the fill at borehole 2. This clay is believed to exist in its present location as a result of erosion from the adjacent hillside.

#### WATER LEVEL

The water level in the boreholes near the stream is from 2 to 5 feet higher than the elevation of the water in the stream. The long term stabilized water level in the impermeable clay on the hill was not determined as it is of little consequence to the construction of this structure.

#### FOUNDATIONS

The looseness of the alluvial sand precludes the possibility of founding the abutments and central pier on footings; therefore piles will be required. As a hard bearing stratum was not encountered to the full depth of the borings (75 feet) nor in the probe to depth 100 feet, the piles must be designed as friction piles. The piles must be driven through the sand and into the underlying clay. A depth of 18 feet into the clay should be sufficient for the total loads expected. As the clay becomes less stiff below a depth of 20 feet, it is recommended that the piles not be driven below this depth. Using steel piles to depth 18 feet into the clay an allowable



load of  $31\frac{1}{2}$  kips per pile may be used for the design of abutments and 26 kips per pile for the central pier. With timber or concrete piles to the same depth, an allowable load of 36 kips per pile may be used for the abutments and 30 kips per pile for the central pier.

One foot diameter piles have been assumed in all cases. For the abutments it was assumed that piles would be driven through the new approach fill. If this is not the case and excavations are made to the present river level the loads should be reduced to the values given for the central pier.

The piles should be driven in two rows with the actual positions of the piles staggered in each row. The piles should not be spaced closer than  $2\frac{1}{2}$  pile diameters apart (centre to centre spacing). The total allowable load of the entire foundation will then be the allowable load on a single pile multiplied by the number of piles.

As an additional aid to the development of the full capacity of each pile and to prevent overlapping of load on adjacent piles it is recommended that every other pile be driven at an inclination away from the abutment of about 1 in 5. Because of the depth of the piles care should be taken to ensure that the piles from the abutments do not interfere at depth with those from the central pier; the minimum spacing given above is also applicable here.

The calculations for these recommendations are given in the Appendix.



### SETTLEMENT OF THE FOUNDATION

The settlement of the foundations under their design pile loads will be approximately  $1\frac{1}{2}$  inches; this should not affect the performance of the structure. These calculations are also given in the Appendix.

### ABUTMENTS

The piles for the abutments may be driven and cut off at river level. No great difficulty should be encountered in excavating to this depth. Alternatively the piles may be driven through the approach fill for perched abutments.

No stability problem of the embankments should arise if the fill is placed directly on the existing surface. The clay near the surface at borehole 2 is stiff and contains many silt seams which will bring about relatively rapid drainage of the clay while the fill is being placed. The underlying sand will adjust immediately to the loading. The deep deposit of clay below, has been exposed to surcharge loads well in excess of the weight of the embankment fill before river erosion cut the valley to present levels. These same remarks apply when considering the stability of the fill in the adjacent depression to the east of this site.

Since there is sufficient space a spill through design for the abutments is recommended. This will greatly reduce the lateral pressure of the soil against abutment walls. With perched abutments, of course, there will be very little wall surface exposed to horizontal earth pressures.



Erosion protection of the river bed around the centre pier and of the slopes bounding the sides of the river will be required. Any rip rap must be placed on a bed of well graded pit run gravel otherwise the fine natural sand will be sucked through the voids in the rock by the river current.

#### SETTLEMENT OF EMBANKMENTS

The settlement of the embankments due to the addition of fill should occur during construction as most of the material is sand. The amount of settlement of the sand and underlying clay should not be great as a weight of soil at least equivalent to the fill has been eroded off the surface in the geologically recent development of the valley system.

Provided the fill is well compacted little differential settlement should occur between the road and the actual structure.

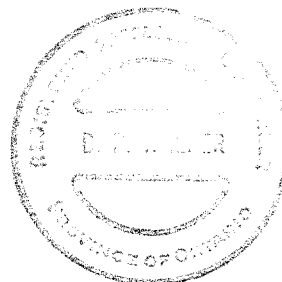
If you have any queries after you have reviewed the contents of this report we shall be glad to hear from you.

Yours very truly,

*Barry Walker*

BW:GC  
ENCLS.

B. Walker, P.Eng., PhD.





## APPENDIX

### PILE CAPACITY

The total load on a pile ( $Q_T$ ) driven through sand into underlying clay will be carried partly by a shear load along the shaft in the sand and clay ( $Q_1$  &  $Q_2$ ) and a base load ( $Q_3$ ).

In the following calculations, 1 foot diameter steel piles are assumed.

