Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

March 16, 1960.

FOUNDATION INVESTIGATION - by William A. Trow & Associates.

Attention: Mr. S. McCombie.

Re: Proposed Crossing of County Road and
Hwy. 401 - Near Ridgetown - District 1
W.P. 86-59. Harwich Twp #1

The detailed foundation investigation for the above site, prepared by W. A. Trow & Associates, has been reviewed by this Section. We are in agreement with the conclusions and suggestions given in this report.

If any queries arise concerning the foundations for the proposed structure, please contact the Foundation Section.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGR.
per:

K Peaker

(K. Peaker,
FIELD FOUNDATION SUPERVISING ENGR.)

KP/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Ramsay
A. Gater
G. U. Howell
J. Roy
A. Watt

Foundations Office
Gen. Files.

cc: Foundations Office

Mr. A. M. Toye, May 3, 1960

Bridge Engineer. REVIEW OF SUBSOIL CONDITIONS

Materials & Research Section.

Attention: Mr. Bruce Davis.

Re: W.P. 81-59: Raleigh Twp. Bridge No. 13
W.P. 86-59: County Road Crossing near Ridgetown.
W.P. 56-59: County Road Crossing
-- Differential Settlements --

As requested, we have reviewed the subsoil conditions at the above noted structure sites with respect to differential settlements between the piers and the abu... ts. Our findings are as follows:-

1. Raleigh Twp. Bridge No. 13, Hwy. 401 - W.P. 81-59:

At this site, according to the subsoil data reported by Dominion Soil Investigation, Ltd., a layer of medium compressible silty clay, approximately 18 ft. in thickness, was encountered at a depth of approx. 32 ft. below the existing ground surface. The consolidation characteristics of this clay layer have not been investigated by the soil consultants but, according to our experience in the area, the coefficient of volume compressibility of 0.007 $ft.^2$/ton has been found to be a reasonable value for computations of consolidation settlements in the clay. Results of settlement calculations are as follows:-

Settlement of abutment due to 3 t.s.f. footing pressure + fill = 1.7".

Settlement of pier adjacent to abutment due to 3 t.s.f. footing pressure + fill = 1".

Settlement of centre pier due to 3 t.s.f. footing pressure = 0.5".

We would like to point out that the above computed settlements are on a long-term basis and we conclude that for design purposes, the maximum differential settlement will be of the order of 1/2" within the lifetime of the structure.

cont'd. /2 ...

2. <u>County Road Crossing near Ridgetown</u> - <u>W.P. 86-59</u>:

The foundation investigation at this structure site was carried out by William A. Trow & Associates. The site, in general, is underlain by a deep deposit of over-consolidated silty clay followed by bedrock. If a single-span design is used, in view of the fact that the subsoil conditions are relatively uniform, little differential settlement of any consequence, need be anticipated.

If it is proposed to use a design incorporating a centre pier, some long-term differential settlement between the abutments and the pier can be expected. An ultimate movement of the order of 2 inches has been estimated by the Consultants. It appears that for an over-consolidated clay, this magnitude of differential settlement has been over-estimated. In view of the slow rate of consolidation, as expected of clays, we are of the opinion that for practical purposes, a differential settlement of the order of 1 inch can be used for design within the lifetime of the structure.

3. <u>County Road Crossing</u> - <u>W.P. 56-59</u>:

The fo[illegible]dation investigation at this structure site was carried out by E. M. Peto Associates. The subsoil consists of a deep deposit of heavily over-consolidated clay. Little differential settlements of any consequence need be anticipated of a single-span or a multi-span structure.

If we can be of further assistance in connection with these projects, please contact our Office.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGR.
Per:

AKLoh

(A. K. Loh,
PROJECT FOUNDATION ENGR.)

AKL/[illegible]deF
cc: S. McCombie
Foundations Office
Gen. Files.

Re: Proposed Crossing of Gravel Road
Revised Line between Lots 6 & 7,
Con. VI and Hwy. 401 - Raleigh Twp.,
Kent County - District 1
W.P. ~~87-59~~

CHECKING OF PRELIMINARY PLANS

Preliminary plans seem to be in agreement with the suggestions contained in the covering letter of the Soil Investigation Report (Dominion Soil Investigation, Ltd.).

Foundation elevation is 579.0'. It is not visible from the plan what bearing capacity has been chosen. In the covering letter, it was suggested to use 3 T/sq.ft.

Remark: The foundation overburden will be approx. 9'. On account of this, the allowable bearing capacity could probably be raised to 4 T/sq.ft.

A. Stermac
A. Stermac,
FOUNDATIONS OFFICE ENGINEER

AS/MdeF

April 11, 1960.

Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

March 3, 1960.

FOUNDATION REPORT - by Dominion Soil Investigation, Limited.

Attention: Mr. S. McCombie.

Re: Proposed Crossing of Gravel Road
Revised Line between Lots 6 & 7,
Con. VI and Hwy. 401 - Raleigh Twp.,
Kent County - Dist. 1 - W.P. 81-59.

The detailed foundation report prepared by Dominion Soil Investigation, Ltd., for the proposed structure at the above location, has been reviewed by the Foundation Section. Comments arising from the review of this report, are as follows:-

1. The subsoil at the above site is generally a clay till.

2. The proposed structure may be supported by spread footings founded at elevation 579.0' or lower. Between elevations 579.0' and 565.0', spread footings may be designed for an allowable bearing pressure of 3 $T/ft.^2$. The recommended allowable bearing pressure has been reduced from 4 $T/ft.^2$ to 3 $T/ft.^2$, after considering total settlement, and possible softening of the bearing surface of the till before placing the concrete.

3. If the footings are not placed immediately after the completion of excavation, consideration should be given to the placing of a 6" concrete working mat at the bottom of the excavation. This working mat will prevent softening of the bearing material.

cont'd. /2 ...

Comments: (cont'd.) ...

4. No problems associated with seepage water, or embankment stability, are anticipated.

If further queries arise regarding this report, please contact the Foundation Section.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGR.

per:

K Peaker

(K. Peaker,
FOUNDATION FIELD SUPERVISING ENGR.)

KP/MdeF
Attach.

cc: Messrs. A. H. Toye (2)
H. A. Tregaskes
D. G. Ramsay
A. Gater
G. U. Howell
J. Roy
A. Watt
Foundations Office
Gen. Files.

BA1018

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

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

884 WILSON AVE.,
DOWNSVIEW, ONT.
ME. 5-5921

Project: J 470

March 8, 1960.

Mr. A. Rutka,
Acting Materials and Research Engineer,
Materials & Research Section,
Department of Highways of Ontario,
Parliament Bldgs.,
Toronto 2, Ont.

Attention: Mr. L. G. Soderman

Foundation Investigation
Proposed County Road Underpass
Hwy. 401 - North-west of Ridgetown, WP 86-59

Dear Sirs:

The enclosed report describes the soil conditions encountered at the proposed County road underpass indicated above.

The soil at this location was found to consist of very stiff to stiff clay or silty clay down to bedrock depth, some 68 feet below the surface of the County road. According to shear strength measurements, abutment footings can be placed directly on the clay at Elev. 610 feet, or about $3\frac{1}{2}$ feet below the level of the adjacent fields. A safe bearing value of 4000 p.s.f. has been recommended.

