

62-F-248M

BOSTON CREEK

# 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: J819

February 26, 1962

McDowell and Jewitt,  
Consulting Engineers,  
92 Kent Street South,  
Simcoe, Ontario

62-F-248M

Attention: Mr. McDowell

Re: Foundation Conditions - Proposed Bridge Replacement, Boston Creek  
Lots 6 and 7, Conc. III, Tuscarora Twp.

Dear Sirs:

We have completed an investigation of the subsoil conditions existing at the site of this township road crossing of Boston Creek.

The foundation soil consists of firm to stiff, highly plastic clay which extends to a depth of approximately 13 feet below the creek bed, or about 25 feet below the deck of the existing bridge. This material becomes siltier and contains sand and some gravel in the lower 3 to 5 feet. At the 13 foot depth below the creek bed, a deposit of loose to medium dense sandy silt till is encountered and about 3 to 6 feet lower down is bedrock which consists of dolomite with seams and layers of gypsum.

The bridge structure may be founded in the clay at a depth of about 4 feet below the creek bed or, alternatively, it can be supported on end-bearing piles driven to bedrock. With the former proposal, the safe net bearing pressure is equal to 2300 psf for a rectangular abutment footing. The settlement resulting from this pressure application should be less than 1 inch. The safe load for a timber pile or for any other end-bearing pile unit will be equal to its permissible capacity when considered as a short column. If timber piles are used, driving should cease as soon as bedrock is reached, in order to avoid damage to the tips. The increase in resistance will be quite marked at the bedrock contact.

The factual information and soil mechanics reasoning, which form the bases for these recommendations are considered in the following sections.

## Site Description

The existing multi-arch concrete bridge structure lies at the south side of the flood plain of Boston Creek. The general surface terrain to the north and south of this crossing lies about 17 feet higher than the flood plain

and it is relatively flat. Numerous elm and other hardwoods grow in the land adjacent to this creek crossing as indicated on the accompanying photographs.

The existing concrete structure was built at some time immediately following the first World War. Numerous cracks are evident along its length. These cracks are believed to be the result of differential settlement of the supports.

The north approach to the bridge consists of an embankment which rises about 7 feet above the flood plain level. It is understood that the low areas to the north of the bridge are covered with about 2 to 3 feet of water during spring flooding and the maximum flood level is reported to be just above the road surface.

The creek surface was covered with snow and ice at the time of the investigation and probings through it encountered refusal in more ice about 2 to 2½ feet below the snow cover. The results of holes 2 and 4 indicate that the creek bed lies about 5 to 5½ feet below the present surface.

#### Geology

This section of Tuscarora township lies near the western end of the Haldimand Clay Plain which was formed by the sediments of glacial Lake Warren during the intermediate stages of ice retreat from southern Ontario. These clay sediments generally cover the till deposits of the Wisconsin glaciation. In this and adjacent townships occasional scattered drumlins of glacial material outcrop through the lake bed deposits.

The underlying bedrock in this part of the Niagara peninsula consists generally of dolomites and gypsum of the Upper Silurian System which were deposited about 350 million years ago. This formation overlies the Lockport dolomites which cap the Niagara escarpment and maintain it as a distinct land form.

#### Field Investigation

The borings of this investigation were performed using wet sampling methods. The holes were cased for the first few feet of depth, but thereafter they were advanced by washing ahead with a large bit. This procedure was followed in order to expedite the field investigation program.

Samples were taken at 5 foot intervals of depth or less, and since the predominant soil consisted of a stiff plastic clay, many undisturbed Shelby tube specimens were recovered. Field vane tests were performed in the clay immediately after the samples were recovered.

Hole 1 was taken to refusal and assumed bedrock. In hole 2 bedrock was proven for a depth of 8½ feet.

It was originally intended only to make two borings at this site. However, since the upper levels of the soil in the flood plain adjacent to the north abutment appeared to be less competent than the clay encountered in hole 1 to the south, it was decided to make two additional shallow borings in order to obtain more information concerning the lateral extent of this weaker soil. Hole 3 was made several feet to the north of the bridge to determine the capacity of the soil in the flood plain to support embankment loads.

The locations of all borings are shown on Dwg. 1 of this report. The elevations of the surface at each boring were referred to the bench mark reference shown on this drawing.

### Subsoil

The logs of the borings are presented as Dwgs. 2 to 5 of this report. The soils information from them has been summarized in the estimated stratigraphical profile shown on Dwg. 1.

It is seen that the upper levels of soil, down to a maximum depth of about 13 feet below the creek bed, consists of firm to stiff clay of high plasticity. This clay is somewhat stiffer in the south bank of the flood plain than it is in the vicinity of the existing north abutment. According to laboratory compression tests and field vane measurements, the undrained shear strength of the clay below the creek bed ranges from 850 to 2000 psf. The approximate strengths under the north and south abutments are 1000 and 1900 psf respectively. The liquid limit of the clay ranges in the order of 56 to 60 percent, and the moisture content lies in the lower quarter to half of the plastic range, depending upon the strength condition.

