



ONTARIO
DEPARTMENT OF HIGHWAYS

Memo to Mr. A. M. Toye, Date March 21, 1960.
Bridge Engineer. Subject FOUNDATION INVESTIGATION -- b
From Materials & Research Section. William A. Trow & Associates,
Limited.

Attention: Mr. S. McCombie.

Re: Proposed Hwy. 21 Interchange,
Hwy. 401 - Ridgetown, Ontario,
W.P. 89-59 -- District No.1.

The Foundation report on the subsoil conditions at the above site, recently submitted by W. A. Trow & Associates, Ltd., is forwarded herewith.

We have reviewed the contents of this report and agree with the recommendations made by the Consultant. The subsoil is a heavily, overconsolidated clay till and is competent to support the approach fills and structural loadings through the use of spread footings.

If piles are to be driven through the approach fills, large displacement piles should be specified, and driven to a depth of 10 feet below the existing ground surface. 'H' piles are not recommended.

If a multi-span, continuous structure is proposed, a differential settlement of the order of 1-1/2" - 2" can be expected between abutments and adjacent piers for end span distances of the order of 50 feet.

If we can be of further assistance in the design of this structure, please do not hesitate to call our Office.

L. G. Soderman

LGS/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.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

BA 1025

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

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

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

Project: J 478

March 25, 1960.

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

Attention: Mr. L. G. Soderman

Foundation Investigation
County Road Underpass
Hwy. 401 - North of Ridgeway, Ont. WP 89-59

Dear Sirs:

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

The predominant soil here was found to be very stiff to stiff silty clay glacial till which begins about 6 to 9 feet below the ground surface. It is overlain by fine brown sand which is wet below 2 feet. The till was proven for a depth of 97 feet.

We have indicated that the bridge structure can be founded at the top surface of the glacial till, at which level a safe bearing value of 6500 psf is available. Very long term settlement of the abutments and adjacent embankment fill has been estimated to be about 6 inches.

We believe that the information contained in this report is sufficient to permit you to appraise the site conditions at this location. If you have any queries concerning the soil properties or the anticipated behaviour of the soil under load, we shall be pleased to discuss them with you.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

WAT/lt
Encl.

WILLIAM A. TROW AND ASSOCIATES

DEPARTMENT OF HIGHWAYS OF ONTARIO
MATERIALS AND RESEARCH SECTION,
PARLIAMENT BLDGS., TORONTO, ONT.

FOUNDATION INVESTIGATION
COUNTY ROAD UNDERPASS
HWY. 401 - NORTH OF RIDGETOWN, ONT. WP 89-59.

Project: J 478

William A. Trow and Associates Ltd.

March 25, 1960.

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FOUNDATION INVESTIGATION
COUNTY ROAD UNDERPASS
HWY. 401 - NORTH OF RIDGETOWN, ONT. WP 89-59

This report describes the soil conditions existing at the site of the intersection of Hwy. 401 and the County road separating Howard and Orford Townships north of Ridgetown, Ont. Recommendations concerning the permissible bearing values and foundation depths have been given and estimates have been made of the amount of settlement to be expected after the bridge and its approach embankments have been installed. Details concerning investigation methods are given in the Appendix.

Site Description

The land in the vicinity of this County road underpass site is slightly more irregular than the crossings investigated a few miles farther to the south-west. However, the ground is still relatively flat and drainage ditches are required to carry off the seepage from the farm tile which are placed at regular intervals throughout the adjacent fields. The locations of two of these ditches are indicated in the borehole location plan, Dwg. 1. Some wood lots lie along the route of the proposed highway a few hundred feet to the east and west of this crossing. The site was snow-covered during the investigation period.

Subsoil Description

The soil conditions at this site were indicated by four borings, two of which were carried to depths of 65 and 97 feet and the remainder were terminated about 25 feet below the present ground surface. The information obtained from these borings has been recorded in the borehole profiles designated Dwgs. 2 to 5 of this report. The locations of the holes are shown on Dwg. 1.

In order to assist in the description of the subsoil types, the information from the two deep borings has been summarized into the profile shown in Dwg. 1. The following stratigraphy is indicated.

Brown sand: This material extends from ground surface to elevations ranging from 652 feet at the north-west side of the bridge site to Elev. 656 feet at the south-east. According to field penetration measurements, it exists in a medium dense condition and it contains ground water below a depth of about 2 feet, or Elev. 658.6 feet. Gravel was noted in this material below Elev. 856 feet and clay layers were found below a depth of about 6 feet.

Stiff grey silty clay: This glacial till deposit underlies the sand described above at a depth of 5 to 9 feet below the ground surface. It contains numerous embedded particles of fine gravel and it is quite similar to materials noted in bridge investigations a few miles to the south-west.

Near the top of this deposit the soil is quite stiff with an undrained shear strength in the order of 3000 p.s.f. However, the strength

diminishes somewhat with depth and at approximate Elev. 605 feet it reaches a value of 1860 psf. The plasticity of the till deposit is relatively uniform with depth, with average values of liquid and plastic limits of approximately 35 and 17 percent respectively. In the first few feet of depth the moisture content of the clay is at the plastic limit, but at greater depths it is 2 or 3 percent above this limit.