Settlement of the abutments will take place at a very slow rate and this will result in large part from the weight of the adjacent embankment fill. A long term total settlement of 8 inches has been computed on the basis of laboratory tests. The actual settlement more probably will be about half this value. Somewhat less movement will occur in the embankments immediately adjacent to the structure. If it is proposed to use a centre pier in the bridge, some long term differential movement between the pier and the abutments should be anticipated. This movement could be of the order of two inches, with the pier settling less, ultimately, than the abutments.

The soil at all depths is quite strong enough to support the weight of embankment fill required for the approaches to this structure. Side slopes of $1\frac{3}{4}:1$ can be utilized. The fill can be placed at any time in the construction schedule.

We hope that the information contained in this report is sufficient for your design purposes. Please contact us if you have any queries about the foundation conditions.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

WAT/lt
Encl.

DEPARTMENT OF HIGHWAYS OF ONTARIO
MATERIALS & RESEARCH SECTION
PARLIAMENT BUILDINGS, TORONTO

FOUNDATION INVESTIGATION -
PROPOSED COUNTY ROAD UNDERPASS
HWY. 401 - NORTHWEST OF RIDGETOWN, WP 86-59

Project: J 470

March 8, 1960

William A. Trow & Associates Ltd.

TABLE OF CONTENTS

APPENDIX

ENCLOSURES

FOUNDATION INVESTIGATION
PROPOSED COUNTY ROAD UNDERPASS
HWY. 401 - NORTH-WEST OF RIDGETOWN, ONT. - WP 86-59

This report describes the soil conditions existing at the site of a County road crossing of Highway 401, near Ridgetown, Ont.

The capacity of the soil at this location has been appraised both with regard to its ability to support the bridge structure and to its stability under the weight of the approach fill. Details concerning field testing methods are given in Appendix.

Site Description

The site of this crossing is typical of the terrain in this part of Southern Ontario. The ground is extremely flat and is used either as pastureland or is sown to corn. Ditches have been cut through the flat countryside to carry away the seepage from the field tile of each farm. One of these ditches parallels the east side of the County road and then turns to the east about 100 feet north of the proposed crossing. The depth of this ditch is about 5 feet below road level, or about 3 feet below the adjacent field. A shallower ditch passes along the west side of the road. During the mild spell in the early part of the investigation, these ditches contained about 8 inches of water, but this run-off flow quickly dried up as the cold weather returned.

Subsoil Description

The subsoil at this site was determined by 5 borings, 2 of which were continued to assumed bedrock, and the remainder were terminated at a depth of about 35 feet. Detailed descriptions of the soil types at each test location are indicated in the borehole logs, Dwgs. 2 to 6 of this report. Also shown is a graphical record of strength and other physical properties.

In order to assist in the general appraisal of foundation conditions, the information from these logs has been summarized into the estimated stratigraphical profile shown in Dwg. 1. The following stratification is indicated:

1) Brown sandy silt

This is a loose surface deposit which extends to a maximum depth of about 3½ feet, or to elevations ranging from 614.4 to 611.1. This material contains the surface water of the adjacent flat ground. The ground water level, at the time of the investigation program was at approximate Elev. 612 feet.

(2) Very stiff clay with silt partings

This stratum underlies the sandy silt described above. It exists in a very stiff condition at upper levels but becomes somewhat less stiff with depth. The thickness of each silt parting varies and they appear to become less prominent with depth. Representative physical properties are as follows:

Undrained shear strength	-	2500 psf decreasing to 1000 psf
Penetration Resistance	-	5 to 19 blows per foot
Atterberg Limits	-	Liquid limit approx. 60%: Plastic Limit approx. 24%.
Natural Moisture	-	Close to plastic limit
Natural Unit Weight	-	Approx. 125 p.c.f.

The consolidation characteristics of this material are indicated by the test result shown in Dwg. 7. It is seen that the soil is heavily overconsolidated and therefore will only undergo recompression from the bridge and embankment loads.

(3) Stiff grey silty clay glacial till

This soil type is quite widespread in the Ridgetown - Blenheim area. At this location it exists in a stiff to very stiff condition. It contains small gravel particles and the percentage of this gravel varies with depth. Typical physical properties are as follows:

Undrained shear strength	-	1750 to 4300 p.s.f.
Penetration Resistance	-	14 to 41 blows per foot.
Atterberg Limits	-	Liquid Limits approx. 34%, Plastic Limits approx. 17%.
Natural Moisture	-	Close to plastic limit.
Natural Unit Weight	-	Approx. 132 p.c.f.

Consolidation Characteristics as indicated in Dwg. 7.

The top surface of this glacial till is at approximate Elev. 600 feet; its bottom limit was found at Elev. 559 feet in Hole 5 at the south-west corner of the site, and at Elev. 572 feet in Hole 4, near the north-east corner.

(4) Stiff reddish grey clay

This very adhesive deposit lies below the glacial till. It contains inclined partings of coarse silt. It is deep enough below footing level so that its effect on the proposed construction will be slight.

(5) Dense wet silty sand with gravel

This material was encountered in Hole 5 just over bedrock. It exists in a very stiff condition.

(6) Bedrock

Although not proven, bedrock was encountered at Elev. 547 feet. It consists of dark grey shale.

Discussion of Foundation Requirements

Except for a slightly less stiff condition encountered near Elev. 600 feet in Hole 4, the subsoil at this County road underpass location was found to be quite uniform in physical character, and it exists in an overconsolidated state. Bridge footings can be placed directly on the soil at Elev. 610 feet, or about $3\frac{1}{2}$ feet below the present level of the ground. On the basis of undrained shear strength measurements, the safe bearing value to apply at this level is 4000 p.s.f.

From the results of consolidation tests, the settlement to be anticipated at the abutment locations is about 8 inches. This estimate is believed to be very conservative and, in any event, the movement will take place over an extremely long period of time. More than half of the settlement will be caused by the weight of embankment fill and therefore the bump or change in level at the intersection of the the bridge structure and the fill will be slight.

Some seepage into the footing excavations may be experienced if work is undertaken during wet periods of the year. This water will travel from the adjacent ditches through thin seams of sand in the clay. The amount of seepage should not be a serious deterrent to construction however. The sides of the excavation should stand unsupported.

Although stability analyses have not been carried out, the shear strength of the soil is quite adequate to support the weight of the embankment safely. No failure, either at right angles to the centre line of the fill or under the abutment foundations, will occur. If desired, the embankment fill can be placed on slopes of $1\frac{3}{4}$:1. The full shear strength of the soil under the abutment footings should be available to resist the horizontal thrust of the fill.

Summary of Observations and Comments

1) The site of the proposed Hwy. 401 Underpass of this County road is underlain by stiff to very stiff clay or silty clay glacial till. The water table in the area is high, because the ground is flat; the ground water at the time of the investigation was at Elev. 612 feet, which is the approximate level of the bottom of the adjacent drainage ditches. Shale bedrock lies about 68 feet below the present surface of the County road.

2) Abutment footings can be placed directly on the soil at Elev. 610 feet, or about $3\frac{1}{2}$ feet below the surface of the adjacent fields. The recommended bearing value to apply is 4000 p.s.f. Some seepage may enter the footing excavations if work is undertaken during wet periods of the year. This water should not be a serious deterrent to construction however. The sides of the excavation will stand unsupported.

