

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 40JB-38

W.P. No. 83-59

CONT. No. _____

W. O. No. _____

STR. SITE No. _____

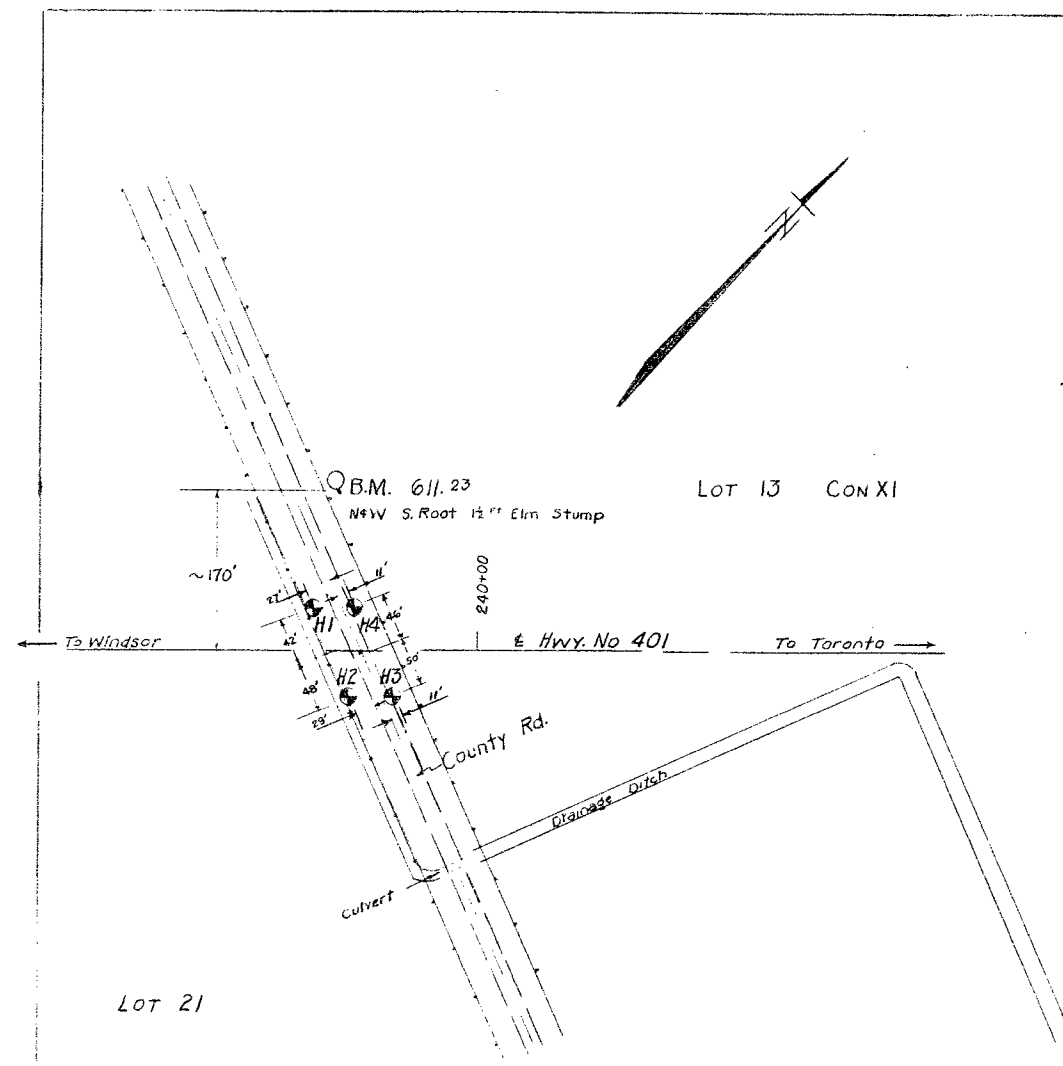
HWY. No. 401 DIST. *1

LOCATION CO. RD. U'PASS,
N. of BLENHEIM,

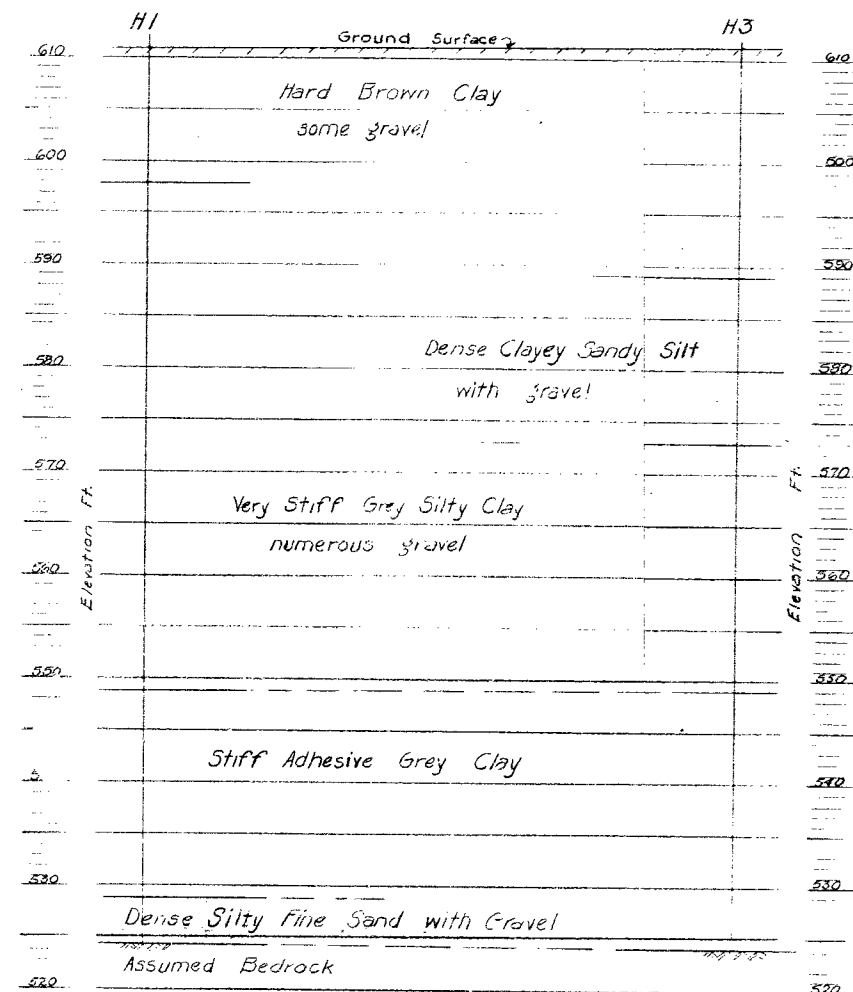
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. ONE

REMARKS: _____

G.I.-30 SEPT. 1976



BOREHOLE LOCATION PLAN 1"=100'



ESTIMATED STRATIGRAPHY 1"=20'

PROPOSED COUNTY ROAD UNDERPASS

NORTH OF BLLENHEIM ONT.
William A. Trow & Assoc. Ltd.

WP 83-59
Mar. 22 1960

40J8-38
GEOCRE No.

BA 1030

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

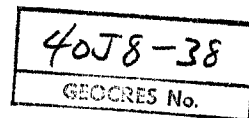
W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

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

Project: J 476

March 18, 1960.

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



Attention: Mr. L. G. Soderman

Foundation Investigation
Proposed County Road Underpass, Hwy. 401,
North of Blenheim, Ont. WP83-59

Dear Sirs:

The enclosed report describes the soil conditions existing at the bridge site noted above.

We have performed two deep borings to bedrock and two shallow holes to a depth of about 25 feet at this site. The two shallow borings were made instead of cone penetration tests because the soil consisted of very stiff clay. Penetration tests would serve no purpose in this material.