Hole No.4 was taken to a depth of 97 feet below the ground surface. Below 70 feet, the soil becomes sandier as evidenced by the lower moisture contents noted below this level. Below 90 feet, the sand was very prominent. It became very difficult to auger or to withdraw the augers below the 80 foot depth, and therefore the hole was terminated at 97 feet, or Elev. 563 feet, as indicated.

On the basis of information obtained from other investigations performed immediately to the south-west, bedrock probably lies another 20 feet farther down, or about 120 feet below the surface. The record of bedrock levels at these other sites are as follows:

Project No.	Location	Dist. from this site in mi.	Bedrock Level
WP 83-59	North of Blenheim	11½	524.4
WP 86-59	West of Ridgetown	7	547.3
WP 87-59	N.W. of Ridgetown	4½	532.6
WP 88-59	North of Ridgetown	2	541.3

In view of the very competent condition of the soil underlying this site, and the great difficulty experienced in augering at these depths, the extension of the holes to bedrock did not appear to be warranted.

Discussion of Foundation Requirements

Reference to the summary of laboratory test data, recorded in Table No.1, indicates that the shearing resistance of the clay till about 9 feet below the ground surface has a value ranging from 3200 to 3450 p.s.f. Since the safe net bearing value of cohesive soils is equal approximately to twice this cohesive resistance strength for the footing sizes required for this structure, the recommended net footing pressure to apply is 6500 p.s.f. If no approach fill was involved, the total settlement to be anticipated with the use of this pressure would be one inch.

Although the upper fine sand below a depth of about 4 to 5 feet is quite competent to support the weight of the bridge abutments, it is suggested that the footings be taken right down to the clay till. This is because the sand is wet below a depth of about 2 feet and it will become disturbed during excavation. The ground water in the sand can be removed by pumping from open sumps dug down to the clay.

Even though the silty clay glacial till, underlying this site, exists in a very stiff to stiff condition, settlement of the abutments and approach fill must take place because the bearing stresses from these overpass members will affect such a great depth of clay. No consolidation tests were performed on this glacial deposit because its compressibility characteristics have been indicated in investigations referred to earlier in this section. Test results from these other projects are included in this report.

In the project designated WP 83-59, the subsoil conditions were found to be quite similar to those existing at this location, although the thickness of clay was 80 feet, or somewhat less than the depths indicated here. The estimated long term settlement at that site, for identical loading conditions, was computed from test results to be about 10 inches. In view of inevitable disturbances during sampling and preparation of the test specimens, this estimate was felt to be unduly conservative and a long term movement of about 5 inches seemed to be more probable.

Because the soil and loading conditions at this site are similar to the WP 83-59 project, the ultimate movement should be proportional. Therefore the anticipated settlement resulting from the compression of 110 feet of clay is in the order of 6 inches. Assuming drainage at the top and bottom surface of the till, 90% of this movement should be complete in about 55 years. If a centre pier is contemplated for this bridge structure, long term differential settlement will occur between it and the abutments. At very great depth the stresses from the fill will tend to compress the soil directly below the pier. Therefore the ultimate differential settlement between this unit and the abutments should not exceed about 4 inches.

No embankment stability problem exists at this site since the shear strength of the soil is more than sufficient to support the weight of fill proposed.

Summary of Comments

- 1) The soil at this bridge site consists for the most part of very stiff to stiff silty clay glacial till which begins about 6 to 9 feet below ground surface and extends at least to a depth of 97 feet. It is overlain by medium dense fine sand which is wet below the 2 foot level. Bedrock was not proven but it is believed to be about 120 feet below the road surface.
- 2) Abutments and piers of the bridge can be founded at the top surface of the glacial till, at which level a safe net bearing value of 6500 p.s.f. can be utilized. Water will flow from the sand when excavating to this depth, but this should not cause any serious construction difficulty, and it will eventually stabilize with continued pumping from open sumps.
- 3) Because of the great depths of clay underlying this site, long term settlement of abutments and embankments should be anticipated. A total movement of the abutments in the order of 6 ins. has been estimated. Most of this movement should be complete after about 5 decades.

- (4) The approach embankments will be quite stable during and after construction.

WAT/lst
J 478
March 25, 1960.



W. Trow
William A. Trow (P. Eng.)

APPENDIXFIELD INVESTIGATION METHODS

Four borings were made at this site. Two at the south-west and north-east corners of the proposed bridge were taken to 65 and 97 feet respectively; the other two were terminated at 25 feet.

The holes were made by a continuous flight auger 5 inches in diameter; no casing was used. Samples were taken generally at 5 foot intervals of depth starting 3 feet below the surface. Both split-spoon and thin-walled shelly tube samples were recovered. In the former instance, the sampler was driven into the ground using an energy of 350 ft.lbs. per blow. The shelly tubes were pushed or levered into the ground in most instances. Field vane tests were attempted but in all cases the ground was too stiff. Cone penetration tests were made to a depth of 10 feet adjacent to each boring.

The elevation of each hole was obtained using the bench mark indicated in Dwg. 1 as reference.