3) Settlement computations, based on two laboratory test results have indicated an overall settlement of 8 inches. Because of sample disturbance and other factors, this estimate is believed to be quite high. In any event, the movement will occur at a very slow rate, lasting for several decades. However, the settlement will be of concern if the bridge structure incorporates the use of a centre pier. The centre pier will not settle as much as the abutments because it will not be affected by the weight of embankment fill. Long term differential settlement across the structure will result.

4) The embankment approaches to the bridge structure will be stable. The fill can be placed at any convenient period in the construction program.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
W. A. TROW
PROVINCE OF ONTARIO

W. Trow

William A. Trow (P. Eng.)

WAT/lt
March 8 1960.
J 470

APPENDIX

SETTLEMENT COMPUTATIONS

Settlement of Abutments

Assume footings at Elev. 610 feet, applying a net bearing pressure of 4000 p.s.f.
Consolidation characteristics for soil as indicated by tests on samples from Hole 2, 10-12 feet, and Hole 4, 34-36 feet, Dwg. 7.
Assume approach embankments 20 feet high, top width = 40 feet, and side slopes of $1\frac{3}{4}$:1.
Take fill and soil weight above water table at 125 p.c.f.; weight below water table = 70 p.c.f.; water table at El. 612 feet.

From Newmark diagram - average increment of pressure at 10 feet depth, or Elev. 605 ft., due to embankment = 1420 p.s.f.
Pressure due to footing = 1780 p.s.f.
Present in [illegible] pressure, at Elev. 605 ft., approximately 900 p.s.f.
Coefficien[illegible] compressibility, Mv from test results Hole 2, 10-12 ft. =

$$\frac{e_o - e_1}{\Delta p\,(1+e_o)}$$

For $P_o = 900$, $e_o = .871$

For $P_o + \Delta p = 900 + 1420 + 1780 = 4100$, $e_1 = .821$

Therefore $Mv = \frac{.871 - .821}{3.2\ (1.871)} = .00835$ sq.ft./kip.

Assume this value to be the average modulus of compressibility for the upper plastic clay down to Elev. 599 feet.

Similarly, Coefficient of Compressibility for conditions existing at depth of 35 feet in Hole 2 = 0.00677 sq.ft./kips. = estimated average modulus for the silty clay till below Elev. 599 feet.

Depth Below Footing	Pressure Due to Footing	Pressure due to Fill	Total Press. Δp	$S = HMv\Delta p$
$2\frac{1}{2}$	2.4	1.6	4.0	2
$7\frac{1}{2}$	1.35	1.35	2.7	1.35
$12\frac{1}{2}$	0.85	1.2	2.05	1.02
$17\frac{1}{2}$	0.60	1.1	1.7	0.69
$22\frac{1}{2}$	0.42	1.02	1.44	0.58
$27\frac{1}{2}$	0.36	1.0	1.36	0.55
$32\frac{1}{2}$	0.29	0.99	1.28	0.51
$37\frac{1}{2}$	0.24	0.97	1.21	0.5
$42\frac{1}{2}$	0.19	0.97	1.16	0.[illegible]

Total approx. 8 inches.

Since the in situ compressibility of the soil will be much lower than the laboratory test measurements, the estimated movement of 8 inches should be taken as the very conservative upper limit.

The settlement of the ground under the fill adjacent to the abutment will be very slightly less than the amount experience at the abutments. Because of the low permeability of the foundation clay and the great thickness of these deposits, settlement will continue at a very slow rate extending over several decades.

\- - - - - - - -

FIELD INVESTIGATION METHODS

The borings of this investigation were performed using continuous flight auger equipment. The holes were uncased for the entire sampling depth.

Two of the borings, Numbers 5 and 4 at the south-west and north-east corners of the proposed structure, were taken to refusal and assumed bedrock. The remainder were terminated at a depth of about 35 feet.

Samples were taken at relatively close intervals of depth in the first approximately 15 feet and then the spacing was increased to 5 foot, and in Hole 4 to 10 foot intervals. Undisturbed 2-inch I.D. shelby tube samples were recovered from Holes 4 and 5 and in one instance in Hole 2. In the remaining holes, split sppon samples were obtained. In most cases, it was possible to push the shelby tubes into the soil under hydraulic pressure. The split spoons were driven under an energy of 350 ft.lbs. per blow. An attempt was made to drive the split spoon a short distance into bedrock in Holes 4 and 5, and a small amount of shale was recovered in the latter instance. Field vane tests were attempted below each sampling interval but in most cases the shear strength of the soil was beyond the capacity of the vane.

Water level observations were taken as the boring program progressed and for a period of about 5 days thereafter. The elevations of the ground surface at each boring location was obtained using a D.H.O. benchmark as reference. The location of this bench mark is shown in Dwg. 1.

WILLIAM A. TROW AND ASSOCIATES LTD.

TABLE NO. 1
SUMMARY OF LABORATORY AND FIELD TEST MEASUREMENTS

Elev. Feet	Shear Str. Ksf & Nat. Unit Wt. γ p.c.f. Hole 1 615.9	2 616.1	3 613.1	4 615.7	5 615.2	Penetration Resistance Blows/ft. Hole 1	2	3	4	5	Natural Moist. % Dry Weight Hole 1	2	3	4	5	Atterberg Limits % Dry Weight H 4 L.L.	H 4 P.L.	H 5 L.L.	H 5 P.L.
612						9	10		push	pusn	32.8	19.0							
610	V=1848	V=1890(check) V=1848	V=1260		V>2100		5	5				25.0	25.4						
608		γ=123.6 Q=2.15		γ=124 Q=2.36	γ=126.2 Q=2.50	8			push	push	35.0	24.8		24.6	27.9				
606	V>2100	V=1470	V>2100	V>2100	V>2100		19	15				21.4	33.7						
604		V>2100		γ=120 Q=1.160	γ=135.1 Q1.75	19			push	push	29.0	31.0		30.2	28.1	56.7	23.0	62.0	24.0
602	V>2100	γ=127	V>2100	V>2100	V>2100		13	12				32.2	32.1						
600		V>2100		γ=116.5 Q=1.0	γ=138 Q=1.94	12			push	push	25.7			36.0	19.7	60.0	24.5	35.3	17.2
598			V>2100	V=1596	V>2100			11					28.5						
596	V>2100				γ=130 Q=1.75		24		push	push		17.8			18.1				
594		V>2100	V>2100	V>2100	V>2100	25		14			18.0		24.5						
592	V>2100				γ=135.4 Q=3.5					push					14.2			29.3	14.8
590			V>2100	γ=139 Q=3.45	V>2100	41	29	23	push		15.6	16.7	16.1	14.2		27.1	15.3		
588	V>2100	V>2100		V>2100	γ=134.5 Q=3.8					push					15.6				
586			V>2100		V>2100		27	25				18.6	17.1						
584		V>2100		γ=132 Q=1.35		39			push	24	17.8			19.5	19.6				
582	V>2100			V>2100	V>2100			26					18.2						
580			V>2100	γ=132.8		36	32		push 8" 40-6"		20.3	23.4		19.0					
578	V>2100	V>2100		γ=129.4 Q=4.32						push					18.5			34.4	16.7

TABLE NO. 1 (CONT.) P 2

Elev. Feet.	Shear St. Ksf & Nat. Unit Wt. γ p.c.f. Hole 1 615.9	2 616.1	3 613.1	4 615.7	5 615.2	Penetration Resistance Blows/ft. Hole 1	2	3	4	5	Natural Moist. % Dry Weight Hole 1	2	3	4	5	Atterberg Limits % Dry Weight H 4 L.L.	H 4 P.L.	H 5 L.L.	H 5 P.L.
576					V>2100				28					15.8					
574										27					18.4				
572				γ=124 Q=1.40	V>2100				push					27.4					
570																			
568					γ=133.8 Q=3.5					push					19.4			33.4	17.5
566					V>2100				37					23.6					
564																			
562										40					18.8				
560					V>2100														
558					γ=124 Q=2.95					push					28.0				
556					V>2100				30					25.7					
554																			
552										19									
550					V>2100														
548									Refusal	Refusal									
546									547.3	546.7									
544																			

LEGEND: Q = Undrained shear strength determined by triaxial test.
V = Field vane shear strength
LL = Liquid Limit
PL = Plastic Limit

PROJECT NO.