Below depths of 8 to 10 feet from the creek bed, the clay becomes siltier and it contains some gravel sizes. This is a transition condition to the underlying loose to medium dense sandy silt till which begins about 13 to 14 feet below the creek bed.

Bedrock was encountered at depths of 17 to 19 feet below the creek bed or 29 to 31 feet below the existing bridge deck. It consists of hard dolomite with numerous intrusions and thick layers of gypsum.

### Foundations

As indicated in the opening paragraphs, two methods are available for the support of this structure. The more positive scheme is to carry the weight of the bridge on piles end-bearing on rock. Although the clay will offer some resistance, there should be no difficulty experienced in reaching bedrock. The level of the rock will be marked by a very sharp increase in the penetration resistance offered to the pile driver. The safe loading of a pile will be equal to its permissible capacity when considered as a short column.

If timber piles are used, driving should be terminated as soon as this sharp increase in resistance is experienced; it should occur at the levels indicated on the borehole logs. If this precaution is not taken, the ends of the piles may be broomed.

Although the strength of the clay is not high, it still has sufficient capacity to support footing loads in a reasonably economic manner. Footings must, of course, be taken about 4 feet below the creek bed level in order to provide protection against scouring. The shear strength,  $c$ , of the clay at this required depth ranges from an average of about 1000 psf under the north abutment to 1900 psf at the south end of the bridge.

The safe net bearing pressure,  $q$ , that may be applied is determined from the expression:

$$q = \frac{CN}{F}$$

where:  $N$  is a bearing capacity factor equal approximately to 7 for a rectangular footing founded at this depth

and  $F$  is the factor of safety required to keep settlements within tolerable limits. For the footing size and soil strength applicable in this problem the use of a value of  $F = 3$  should suffice to keep settlement movements less than 1 inch.

Substituting, in the above expression, the safe bearing value for the north side of the bridge is computed to be 2300 psf.

This pressure represents the safe net increase in pressure that may be applied in excess of the surcharge weight of soil below river bed level. All dead load, including footing weight, less soil displaced by it, and all live load occurring at least once per year should be considered when utilizing this pressure for design purposes.

Since the clay is relatively impermeable, no difficulties should be experienced with ground water, provided that measures are taken to divert the creek away from the footing excavations. The clay in the bed of the footing may appear soft, however, particularly if workmen are permitted to walk over the bearing surface. For this reason the footing bed should be covered with a thin skim coat of weak concrete as soon as it is exposed.

It is possible that the excavation at the north tank may have to be taken deeper at local points in order to extend below weak clay. An approximate indication, when clay of satisfactory bearing strength has been reached, can be obtained by utilizing a man's weight on a bearing surface of given area.

The ultimate capacity of the clay for surface loading can be determined from the above noted formula, setting  $F = 1$  and  $N = 6.2$ . Square blocks of different sizes can be placed on the clay surface and the ultimate capacity can be determined by applying one's full weight to the blocks. A shear strength, at least equal to 1000 psf, or 7 psi, is required in order to obtain a safe bearing pressure of 2300 psf. The equivalent failure load on the blocks is equal to

$$\begin{aligned} W &= CNA \\ &= 43.4 A \end{aligned}$$

where:  $W$  is the weight of a man in lbs. and  $A$  is the bearing area of the square block in square inches.

The soil adjacent to the existing embankments is quite strong enough to support the weight of approach embankments taken several feet higher than present fill levels. Therefore, since no significant increase in embankment height is envisaged, there is no stability problem in this regard. If the width of the embankments is increased, however, surface layers of topsoil and other soft material must be cleared away from the loading areas.

The clay at this site will be quite resistant to the erosion forces of the creek. If the footings are poured neat against the excavation walls, no particular protection should be required. Erosion should occur only if the excavation has been made too big and the overcut is backfilled with chunks of clay or fine grained materials.

With regard to horizontal earth pressures against the abutments, very little immediate force will be exerted by the natural clay, since it has sufficient strength safely to support its own weight on vertical slopes for the depths of excavation required in this project. Where granular embankment fill presses against the abutments, the unit horizontal earth pressure,  $P$ , at any backfill depth,  $h$ , is given by the expression:

$$\begin{aligned} P &= k\gamma h \\ &= 1/3 \times 130h \\ &\approx 43 h \end{aligned}$$

This same expression can be used for the approximate indication of the very long term pressure exerted by the clay. The value indicated for  $k$  probably is somewhat too low and consequently the value for  $P$  also, theoretically, will be low. However, this error will be more than offset by the residual effective cohesion resistance developed in this overconsolidated material.