TABLE NO. 1
SUMMARY OF LABORATORY & FIELD TEST DATA WP 89-59

Depth Ft.	Shear Strength psf and Unit Wt., γ , pcf				Natural Moisture % dry weight				Atterberg Limits		Penetration Res. Blows/ft.			
	H1	H2	H3	H4	H1	H2	H3	H4	L.L. H2 except where noted	P.L.	H1	H2	H3	H4
6						16.9	18.2		35.5	17.6	13	28	30	23
8														
10		3200			17.3	18.2	18.1	18.6			31	push	28	35
		$\gamma=133$												
12			3450					19.1	35.3 - H4-	17.0				26
			$\gamma=137$											
14					18.7	18.0	14.6		34.3	17.2	21	25	22	
16								19.8						21
20		2550		3150	20.4	19.7	18.7	19.4			24	push	31	34
		$\gamma=133$		$\gamma=134$										
24					18.7	19.8	20.0	20.0	34.6	16.1	26	24	24	19
30		2140				20.9						push		26
		$\gamma=132$												
34						20.4		22.1	37.7	18.2		19		22
40			2250					21.7						31
			$\gamma=133$											
44						19.4		19.7				35		23
54			1860			22.2		23.0	37.9	18.5		25		26
			$\gamma=130$											
64						22.5		18.1				33		52
74								13.2						55
84								15.6						33

PROJECT NO. J 478

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT County Road Underpass WP 69-50

LOCATION North of Ridgetown, Ont.

HOLE LOCATION See Dwg. 11

HOLE ELEVATION AND DATUM 561.3

BOREHOLE NO.

FIELD SUPERVISOR

DRILLER

PREP.

DRAWING NO. 2

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	STRENGTH AND PENETRATION RESISTANCE		
				Shear Str.	2000 P.S.F.	4000 P.S.F.
	Ground surface	661.3	0	10	20	30
	Medium dense brown fine sand; Gravel sizes up to 1/2 inch below 6 feet, some clay layers below 7 ft.					
	Stiff grey silty clay with gravel sides. (Glacial Till)	652.3	10			
	End of bore	637				

NOTES: 1) Boring by continuous flight
auger; hole uncased to full depth.
2) Sampler driven under energy
of 350 ft.lbs.per blow.
3) Cone test 6 feet north of
hole.

[illegible]

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT County Road Underpass WP 99-59

LOCATION North of Hidgetown, Ont.