DRAWING NO. 2

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

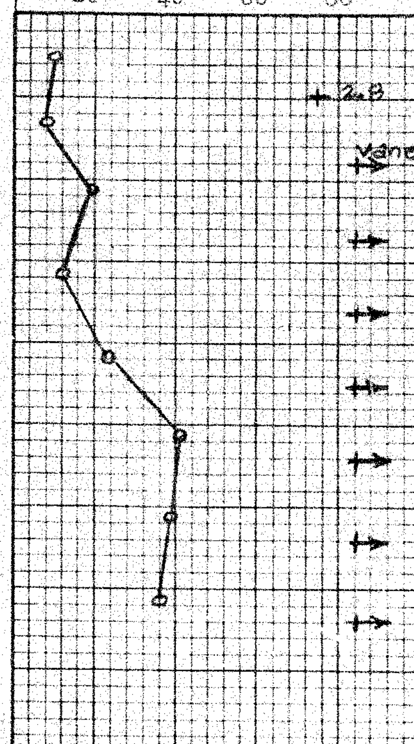
PROJECT Proposed County Rd. Overpass of Hwy. 401
LOCATION North Ridgetown WP 86-59
HOLE LOCATION See Dwg. No. 1
HOLE ELEVATION AND DATUM 615.9 BM see Dwg. 1

BOREHOLE NO. 1
FIELD SUPERVISOR
DRILLER
PREP.

LEGEND
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (Qu)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Side of Road	615.9	0
	Gravel	614.4	
	WL	612.2	
	Very stiff brown clay with coarse silt partings.		10
		~599	
	Very stiff grey silty clay with fine gravel sizes (Glacial Till)		20
			30
		579	
	End of Bore.		40

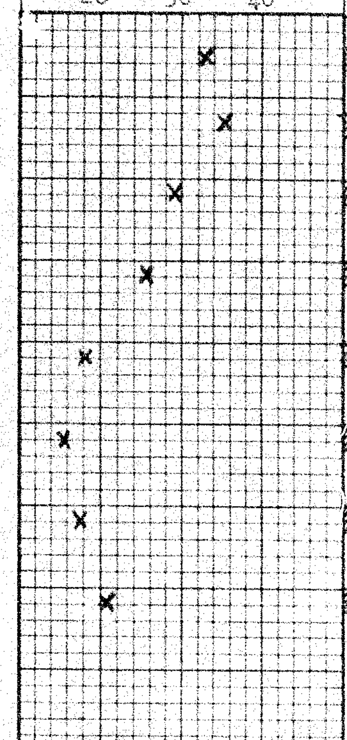
STRENGTH AND PENETRATION RESISTANCE — Shear Str. 1000, 2000 P.S.F.; BLOWS/FT. 20, 40, 60, 80



CONSISTENCY — MOIST. CONTENT - % DRY WT. 20, 30, 40

SAMPLE

NATURAL UNIT WT. P.C.F.



NOTES: 1) Boring by continuous flight auger
Hole uncased to full depth.
(2) Samples driven under an energy of
350 ft.lbs. per blow.
(3) Water level after 2 days - 3.7 ft.

PROJECT NO. ....

DRAWING NO. 3

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

LEGEND
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION [Qu] ⊕
- VANE TEST [C] AND SENSITIVITY [S] $+^{S}$
- NATURAL MOISTURE AND LIQUIDITY INDEX X^{LI}
- LIQUID LIMIT
- PLASTIC LIMIT

PROJECT Proposed County Rd. Overpass of Hwy.401 BOREHOLE NO. 2

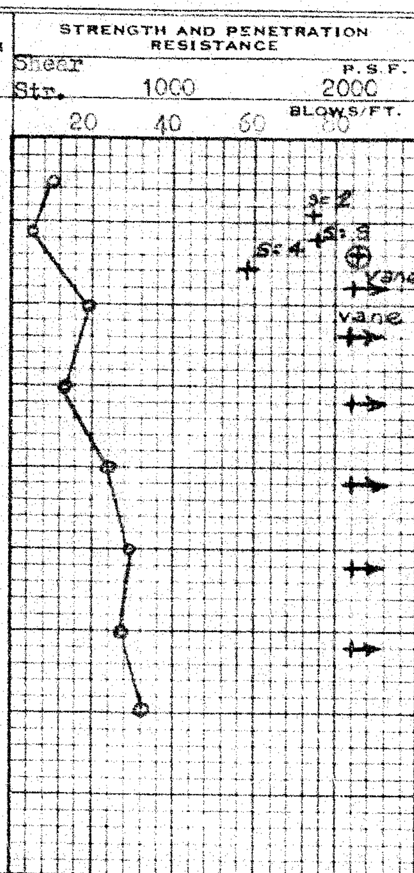
LOCATION North Ridgetown WP 86-59 FIELD SUPERVISOR

HOLE LOCATION See Dwg. No.1 DRILLER

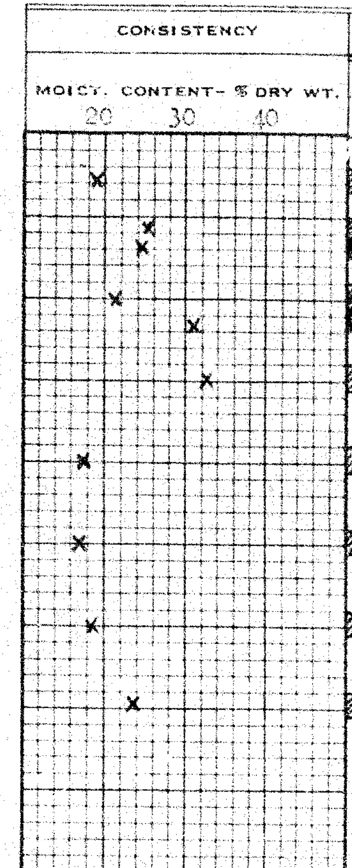
HOLE ELEVATION AND DATUM 616.1 BM - See Dwg.1 PREP.

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Side of Road	616.1	0
	Pit run gravel	614.4	
	W.T.	612.1	
	Stiff moist brown sandy clay becoming very stiff brown clay with coarse silt partings (water enters hole through sand seams at approx. 5 ft.)		10
	Very stiff grey silt clay with fine gravel sizes (glacial till)	~599	20
			30
	End of bore. NOTES: (1) and (2) as in Hole 1 (3) Water level after 2 days= 4 ft.	580.6	40

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY



SAMPLE	NATURAL UNIT WT. P.C.F.
	123.6
	127

PROJECT NO.