Some of the resistance to these horizontal forces will be provided by the bearing capacity of the clay under the abutment and wing wall footings and

by the adhesion force developed between the clay and the base of the footing. This latter value probably will be equal to the undrained shear strength of the clay for short term loading. For very long term sliding conditions, the clay can be considered as granular and the resistance along this surface will be equal to the normal weight on the footing times the tangent of the effective angle of internal friction,  $\phi'$  of the clay. The estimated value of  $\phi'$  for this clay is about 24 degrees and therefore  $\tan \phi'$  is equal to 0.44.

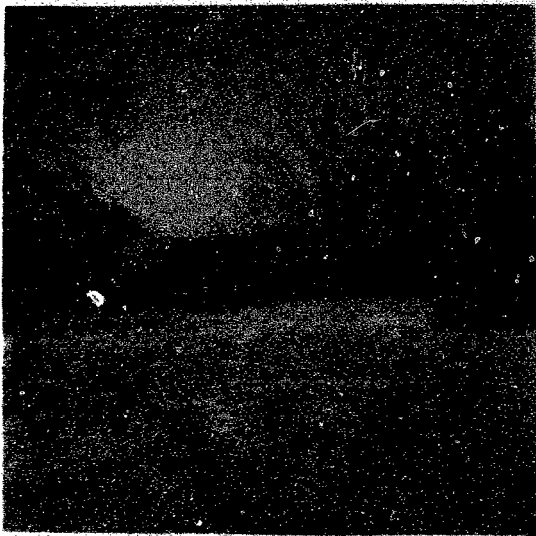
We shall be pleased to discuss any foundation problem that may concern you after you have reviewed the contents of this report.

Yours very truly,

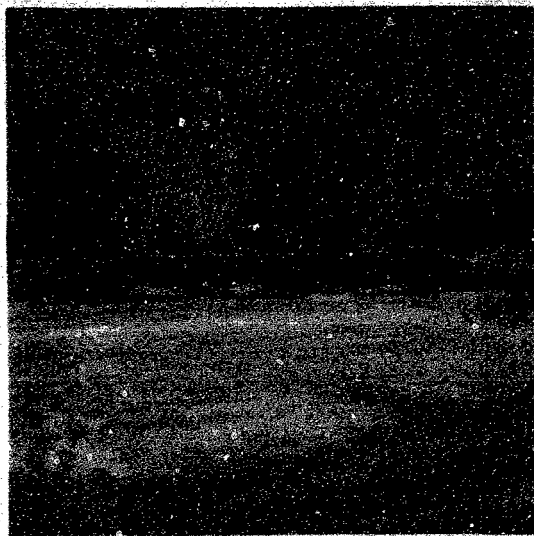
WAT/gc  
Encls.



*W. Trow*  
William A. Trow, P.Eng.