The soil at this location consists for the most part of very stiff to hard clay till. Bedrock lies about 87 feet below the surface. Although the soil is quite stiff, settlement of the abutments will take place because of the weight of the adjacent fill. The probable long term settlement is estimated to be in the order of 4 to 5 inches. No embankment stability problem exists.

We hope that the contents of this report provide you with sufficient information to proceed with the foundation design of this structure. If you wish us to consider design proposals other than those assumed in this report, please do not hesitate to contact us.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

WAT/lt
Encl.

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

FOUNDATION INVESTIGATION
PROPOSED COUNTY ROAD UNDERPASS HWY. 401,
NORTH OF BLENHEIM, ONT. WP83-59

Project: J476 March 18, 1960.
William A. Trow & Associates Ltd.

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FOUNDATION INVESTIGATION
PROPOSED COUNTY ROAD UNDERPASS, HWY. #401,
NORTH OF BLENHEIM, ONT., WP 83-59

The following report describes the soil conditions encountered at the county road intersection noted above. The safe bearing value of this soil and its stability under embankment loads has been discussed. Details concerning investigation methods are given in the Appendix.

Description of Site

In the vicinity of this crossing the land is relatively flat with changes in elevation only of the order of 3 to 5 feet. The higher portions of the ground are covered with a thin veneer of fine sand which provided material for minor dust storms during the investigation period. The surrounding land is sown to corn for the most part although some areas are still tree-covered.