The frictional load due to the sand will be calculated for the case of the central pier; the load is given approximately by the following expression:

$$Q_1 = A_s \times F$$

where:

$A_s$  is the area of the exposed shaft; taking the depth of sand below river bed to be 18 feet,  $A_s = 18\pi$

$F$  is the friction force which is conservatively taken as  $0.2 \times P_c$  (the effective overburden pressure at the centre of the sand stratum, i.e., depth 9 ft.)  $F = 0.2 \times 500 = 100$  psf

$$\therefore Q_1 = 5.7 \text{ kips.}$$

For the case of the abutments, if the piles are driven from the surface  $Q_1$  may be taken as 20 kips for both locations. The load,  $Q_2$ , taken by the shaft in the clay is given by the following expression:





$$Q_2 = \alpha c_u A_s$$

$A_s$  per foot of depth =  $\pi d_1$ , where  $d_1$  is the depth of penetration in the clay

$c_u$  is the undrained shear strength of the clay (2000 psf within the first 20 feet depth of penetration); a lower value applies below 20 feet)

$\alpha$  is a reduction factor to give the adhesion between the pile and the clay (use 0.40)\*

Substituting

$$Q_2 = 2.5 d_1$$

where:  $Q_2$  is in kips and  $d_1$  (the depth into the clay) is in feet  
For a penetration of 18 feet  $Q_2 = 45$  kips

The load taken by the base ( $Q_3$ ) is given by the following expression:

$$Q_3 = A_b c_u \times N_c$$

where:  $A_b$  is the area of the base (  $\frac{\pi}{4}$  )  
 $N_c$  is a bearing capacity factor (9)

Substituting for  $c_u = 2$  ksf

$$Q_3 = 14 \text{ kips.}$$

\* See M.J. Tomlinson - 'Foundation Design and Construction' P. 373.

The allowable load on the pile (QA) is the summation of the 3 loads reduced by a factor of safety (F)

thus:

$$QA = \frac{1}{F} (Q_1 + Q_2 + Q_3)$$

and using  $F = 2.5$

$$QA = \frac{1}{2.5} (Q_1 + 2.5 d + 14)$$

where:  $Q_1$  = load taken by the sand, 20 kips for each abutment and 5.7 for the central pier.

For piles under the abutments,  $QA = 31\frac{1}{2}$  kips, while under the central pier  $QA = 26$  kips, for a depth of 18 feet into the clay.

For timber or concrete piles 1 foot diameter where  $\alpha = 0.5^*$  and the other factors the same.

$$QA = \frac{1}{2.5} (Q_1 + 3.1d + 14)$$

Again for depths 18 feet into the clay QA for the abutment piles = 36 kips while for the central pier  $QA = 30$  kips.

\* M.J. Tomlinson - Ibid

SETTLEMENT OF PILE FOUNDATION

The settlement of each abutment will depend on the depth of the piles, the actual load which the foundation is designed to carry the number of piles and the spacing. The following calculations will give an order of magnitude only.

Assume an area 5 feet x 40 feet and 26 steel piles, 18 feet into the clay, (i.e. average total depth about 50 feet); thus the total abutment on pier load  $= 31\frac{1}{2} \times 26 = 820$  kips.

For purposes of immediate settlement replace the pile foundation by a rigid footing (5x40) at a depth equal to  $\frac{2}{3}$  of the total depth and carrying the equivalent average stress (i.e., about 4 ksf) . The immediate settlement is given by the following expression:

$$S_i = \mu \mu_o \frac{qB}{E} 0.75 K$$

where:  $\mu, \mu_o$  are factors which depend on the dimensions of the footing and depth of soil ( 1)

q is the average stress ( 4 ksf)  
 B is the width of the footing (5 feet)  
 K is a rigidity factor (0.8)  
 E is the stress strain modulus of the soil (assume 200 ksf)

Substituting:

$$S_i = 3/4 \text{ inches.}$$



For purposes of consolidation settlement assume the load from each pile to be distributed to the soil at a depth of  $2/3$  the total depth.

The settlement due to consolidation is given by the following expression:

$$S_c = \lambda M_v \Delta h \Delta p K$$

where:

- $\lambda$  is a factor to account for overconsolidation (assume 0.8)
- $M_v$  is the coefficient of compressibility (assume .005 sq.ft./kip)
- $\Delta h$  is the thickness of compressible layer (assume 20 ft.)
- $\Delta p$  is the increase in stress at the midpoint of the compressible layer. An estimate of this may be made by considering each pile to be a point load and summing the increase in stress under the centre of the foundation due to each load; stress distribution according to Boussinesq was assumed. At a depth of 15 feet below the point loads  $\Delta p \sim 0.6$  ksf
- $K$  is a rigidity factor (0.8)

Substituting:

$$S_c = 0.5 \text{ inches}$$

The total settlement will therefore be in the order of  $1\frac{1}{4}$  inches.