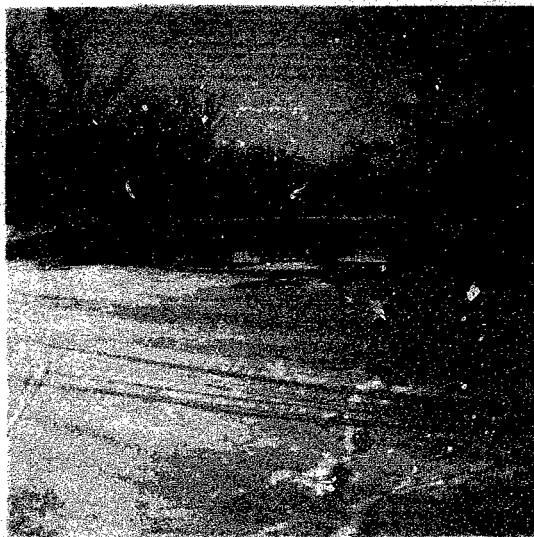
Boston Creek From The East



View Of Bridge From The East



View of Site From The North

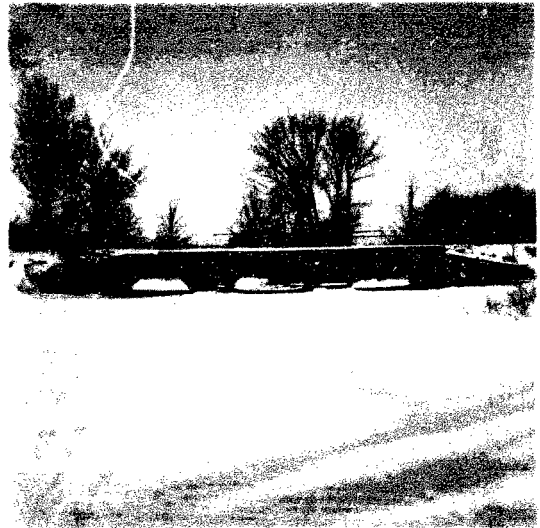


View of Bridge From The West





Boston Creek From The East



View Of Bridge From The East



View of Site From The North



View of Bridge From The West



View of Site From The South  
Drill on BH 2



View of Site From The S.E. Drill on BH 1  
Flood Plain In Background



Boston Creek From The N.W.  
Flood Plain In Foreground  
Drill On B.H. 1



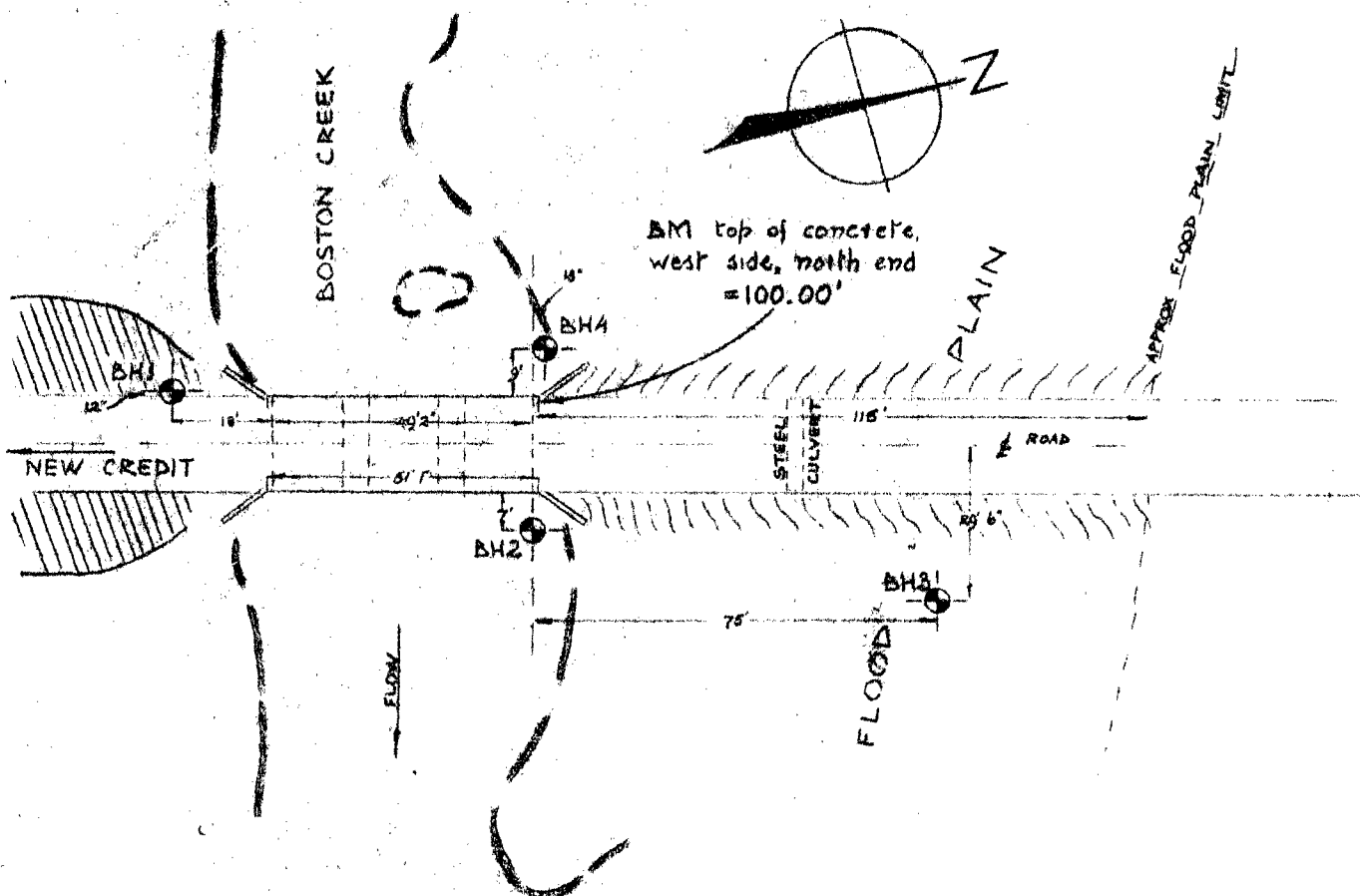
View of Site From The South  
Drill on BH 2