HOLE LOCATION See Dwg. 1

HOLE ELEVATION AND DATUM 661.7

BOREHOLE NO. 2

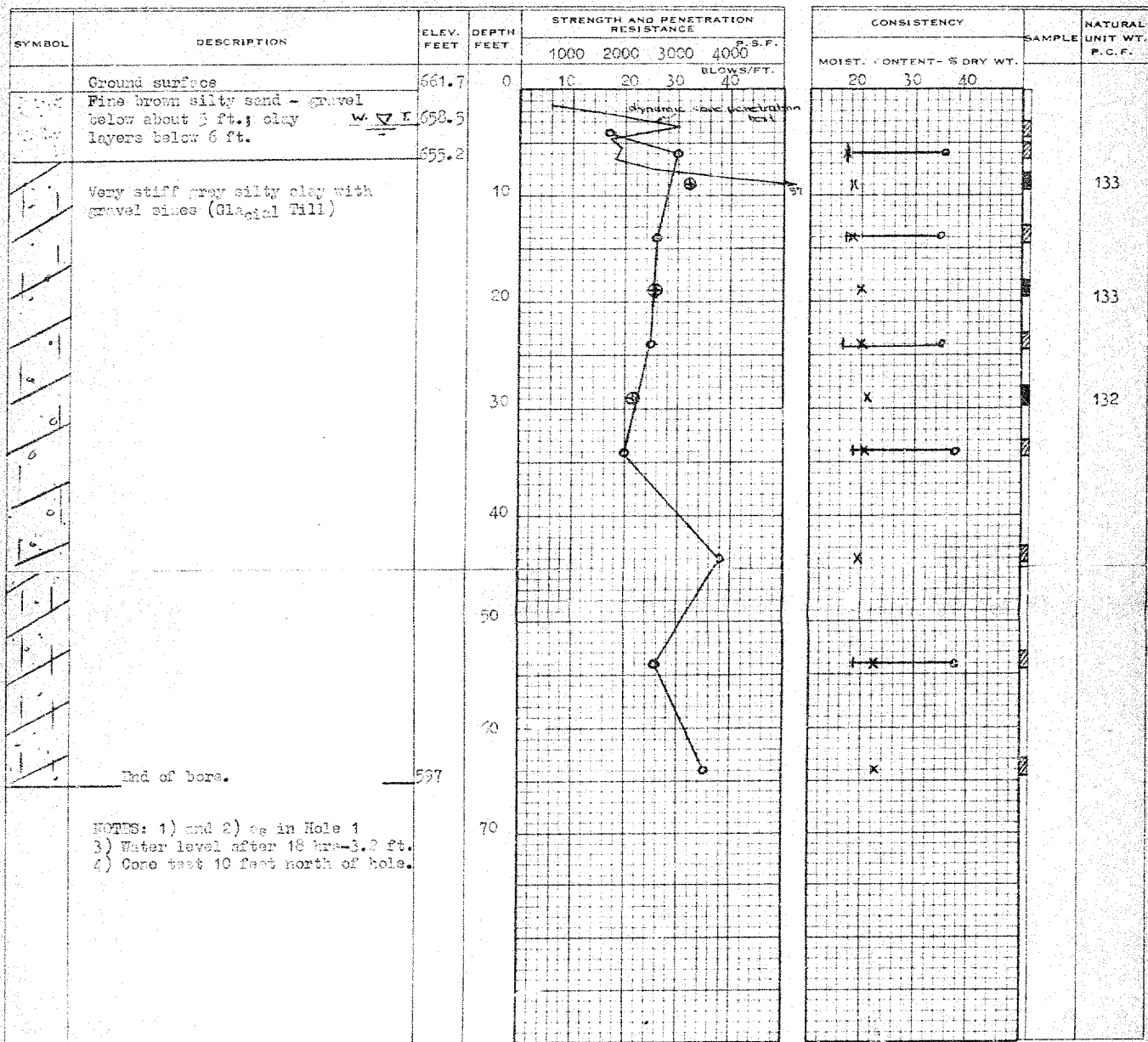
FIELD SUPERVISOR

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



PROJECT NO. J 478

DRAWING NO. 4

WILLIAM A. TROW & ASSOCIATES LTD.

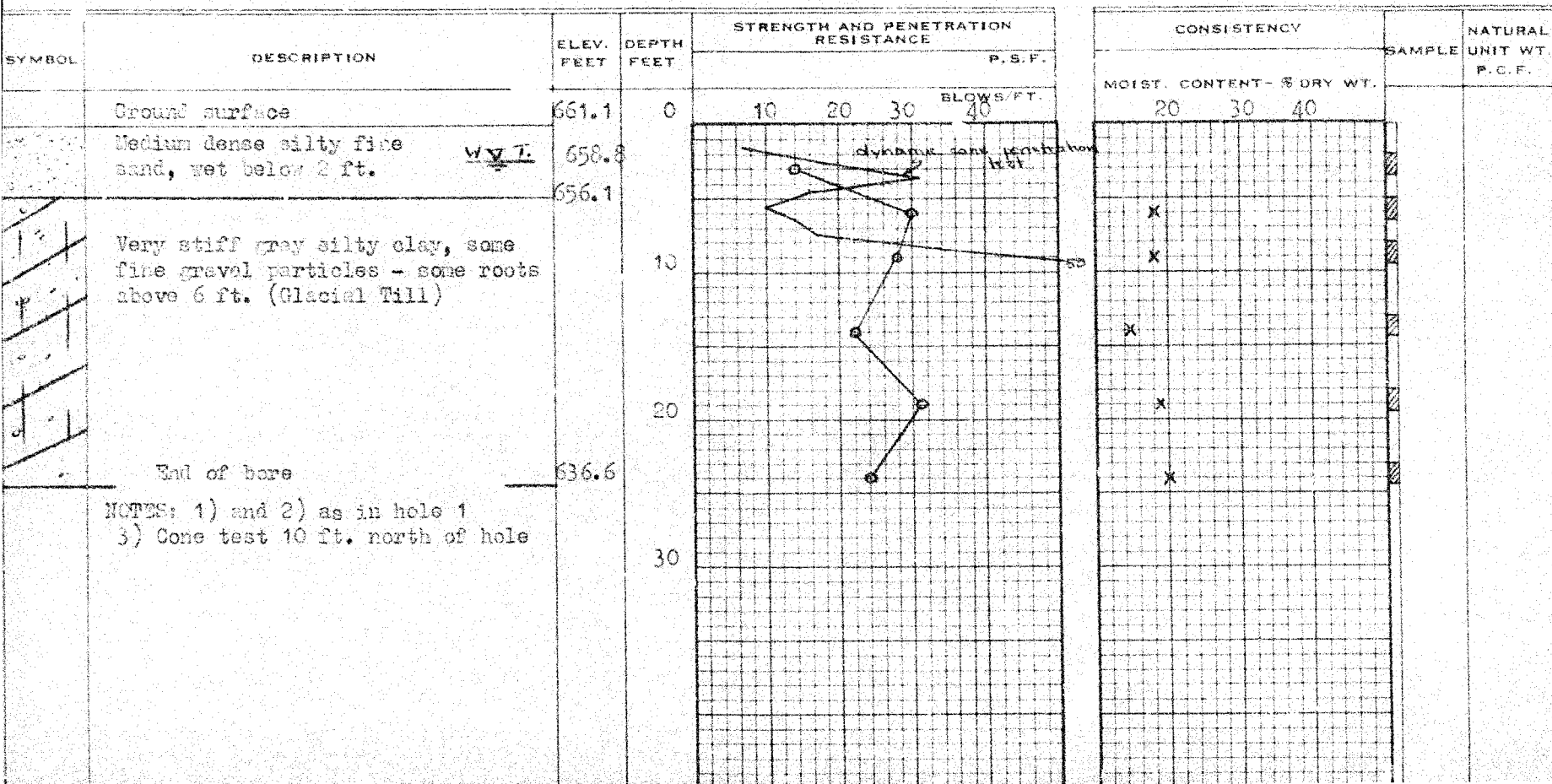
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT County Road Underpass WP 89-59
 LOCATION North of Ridgeway, Ont.
 HOLE LOCATION See Dwg. 1
 HOLE ELEVATION AND DATUM 661.1

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



WILLIAM A. TROW & ASSOCIATES LTD.