View of Bridge Looking East



View Looking Upstream

SUPER IMPOSED DOCUMENT MAY  
APPEAR AS MULTI-FEED ON FILM.



View of Bridge Looking East



View Looking Upstream



View Looking Downstream From Bridge

SUPER IMPOSED DOCUMENT MAY  
APPEAR AS MULTI-FEED ON FILM.



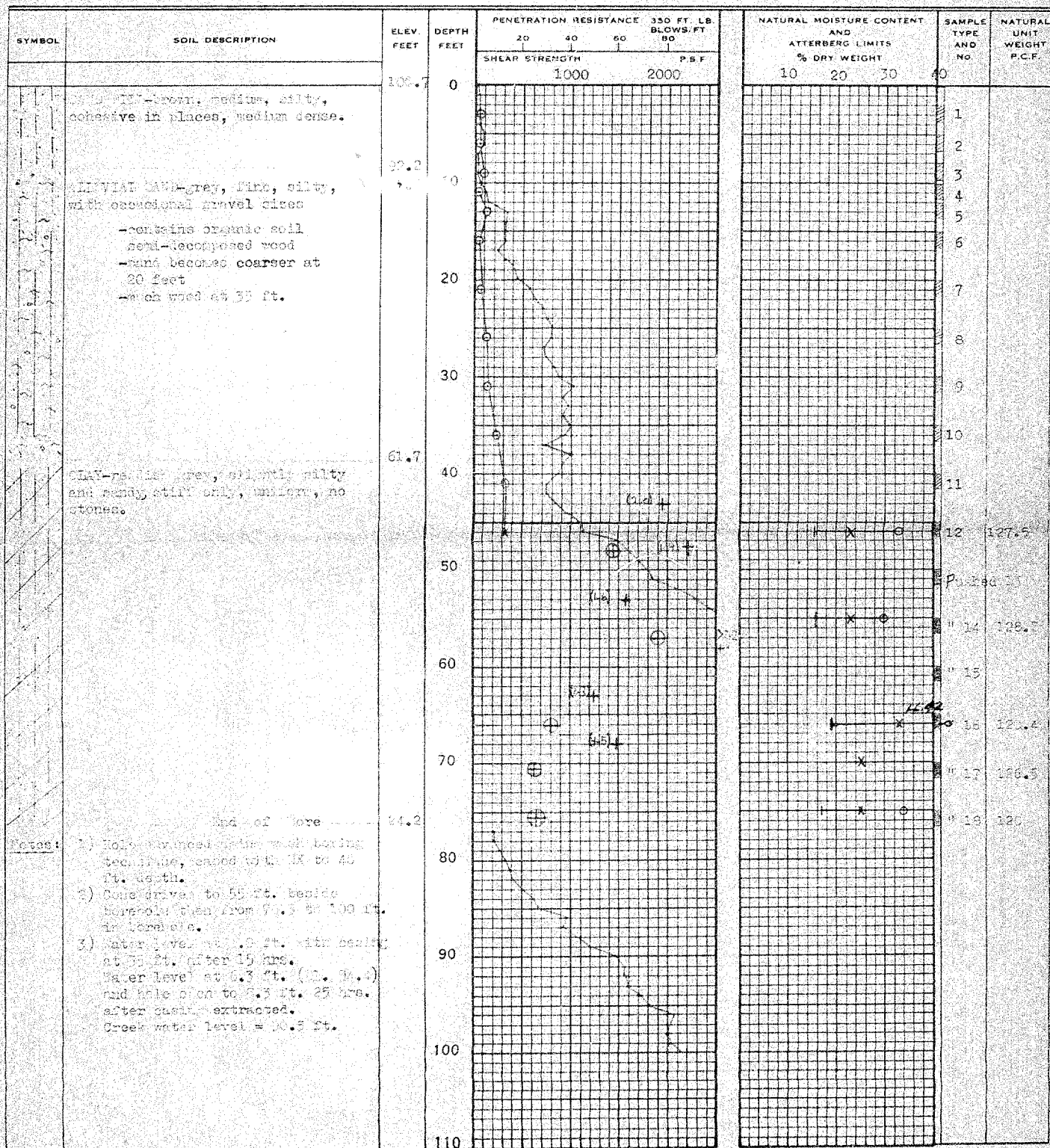
View Looking Downstream From Bridge



BOREHOLE No. 1.  
PROJECT Big Creek Bridge.  
LOCATION Hendon, Ft. Howard.  
HOLE LOCATION See Wgs. 1.  
HOLE ELEVATION 100.7 ft.  
DATUM See Wgs. 1.