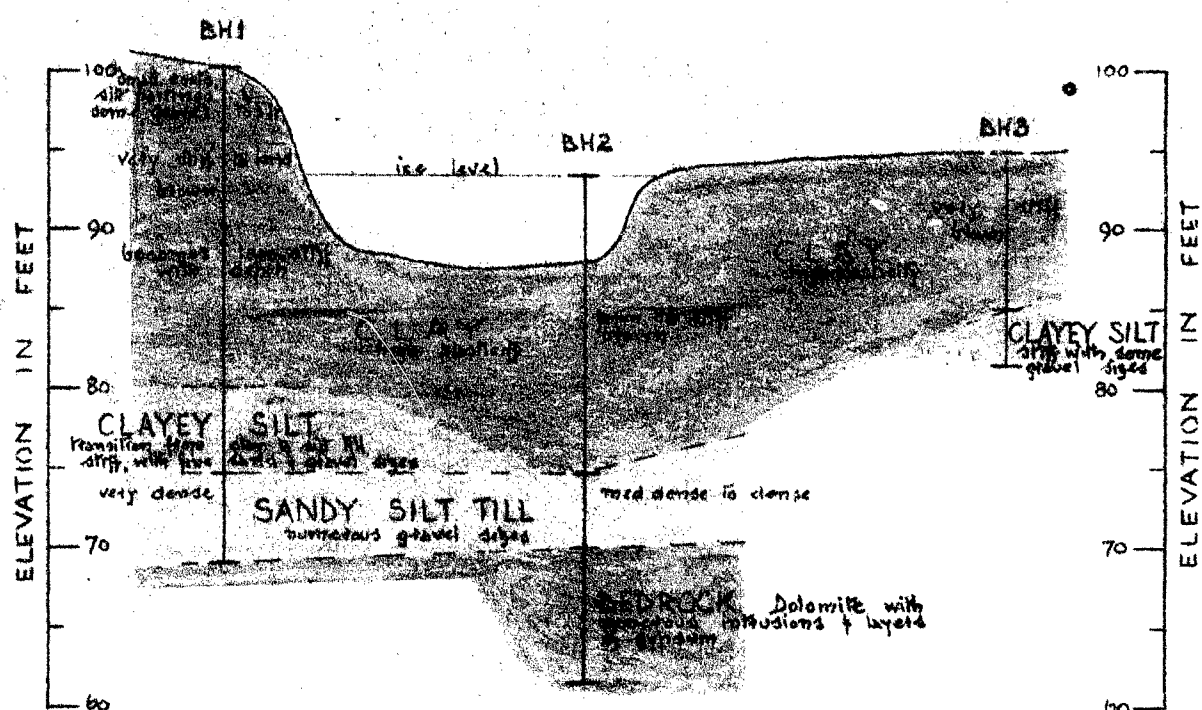
View of Site From The S.E. Drill on BH 1  
Flood Plain In Background



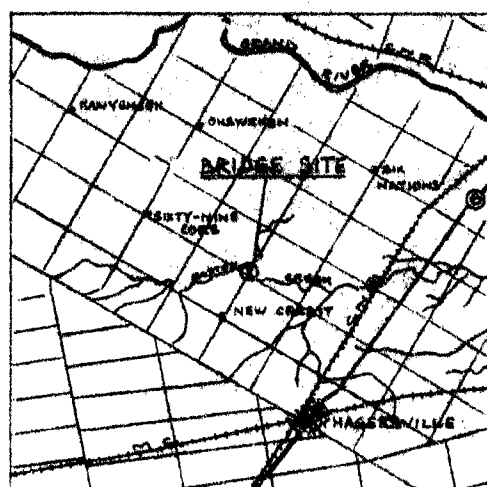
Boston Creek From The N.W.  
Flood Plain In Foreground  
Drill On B.H. 1



BOREHOLE LOCATION PLAN  
SCALE 1" = 30'



ESTIMATED SUBSOIL STRATIGRAPHY  
BOREHOLES 1, 2 & 3  
SCALES: HOR 1" = 30', VERT 1" = 10'



SITE PLAN  
SCALE 1" = 4 MILES

BOSTON CREEK BRIDGE  
SIX NATIONS INDIAN RESERVE  
LOT 6 & 7, CONCESSION III  
TOWNSHIP OF TUSCARORA  
ROAD #263, COUNTY OF BRANT  
ONTARIO

W. A. TROW & ASSOCIATES LTD  
PROJECT NO 819  
DRAWING NO 1

# WILLIAM A. TROW & ASSOCIATES LTD.




SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

## LEGEND




DRAWING NO. 2  
PROJECT NO. J819

BOREHOLE NO. 1  
PROJECT Boston Creek Bridge  
LOCATION Lots 6 and 7, Conc. III, Tuscarora Twp.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 101.1 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) 