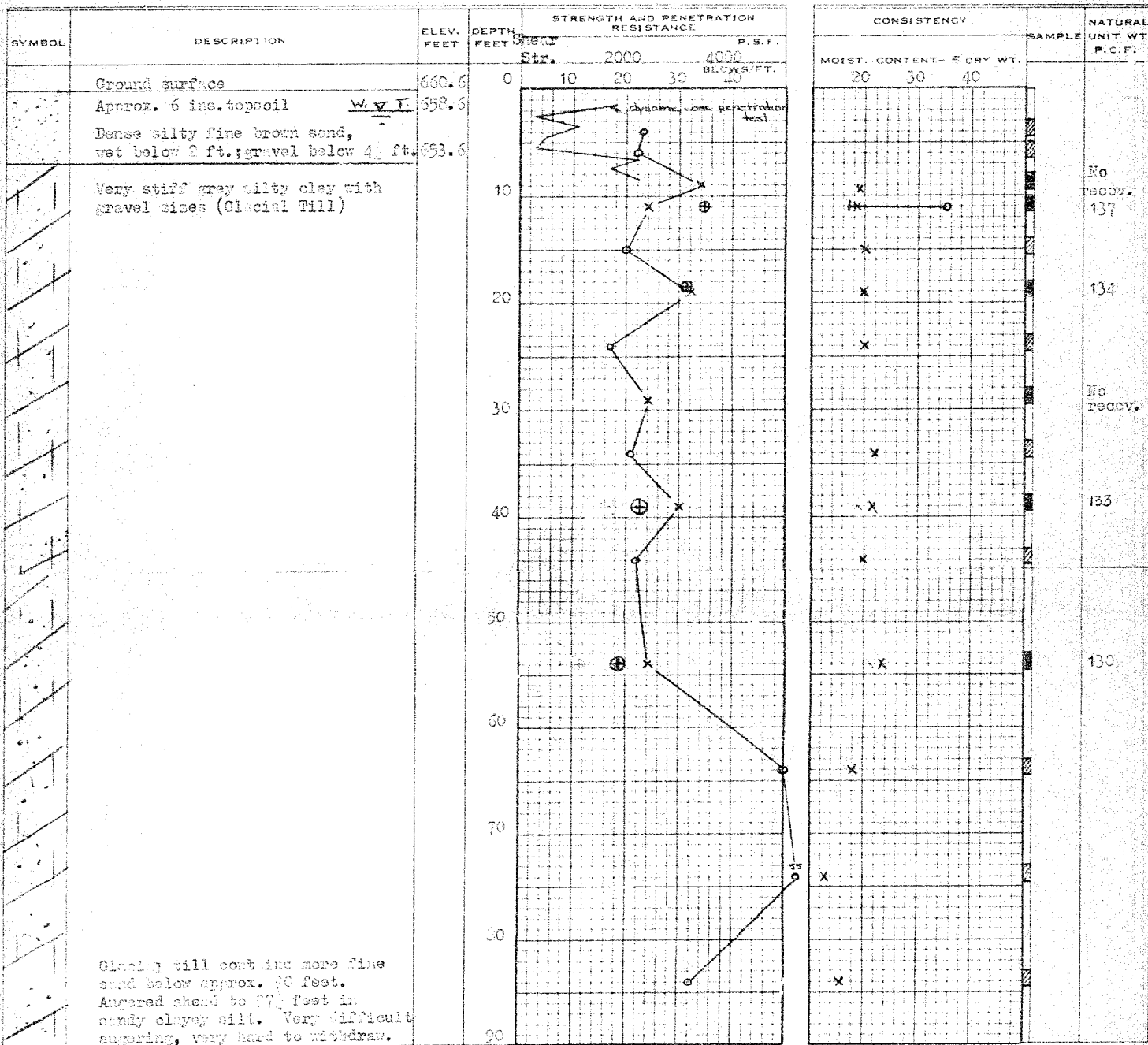
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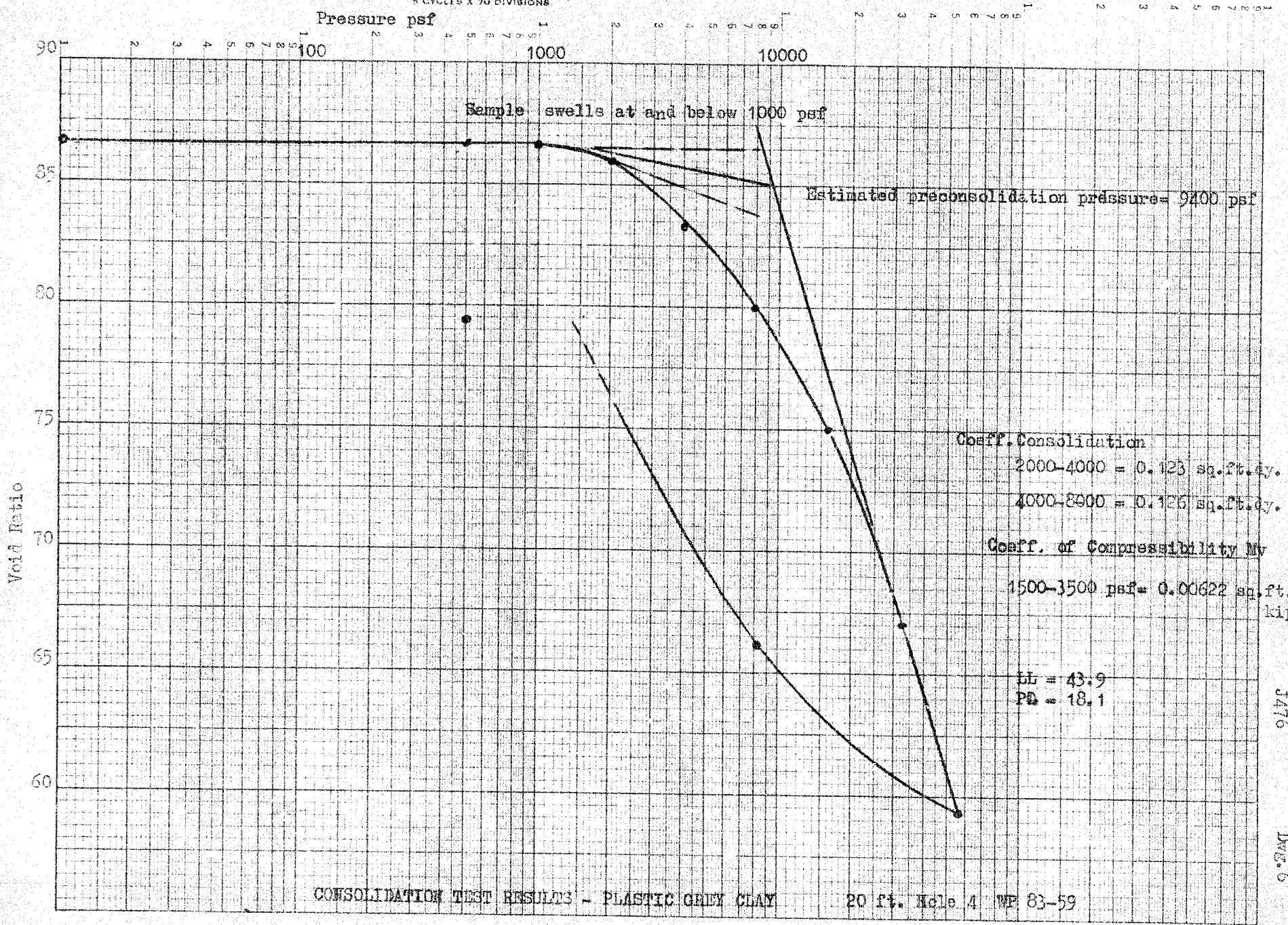
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 LOCATION: North of Ridgeway, Ont.
 HOLE LOCATION: See Dwg. 1
 HOLE ELEVATION AND DATUM: 660.6