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)

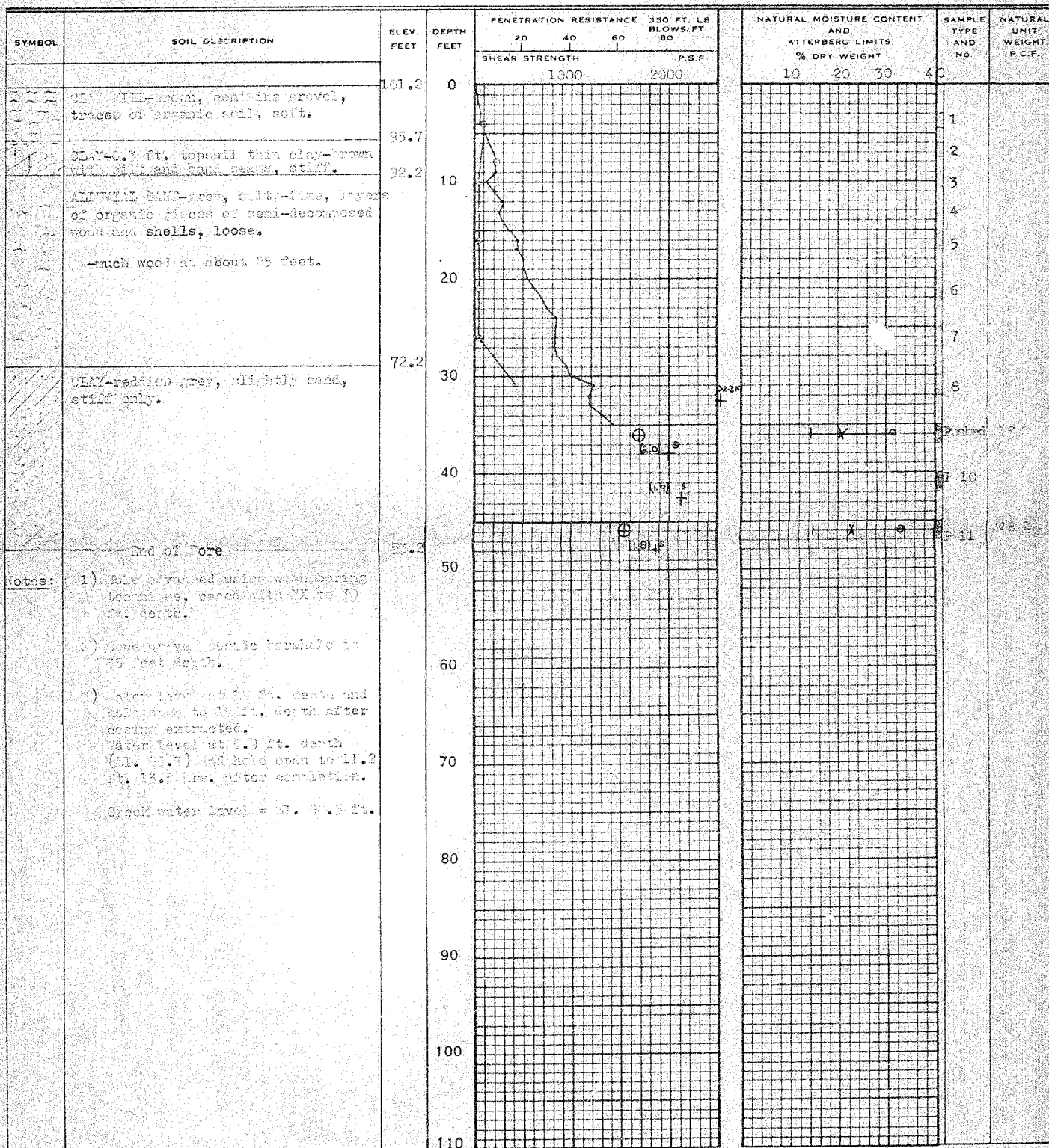
NATURAL MOISTURE CONTENT  
AND LIQUIDITY INDEX  
ATTERBERG LIMITS  
LIQUID LIMIT  
PLASTIC LIMIT  
SAMPLE TYPE  
2" O.D. SPLIT TUBE  
2" I.D. SHELBY TUBE  
3" O.D. SHELBY TUBE



BOREHOLE No. 2.  
PROJECT Big Creek Bridge.  
LOCATION Near Ft. Rowan, Ont.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.2 Ft.  
DATUM See Dwg. 1.