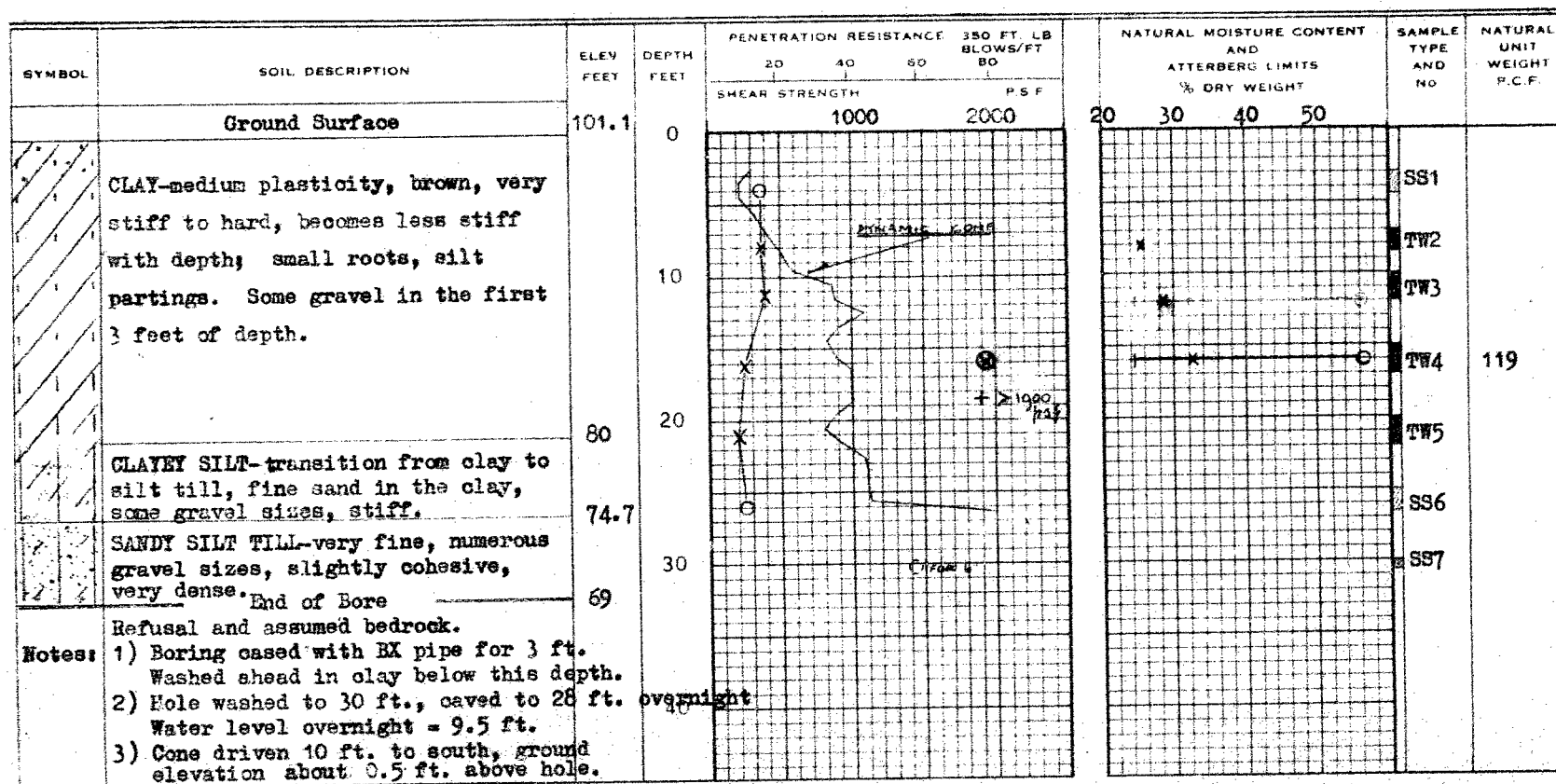
### NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

#### 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

## LEGEND

DRAWING NO. 3  
PROJECT NO. J819

BOREHOLE NO. 2  
PROJECT Boston Creek Bridge  
LOCATION Lots 6 and 7, Cono. III, Tuscarora Twp.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 93.4 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) +<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 —■—

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	60			
	Ice Surface	93.4	0	1000		2000	30 40 50		
	Ice and water.								
		88							
	CLAY-medium plasticity, soft, dark grey and organic stained to El 87 ft. then firm to stiff, brown below this level.		10					TW1	115 Levered
								TW2	Lost
								TW3	Levered
								TW4	Bounce on stons.
		74.7	20						
	SANDY SILT FILL-med. dense, to dense grey, numerous subangular med. gravel	70.0							
			30						
	BEDROCK-dolomite with numerous intrusions and layers of gypsum up to 14 ins. thick.								
	End of Bore	61.4							
Notes: 1) Hole cased for first 3 ft.									
			40						


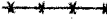
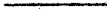




# WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 4  
PROJECT NO. J819

## 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

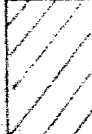

### ATTERBERG LIMITS

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 Boston Creek Bridge  
LOCATION Lots 6 and 7, Conc. III, Tuscarara Twp.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 94.9 ft.  
DATUM See Dwg. 1.