DRAWING NO. 4

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Proposed County Rd. Overpass of Hwy. 401
WP 86-59
LOCATION North Ridgetown
HOLE LOCATION See Dwg. No. 1
HOLE ELEVATION AND DATUM 613.1 BM - see Dwg. 1

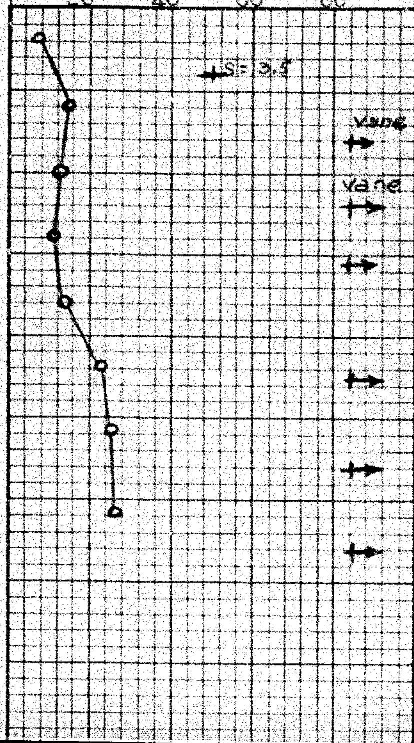
BOREHOLE NO. 3
FIELD SUPERVISOR
DRILLER
PREP.

LEGEND
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION [Qu]
- VANE TEST [C] AND SENSITIVITY [S]
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Ground surface	613.1	0
	5 ins. topsoil		
	Brown sandy silt W.T.	611.1	
		~610.5	
	Very stiff brown clay with fine silt partings.		10
	Layer silty clay till 17-18 ft.		
		~595	20
	Stiff grey silty clay with fine gravel - glacial till		30
	End of bore.	581	
	NOTES: (1) and (2) as in Hole 1		
	(3) Water lever after 4 dys. = 2 ft.		40

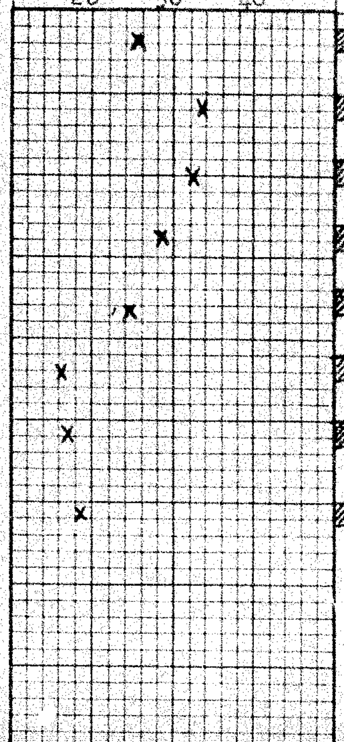
STRENGTH AND PENETRATION RESISTANCE

Shear Str. P.S.F. 1000 2000
BLOWS/FT. 20 40 60 80



CONSISTENCY

MOIST. CONTENT - % DRY WT. 20 30 40



SAMPLE

NATURAL UNIT WT. P.C.F.

PROJECT NO.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

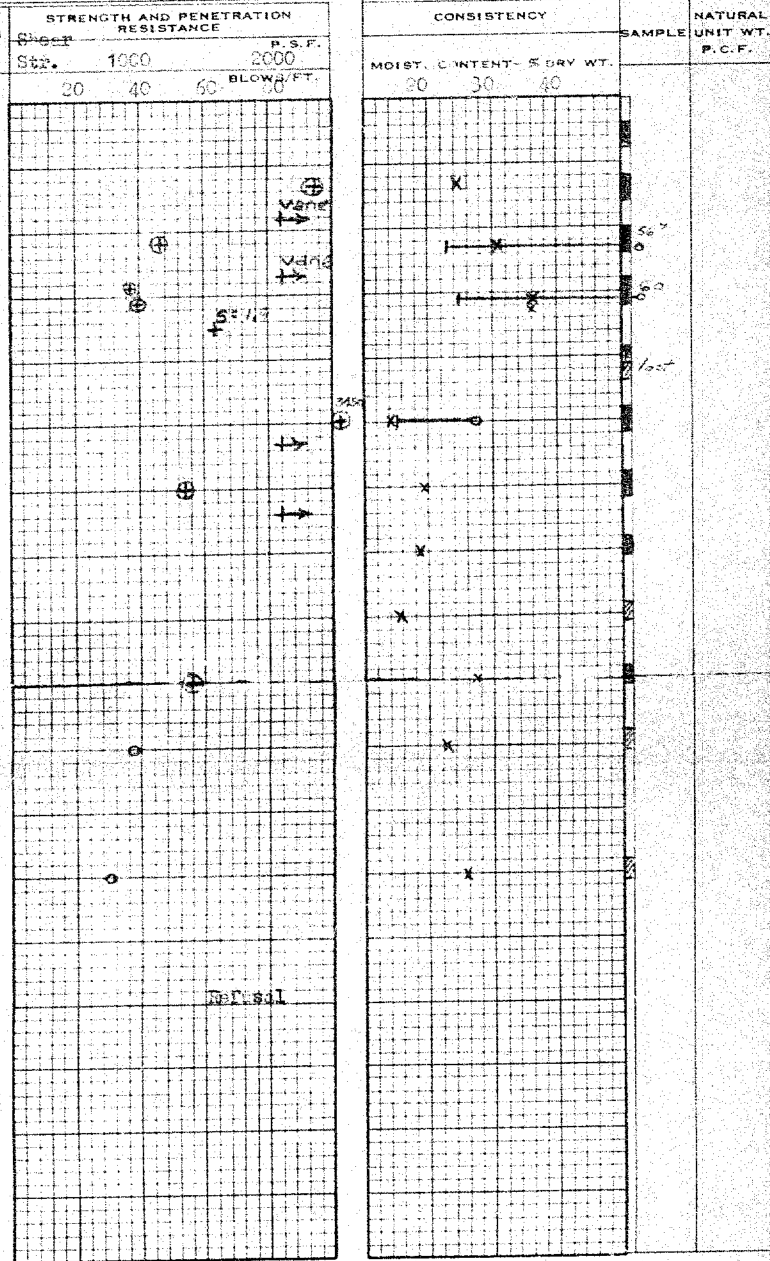
LEGEND
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

PROJECT Proposed County Rd. Overpass of Hwy. 401 WP 26-59
LOCATION Near Ridgetown
HOLE LOCATION See Dwg. No. 1
HOLE ELEVATION AND DATUM 615.7: For BM see Dwg. 1

BOREHOLE NO. 4
FIELD SUPERVISOR
DRILLER
PREP.

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Side of Road	615.7	0
	Gravel	614.2	
	Grey br. silt, sand & some topsoil	612.7	
	Very stiff brown clay with coarse silt partings.		10
	Stiff grey silty clay with fine gravel sizes (Glacial Till	599	20, 30, 40
	Stiff reddish grey very adhesive clay with inclined silt partings.	572	50, 60
	Black shale - Bedrock.	547.3	70

NOTES: (1) and (2) as in Hole 1
(3) All shelby tubes pushed.



PROJECT NO.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

DRAWING NO.