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 X  
ATTERBERG LIMITS  
LIQUID LIMIT —○—  
PLASTIC LIMIT —  
SAMPLE TYPE  
2" O.D. SPLIT TUBE ⊕  
2" I.D. SHELBY TUBE ⊕  
3" O.D. SHELBY TUBE ⊕






## SITE INVESTIGATIONS • SOIL MECHANICS CONSULTATION


PROJECT No. J1775.


## LEGEND

BOREHOLE NO. 3.  
PROJECT Big Creek Bridge.  
LOCATION Near Pt. Rowan, Ont.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 108.4 ft.  
DATUM See Dwg. 1.

## 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

X

## ATTERBERG LIMITS

LIQUID LIMIT \_\_\_\_\_  
PLASTIC LIMIT \_\_\_\_\_

SAMPLE TYPE

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.	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS				SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.					
				20	40		60	80	% DRY WEIGHT								
				SHEAR STRENGTH				P.S.F.									
	Topsoil to 1.0 ft.	108.4	0	1000				2000				10	20	30	40		
	SILO FILL-brown, cohesive, with much sand and gravel, dense.	105.4										Pushed	129.0				
	CLAY-brown, organic, stiff, with silty and sandy layers.	102.9															
	ALLOVIAL SILT-grey, clayey, organic semi-decomposed wood.	99.4	10														
	CLAY-reddish grey, silty, slightly sandy -becoming less silty and slightly sandy with depth.		20														
			30														
			40														
	End of Bore	81.9															
Notes:	1) Hole advanced using wash boring technique, cased with NX to 12 feet depth. 2) Cone driven beside borehole to 16 feet. depth.																

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 5.  
PROJECT No. J1775.

## LEGEND

### PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—  
2" I.D. SHELBY TUBE —x—x—x—x—  
2" DIA. CONE —————

### SHEAR STRENGTH

UNDRAINED TRIAXIAL  
AT OVERBURDEN PRESSURE ⊕  
UNCONFINED COMPRESSION ⊗  
VANE TEST AND SENSITIVITY (S) +<sup>s</sup>

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. 4.  
PROJECT Big Creek Bridge.  
LOCATION Near Pt. Rowan, Ont.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 113.1 ft.  
DATUM See Dwg. 1.

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT.		NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
				20	40			
	Sand & Gravel Fill-to 0.4 ft.	113.1	0	SHEAR STRENGTH P.S.F.				
	CLAY-brown, silty, slightly sandy, very stiff, some small gravel sizes. -change to greyish brown at about 13 ft.		10					
	End of Bore	96.6	20					
Notes:	1) Hole advanced using wash boring technique, cased with NX to 3 ft.		30					
			40					

SHEAR STRESS ksf

2

1

Borehole 1  
Depth 38 ft.  
 $\gamma = 118.2$  pcf.

Borehole 1  
Depth 45 feet  
 $\gamma = 127.5$  pcf.