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	30	40	50		
		94.9	0	1000			2000				
	CLAY-med. plasticity, very stiff, brown.			X			X			TW1	
										TW2	
	CLAYEY SILT-sandy, some gravel sizes up to 2 ins., stiff. End of Bore	85	10	X						TW3	
		81.4								SS4	
Notes: 1) Hole cased to 5½ ft. 2) Soil too stiff for vane tests.											
			20								
			30								
			40								

# WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

## LEGEND

DRAWING No. 5  
PROJECT No. 3819

BOREHOLE No. 4  
PROJECT Boston Creek Bridge  
LOCATION Lots 5 and 7, Conc. III, Tuscarora Twp.  
HOLE LOCATION See Dwg. 1.  
HOLE ELEVATION 93.4 ft.  
DATUM See Dwg. 1.

### 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) +

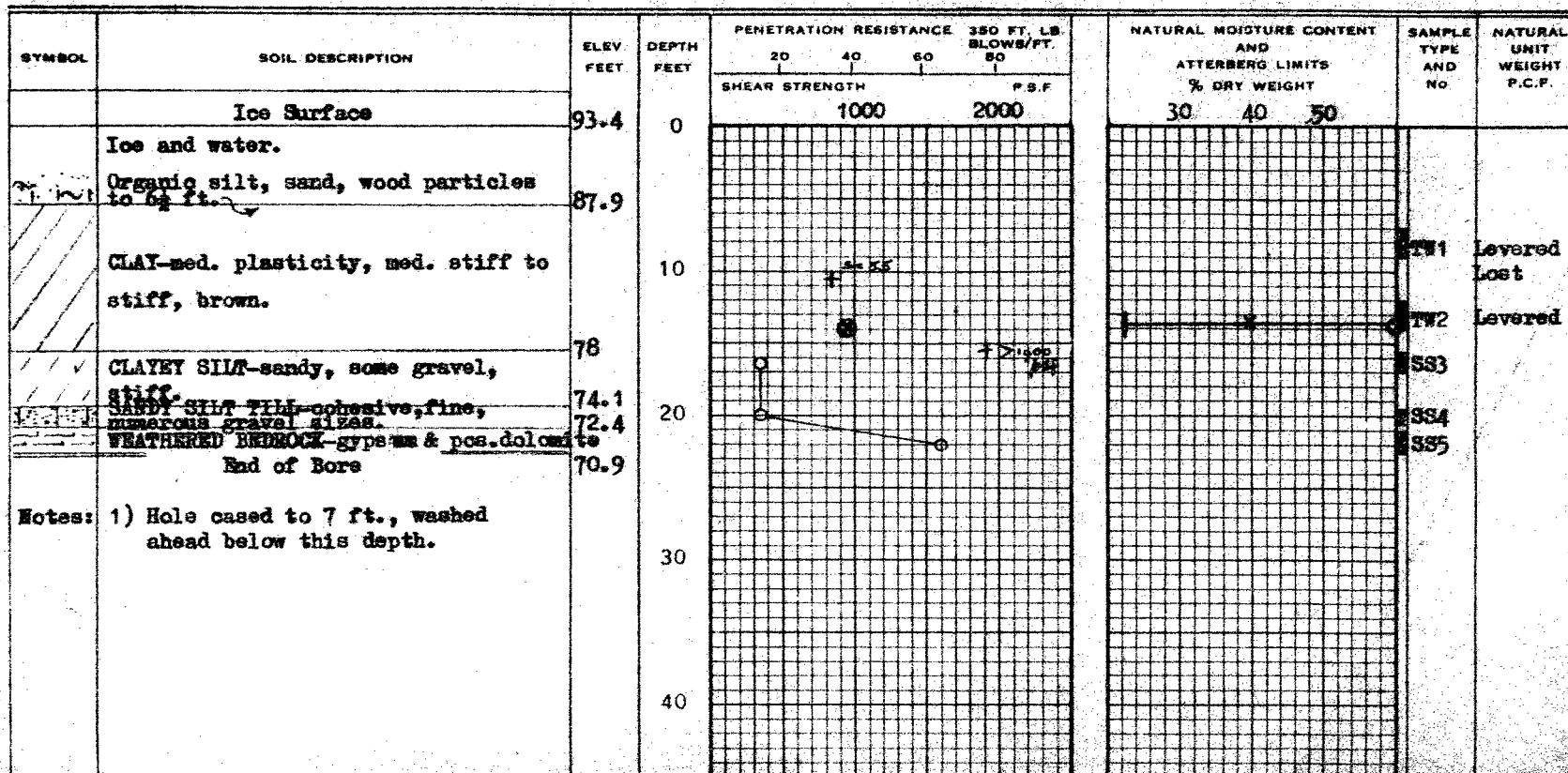
### NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