LEGEND
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION [Qu]
- VANE TEST [C] AND SENSITIVITY [S]
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

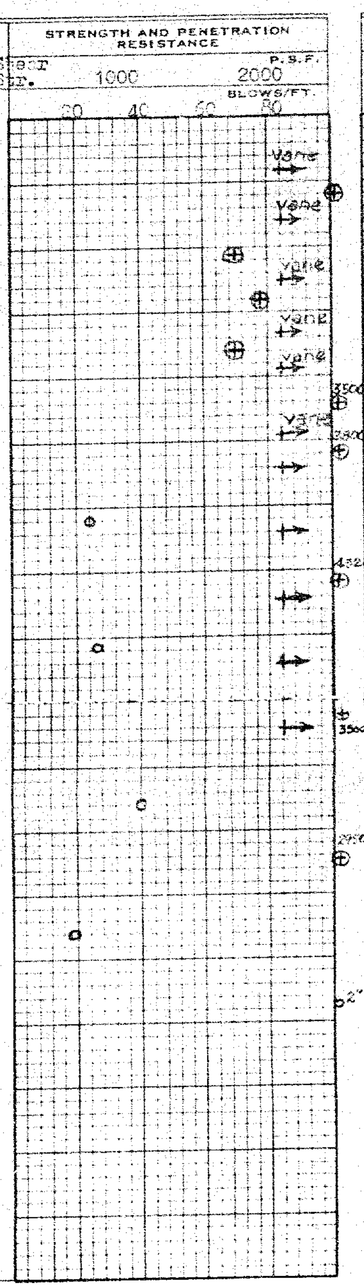
PROJECT County Road Overpass of Hwy.401: WP 88-59
LOCATION Near Ridgetown
HOLE LOCATION See Dwg. No.1
HOLE ELEVATION AND DATUM 615.2 See Dwg.1 for BM

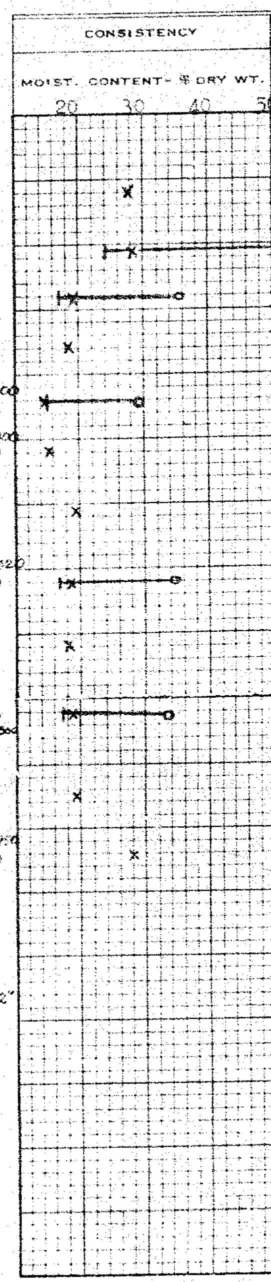
BOREHOLE NO. 5
FIELD SUPERVISOR
DRILLER
PREP.

DESCRIPTION	ELEV. FEET
Ground surface	615.2
Approx. 3 ins. topsoil	611.8
Fine sandy cohesive silt with root fibres (Wet sand seam) W▽T (3.4)	~611
Very stiff brown clay with coarse silt partings or thin layers.	~603
Stiff to very stiff grey silty clay with rounded gravel (Glacial Till)	559
Very adhesive stiff reddish grey clay with silt partings	553
Dense wet silty sand with some gravel	546.7
Bedrock: Black shale End of bore.	

NOTES: (1) and (2) as in Hole 1
(3) All shelby tubes pushed in by machine.
(4) Water level after 5 days - 3.3 ft.

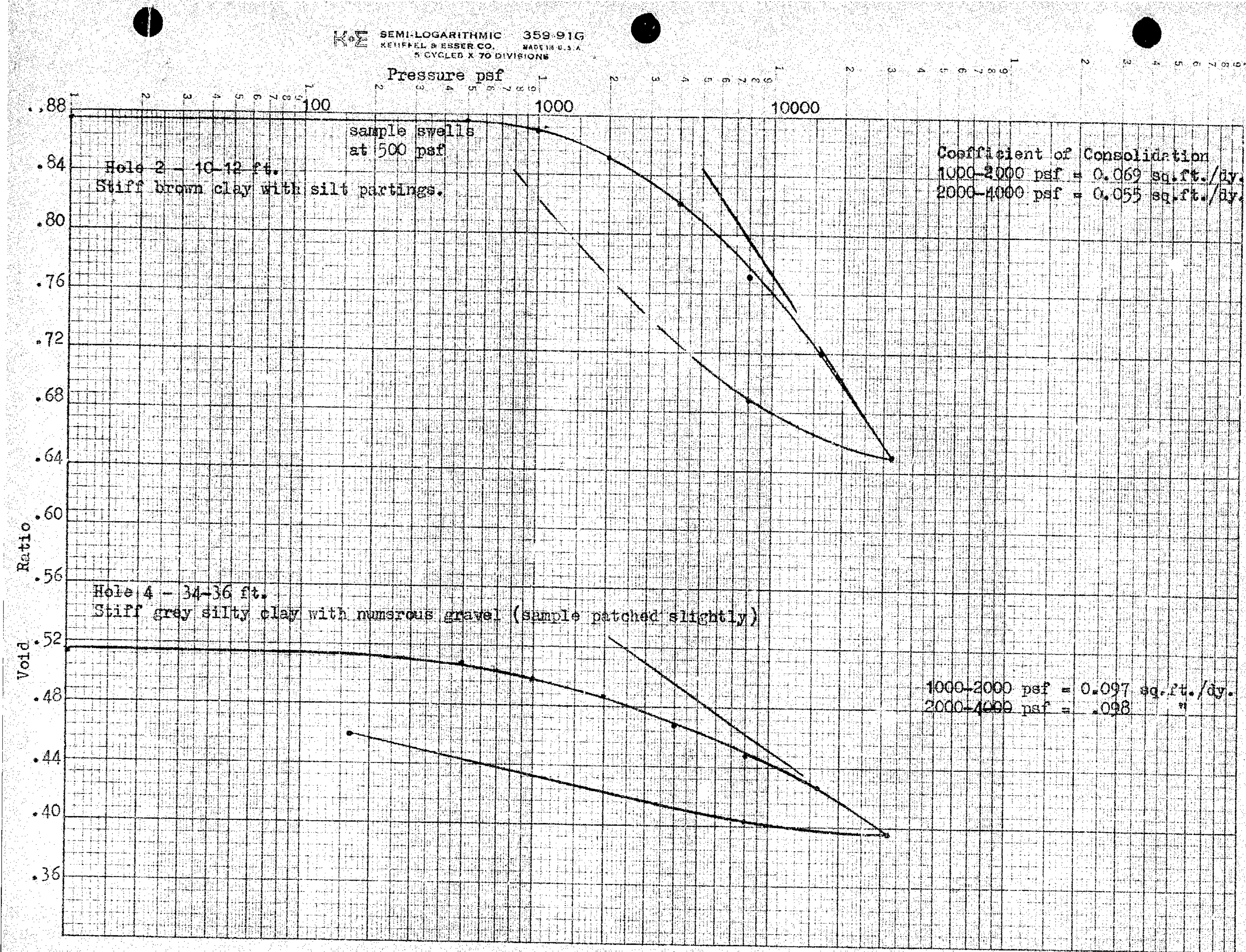
DEPTH FEET: 0, 10, 20, 30, 40, 50, 60, 70