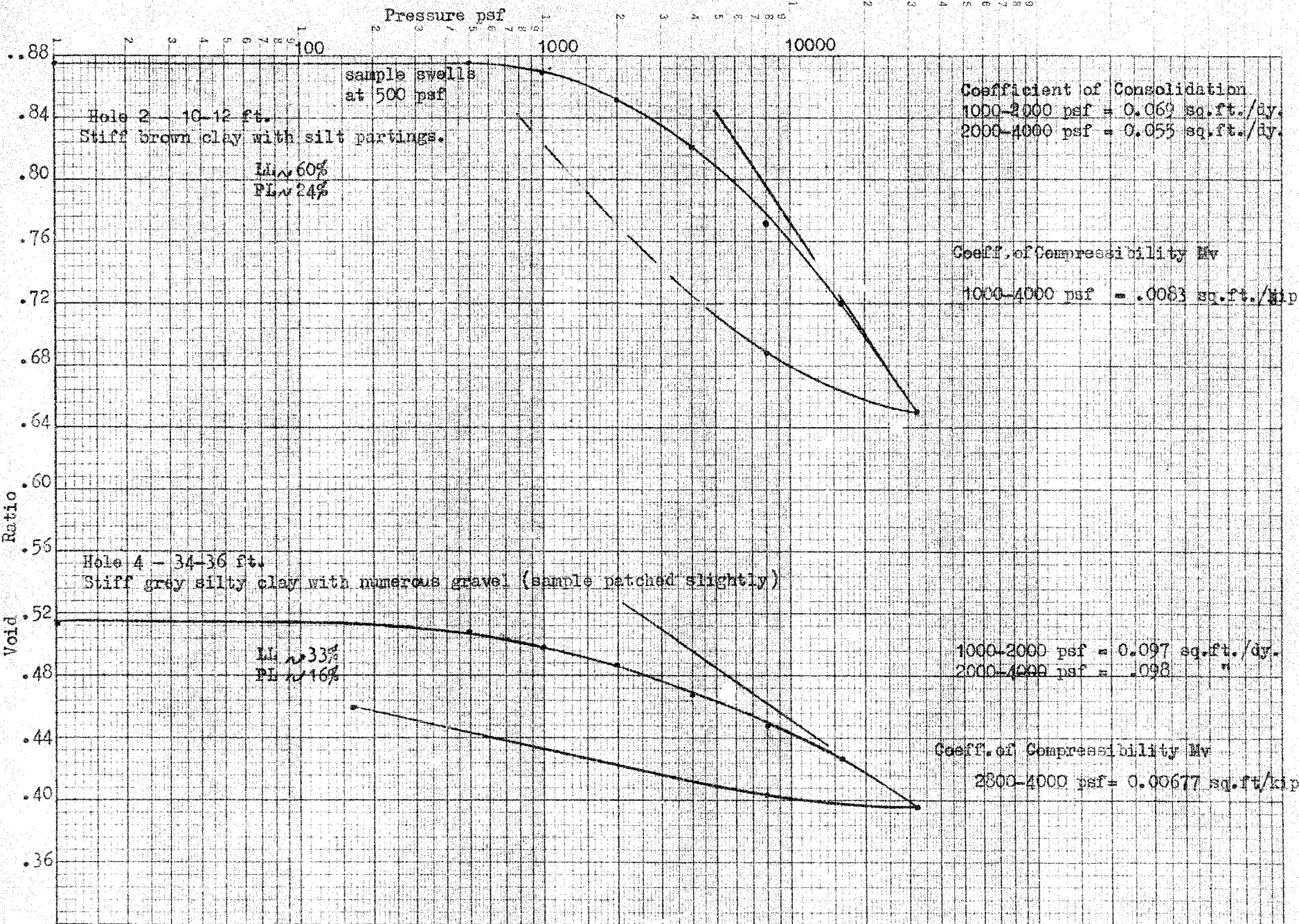
BOREHOLE NO. 4
 FIELD SUPERVISOR
 DRILLER
 PREP.