#### ATTERBERG LIMITS

LIQUID LIMIT —○—  
PLASTIC LIMIT ———

#### SAMPLE TYPE

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

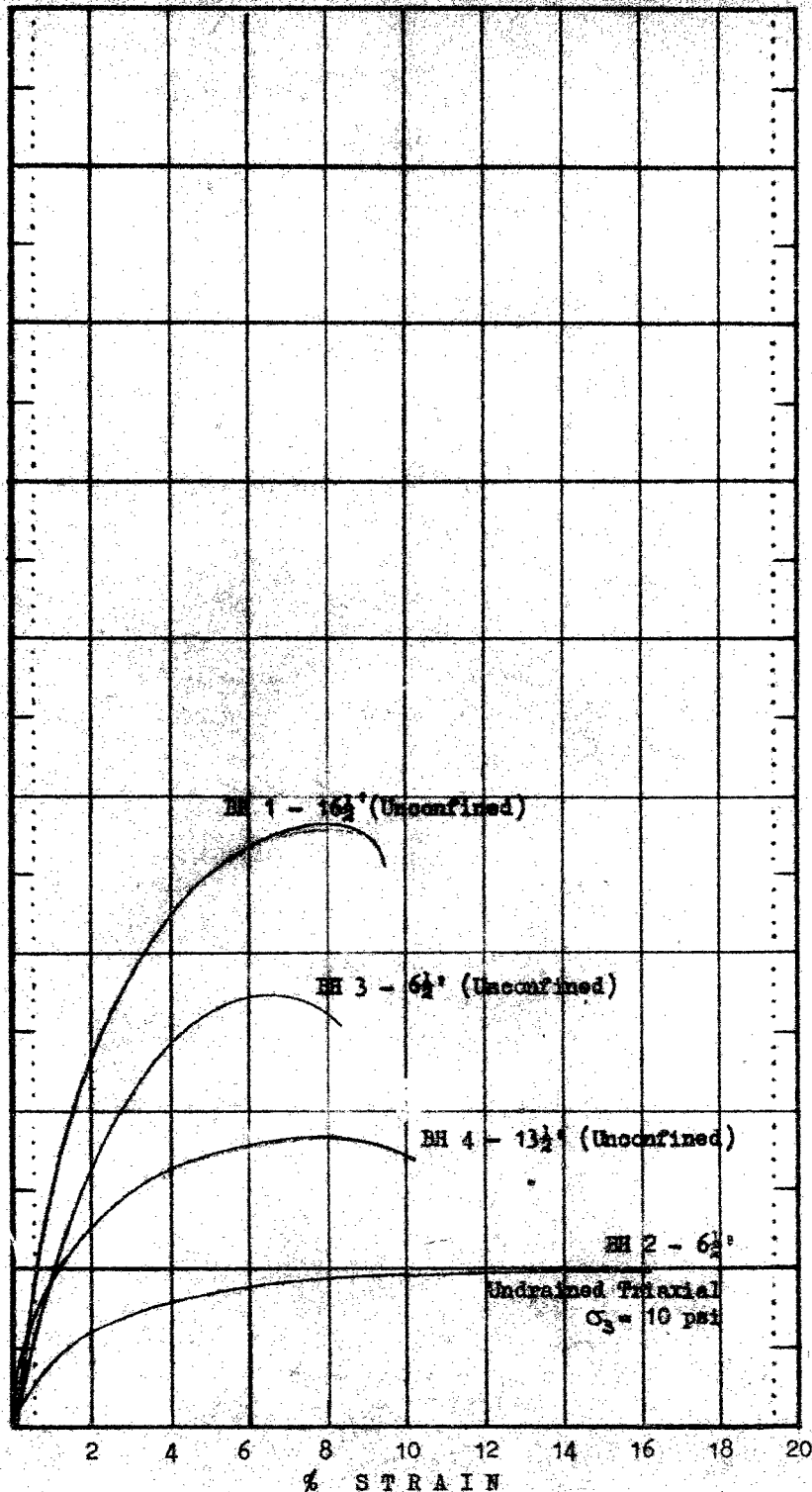




SHEAR STRESS  $\text{ksf}$ 

2

1



STRESS-STRAIN CURVES - CYLINDRICAL COMPRESSION TESTS ON CLAY

FROM BOBSON CREEK CROSSING

WILLIAM A. TROW AND ASSOCIATES