% STRAIN

UNDRAINED TRIAXIAL TESTS ON STIFF CLAY

SHEAR STRESS  $\text{knf}$ 

1.0

.5

Borehole 1  
Depth 66 ft.  
 $\gamma = 120.4 \text{pcf}$

Borehole 1  
Depth 76 ft.  
 $\gamma = 120 \text{pcf}$

Borehole 1  
Depth 70 ft.  
 $\gamma = 126.5 \text{pcf}$

% STRAIN

UNDRAINED TRIAXIAL TESTS ON LESS STIFF CLAY

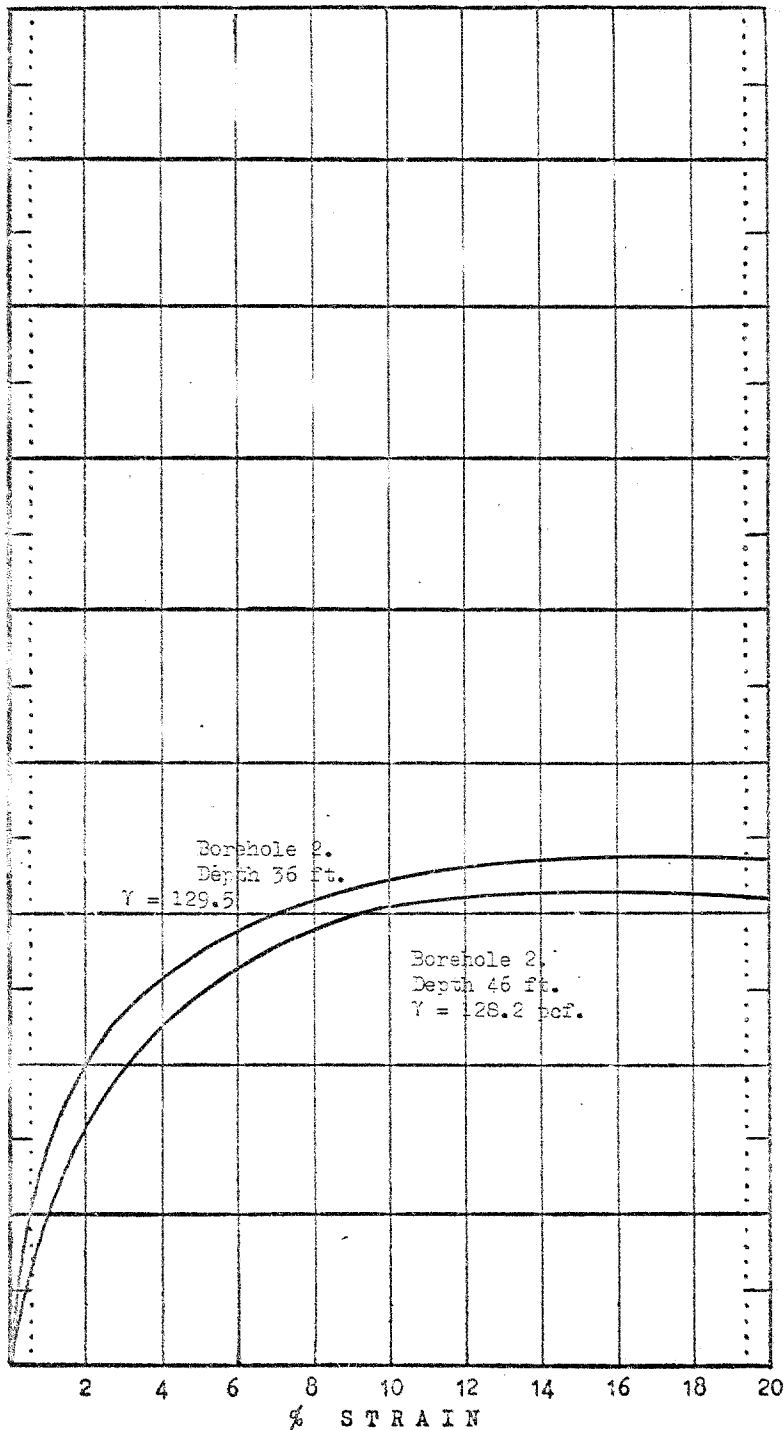