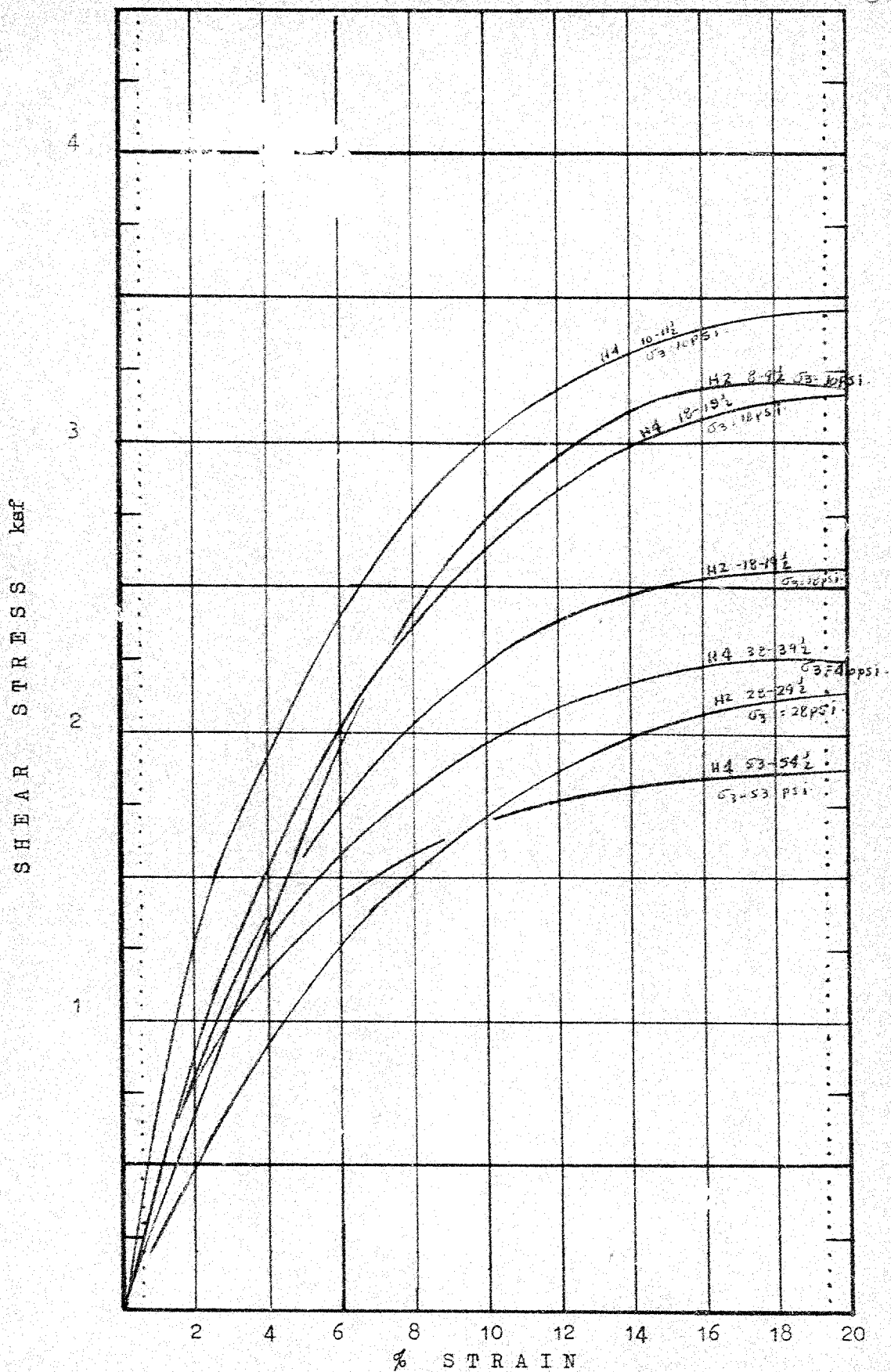
LEGEND

2" DIA. S. TUBE
 2" SHELBY LOG
 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









STRESS STRAIN CURVES UNDRAINED TRIAXIAL TESTS WP89-59

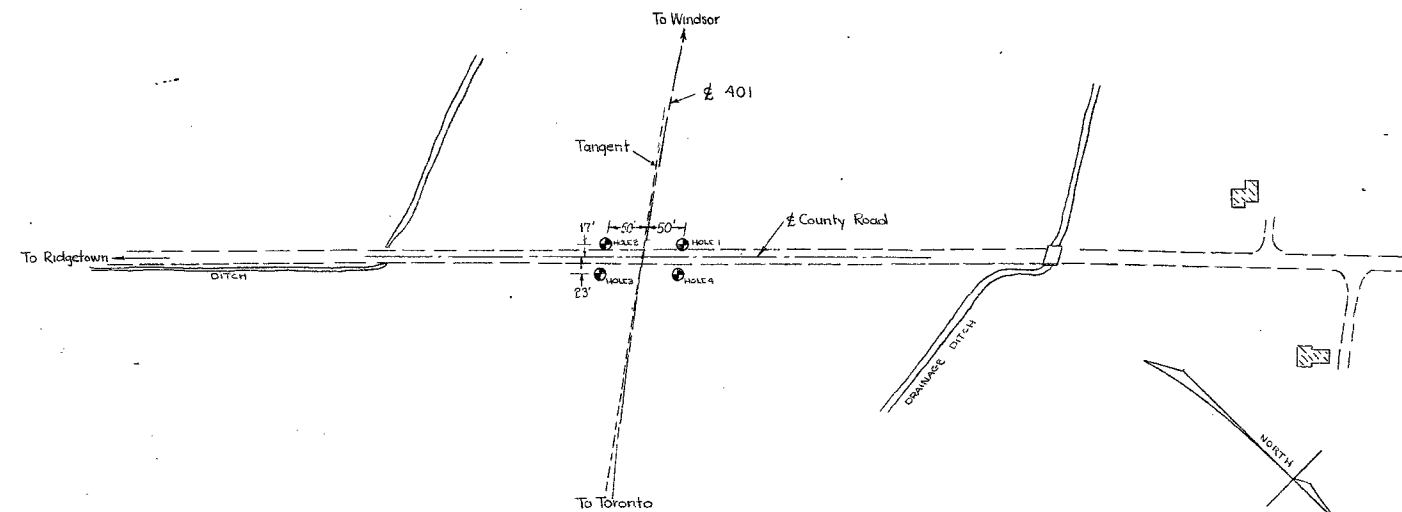
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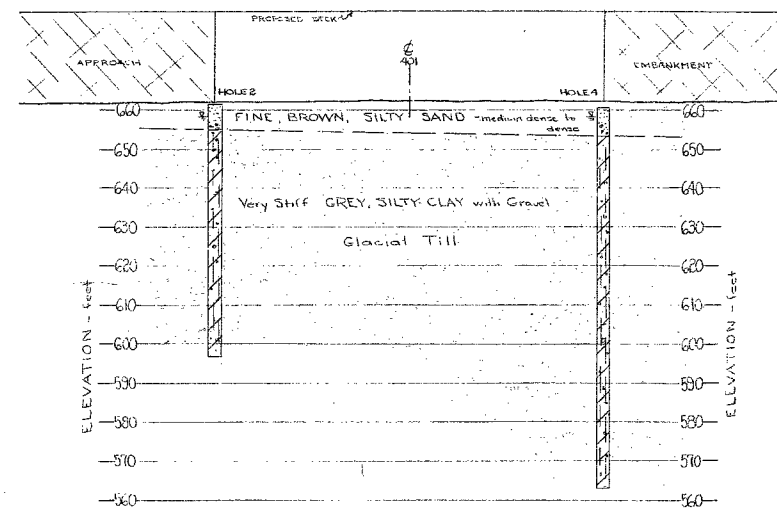
HWY.#401, UNDER-

PASS, CTY. RD.#16

RIDGETOWN



BOREHOLE LOCATION PLAN SCALE 1"=100'



B.M. Nail & Washer in NW Root of
2 1/4" Maple 24 ft. Left of Sta. 351+45
664.31

PROPOSED UNDERPASS
W.P. 89-59
FOUNDATION INVESTIGATION by
Wm. A. TROW & ASSOCIATES, Ltd.

ESTIMATED SUBSOIL STRATIGRAPHY SCALE 1"=20'