SAMPLE

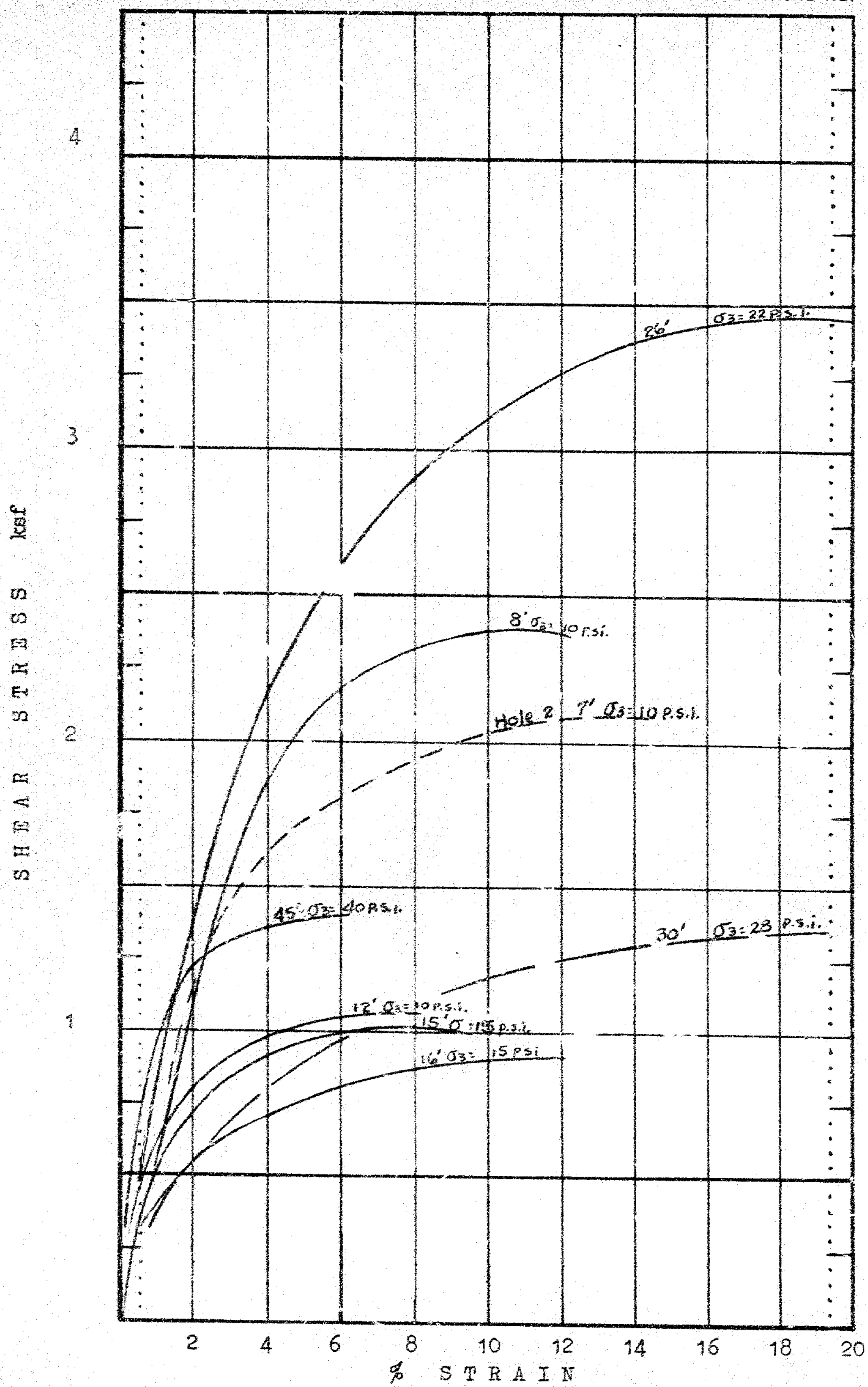
NATURAL UNIT WT. P.C.F.
126.2
135.1
138
130
135.4
134.5
129.4
133.8
124