BOREHOLE 1

WILLIAM A. TROW AND ASSOCIATES

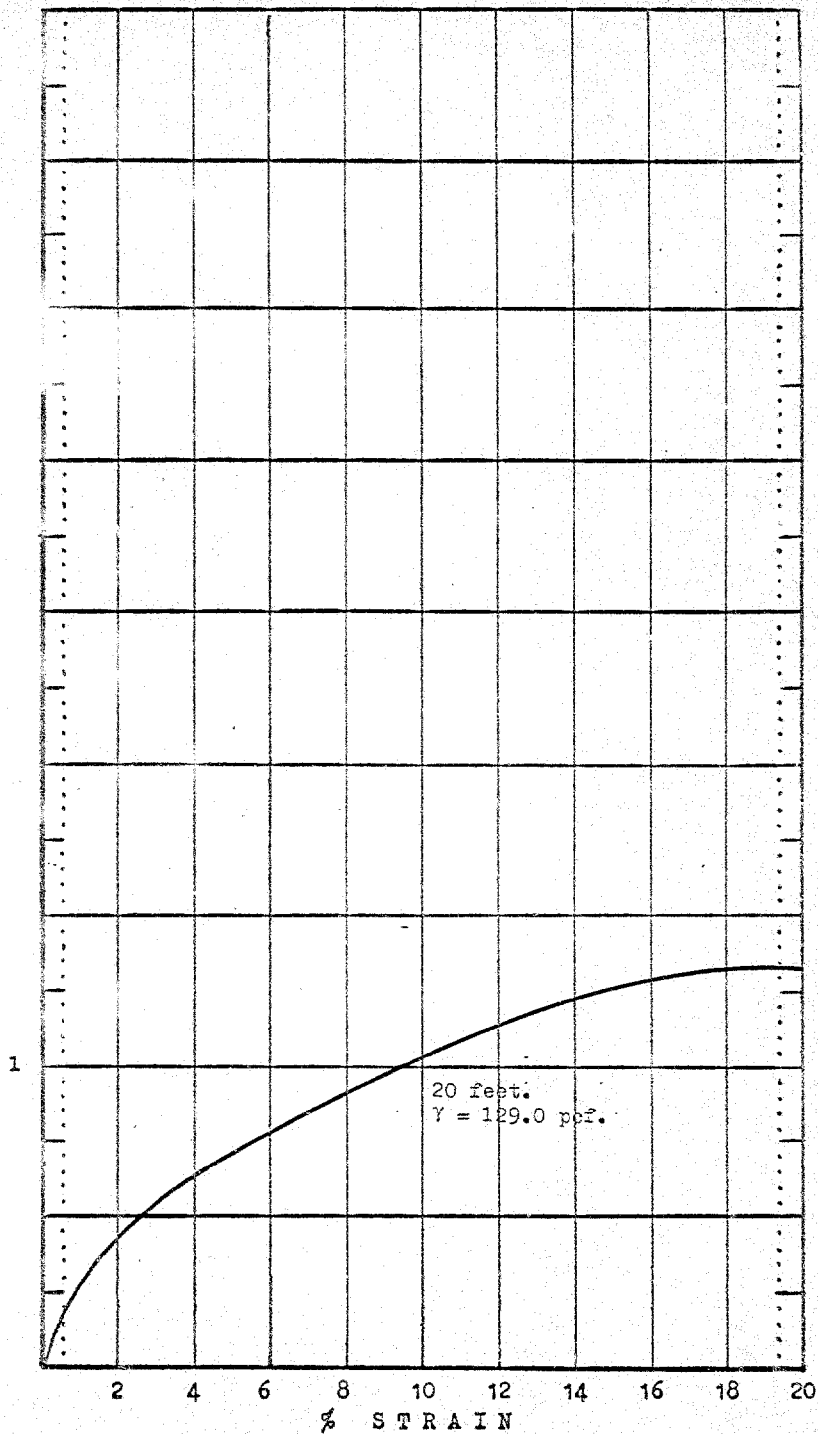
SHEAR STRESS  $\text{ksf}$ 

2

1



UNDRAINED TRIAXIAL TESTS ON STIFF CLAY (B.H. 2)

SHEAR STRESS  $kcf$ 

UNDRAINED TRIAXIAL TEST ON STIFF CLAY (B.H. 3)

WILLIAM A. TROW AND ASSOCIATES

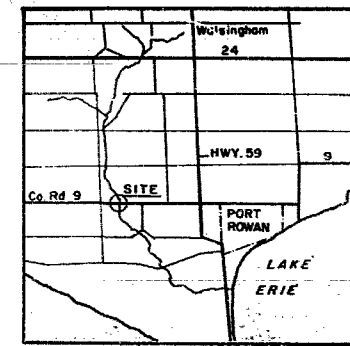
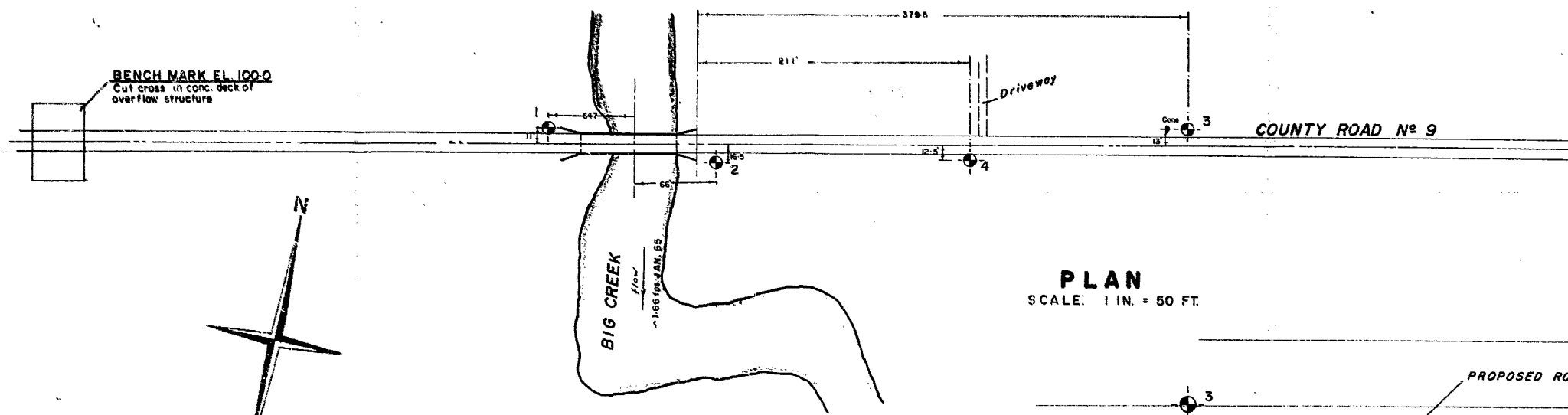


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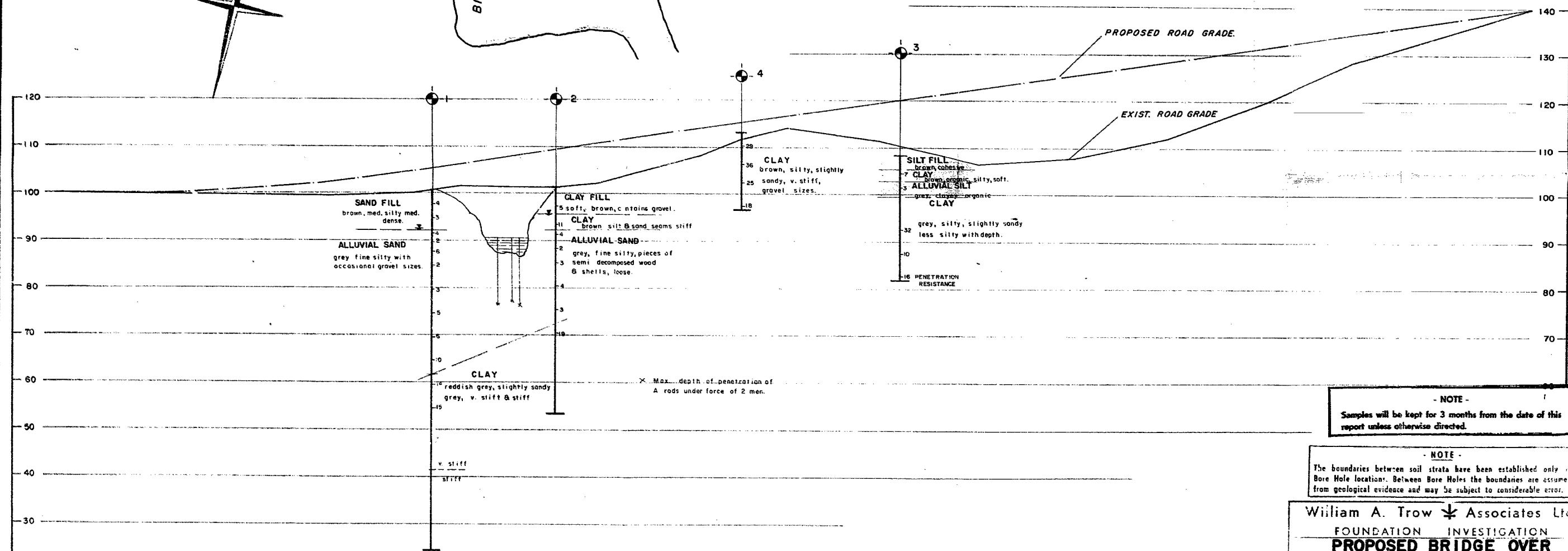
NORFOLK CTY RD #9

BIG CREEK

BRIDGE



**PLAN**  
SCALE: 1 IN. = 50 FT.



**INTERPRETED SUBSOIL STRATIGRAPHY**  
SCALE: VERT. 1 IN. = 10 FT.  
HOR. 1 IN. = 50 FT.

**- NOTE -**  
Samples will be kept for 3 months from the date of this report unless otherwise directed.

**- NOTE -**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

**William A. Trow & Associates Ltd.**  
FOUNDATION INVESTIGATION  
**PROPOSED BRIDGE OVER BIG CREEK**  
NORFOLK COUNTY ROAD No. 9  
TWP. SOUTH WALSHAM ONTARIO  
PROJ. 1775 | DATE JAN. 1965 | DWG. No. 1