CONSOLIDATION TEST RESULTS HOLES 2 and 4 WF 86-59

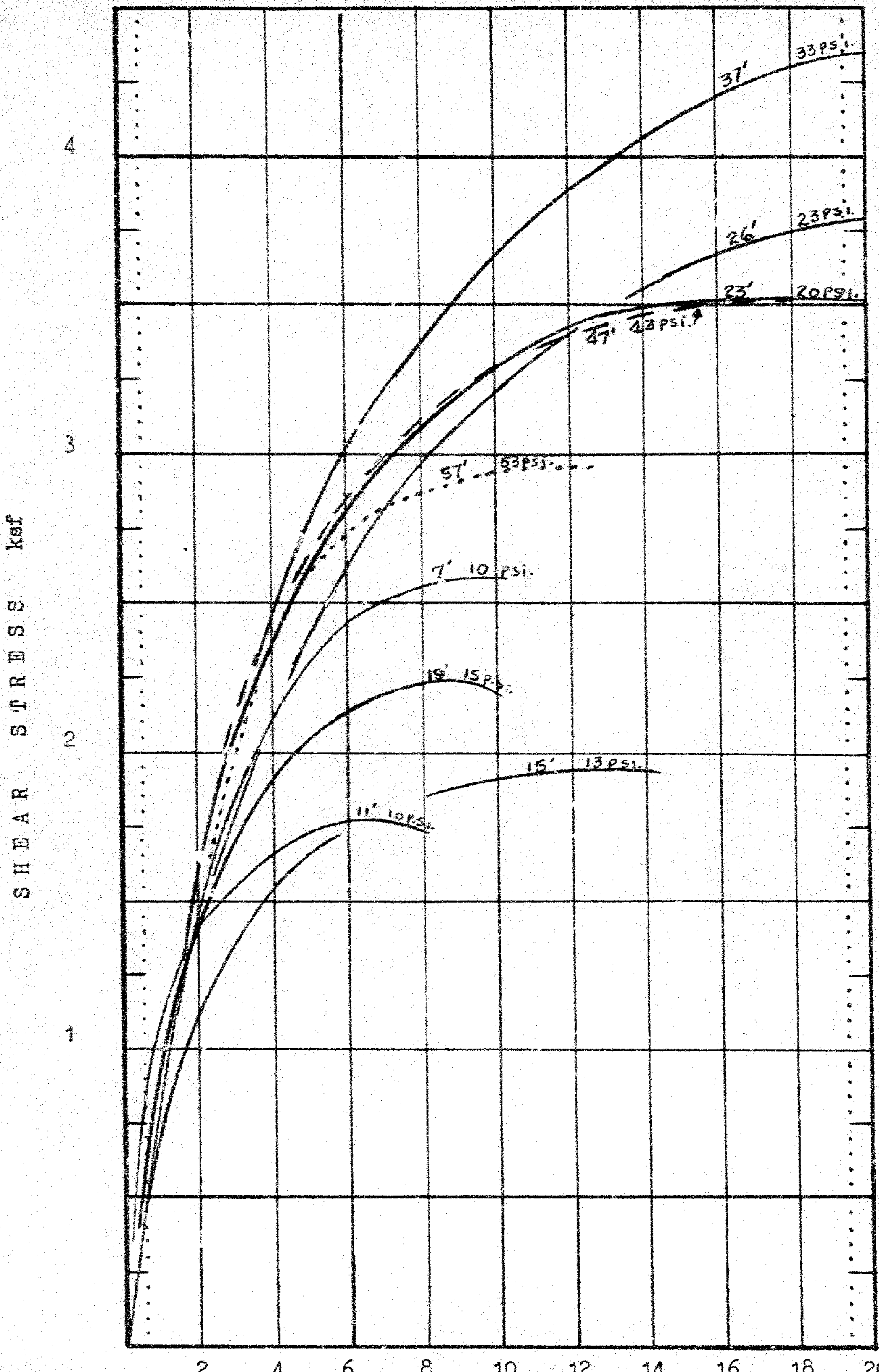
Project J470

Dwg. 7



UNDRAINED TRIAXIAL SHEAR TEST RESULTS

HOLES 4 and 2 - WP 86-59

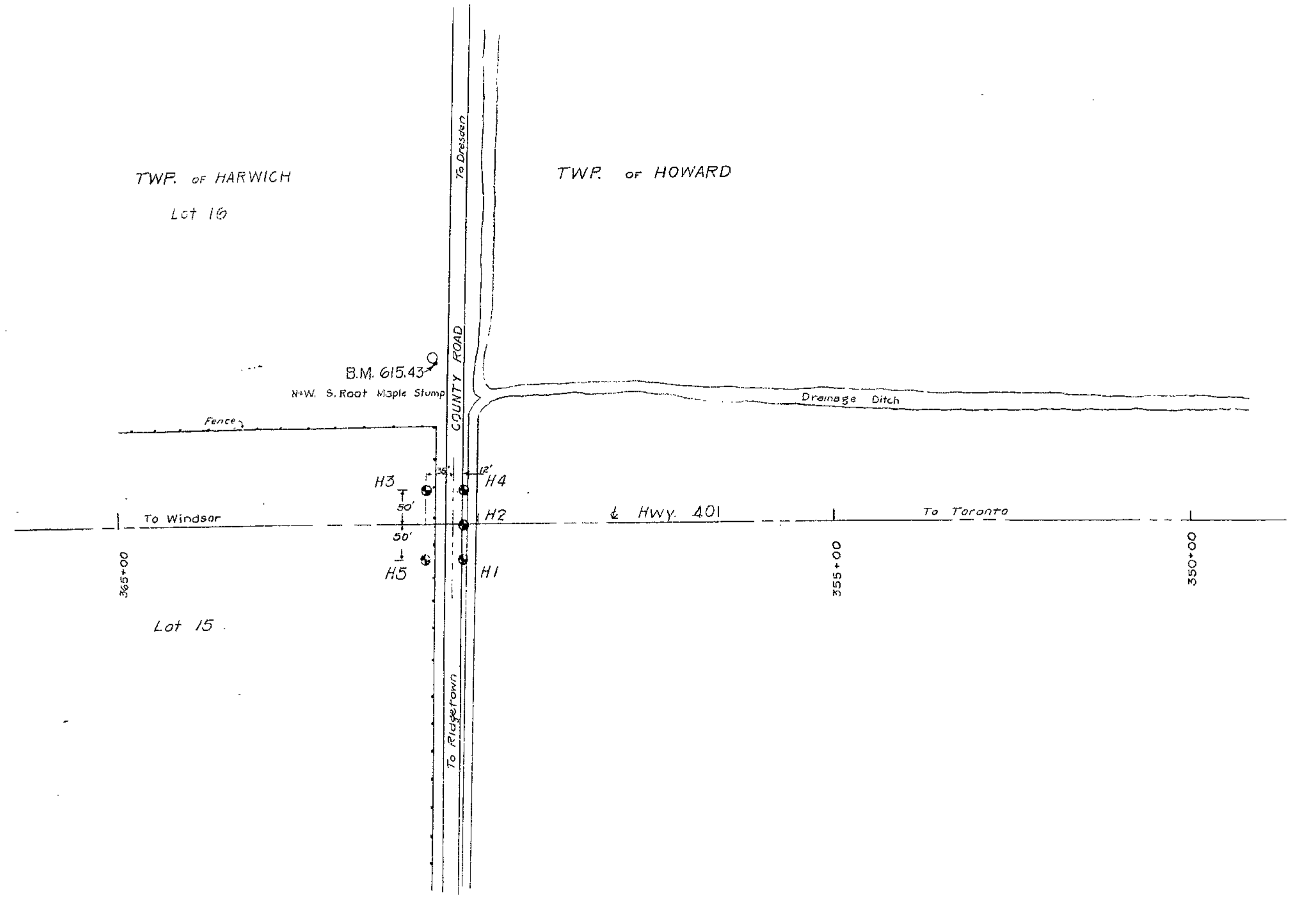


UNDRAINED TRIAXIAL SHEAR TEST RESULTS

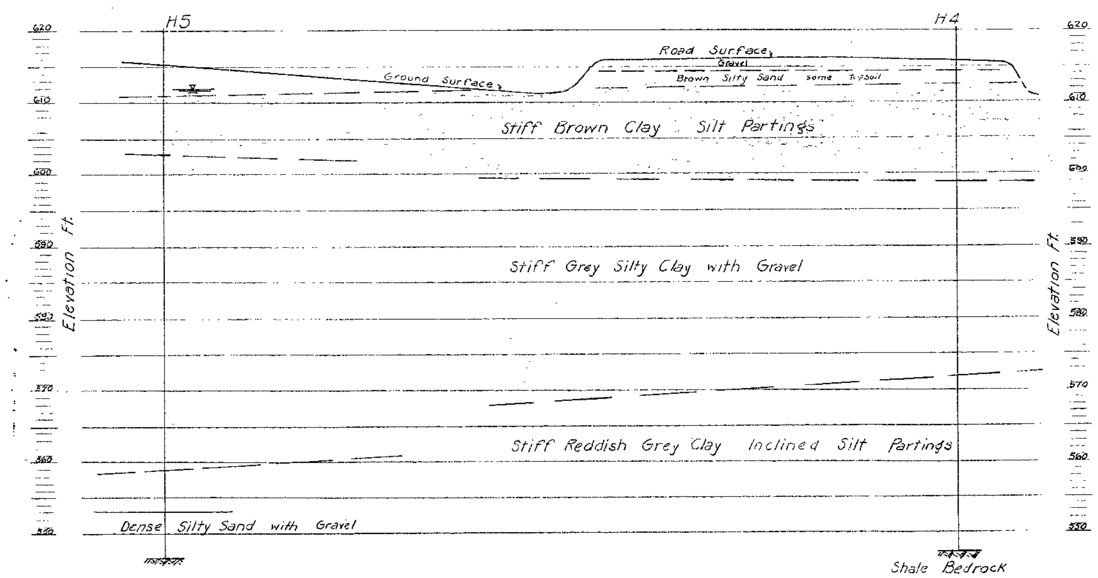
HOLE 5 WP 86-59

WILLIAM A. TROW AND ASSOCIATES

#60-F-286-C
W.P. #86-59
HWY. #401,
UNDERPASS, CTY.
RD NO. #15.
RIDGETOWN



BOREHOLE LOCATION PLAN
1 inch = 100 Ft.



ESTIMATED STRATIGRAPHY
Scale 1" = 10'

PROPOSED COUNTY ROAD UNDERPASS

WP 86-59

William A. Trow & Assocs Ltd. Mar 9 1960