A shallow drainage ditch bounds the east side of the gravel road and it was dry. A deeper ditch, paralleling the road just beyond the west boundary fence, contained about one foot of water. This ditch passes through a box culvert about 200 feet to the south and then moves east at right angles to the county road.

Shallow commercial gravel pits were noted adjacent to a paved county road about 3 miles to the north-east of this site. Some gravel outcropping was noted also approximately 2 miles to the south.

Subsoil Description

The soil at this location was found to exist in a very stiff or dense condition for almost its entire depth. The description and record of physical properties of the soil at each test location are presented in Dwg. 2 to 5, and the summary of laboratory and field test data is given in Table 1. In order to assist in the appraisal of subsoil conditions these borehole logs have been condensed into the stratigraphical profile shown as part of Dwg. 1.

In referring to this drawing, it can be seen that the soil consists predominantly of glacial drift material down to bedrock depth located at Elev. 524.4, or about 87 feet below present ground level. The soil in the first 15 to 20 feet is somewhat more plastic than the material below and it contains fewer gravel particles. It has been weathered to a brown colour for the first approximately 14 feet.

Below these depths of 15 to 20 feet, the gravel content of the clay increases and in Holes 2, 3 and 4, the soil becomes a slightly cohesive very dense sandy silt with gravel for several feet. In general, the moisture content of these glacial deposits are at or close to the plastic limit and the shear strength is equal to or well in excess of 2450 p.s.f.

A more or less stratified reddish grey very adhesive clay begins about 62 feet below ground level and this extends to a depth of approximately 82 feet. It exists in a stiff condition with a shear strength in the order of 1400 p.s.f. and a moisture content midway between the liquid and plastic limit.

A dense dark grey silty fine sand with shale gravel overlies assumed bedrock at Elev. 524.4 feet. Although bedrock was not proven, it could not be penetrated either with the augers or by means of the split spoon. It was found at exactly the same elevation in two holes located about 125 feet apart at diagonally opposite corners of the proposed bridge structure.

Discussion of Foundation Requirements

In view of the generally excellent condition of the foundation soil at this site, no difficulties are anticipated either during construction or after the structure is put into service.

Footings for the bridge can be placed at shallow depth of about 4 feet below present ground surface; at and below this level, the safe bearing value is equal to 8000 p.s.f. Although the ground water table probably lies about 4 feet below the surface, it will take several days to establish itself at this level. Therefore footing excavations should remain dry during the construction period.

If no embankment pressures were involved in this construction, the settlement to be anticipated with a bearing value of 8000 p.s.f. should be less than one inch and this movement should take place within a few months after full load is applied. However, since approach fill reaching a height in the order of 20 feet is required for this overpass, and since the pressures from this fill will compress most of the soil above bedrock, considerably more settlement than one inch will be experienced at the abutment locations, even though the soil is in a very stiff to hard condition.

In order to obtain an estimate of the amount of settlement to be anticipated, reference is made to the results of a consolidation test performed on a sample of the more plastic clay found at a depth of 20 feet in Hole 4. This test information is shown in Dwg. 6. The significant fact indicated by this test is that the soil exists in a highly overconsolidated state with an estimated preconsolidation pressure in the order of 9400 p.s.f., and, as a consequence, any pressures less than this value will merely produce recompression of this material. In this particular sample, the clay exhibited a tendency to swell at pressures of 1000 p.s.f. and below.

The coefficient of compressibility, M_v , of this material is given by the expression -

$$M_v = \frac{e_0 - e}{\Delta p(1+e_0)}$$

where e_0 and e are the void ratios of the clay at the present time and after complete consolidation under the weight of the bridge structure.

Δp is the increase in pressure at a depth of 20 feet, resulting from the bridge and embankment loads.

Computations of the value of M_v for the assumed pressure conditions at this depth are indicated in the Appendix. The result is believed to be unduly conservative not only because consolidation tests tend to underestimate this measurement, but also because the assumption of bridge loading is somewhat severe. Included also in the Appendix is the estimated settlement of each abutment. A value of 10 inches has been indicated which, again, is believed to be quite conservative. Because of the very low permeability and great depth of the soil, this movement will continue at a decreasing rate for an estimated period of 30 years. The estimated settlement of the fill about 50 feet back of the abutment has been computed to be about 12 inches.

Although the foregoing values are believed to represent severe overestimates of the movement actually to be experienced, there is little doubt that some consolidation of the abutments must take place. This fact becomes important if the structure is to incorporate a centre pier. Settlement of this unit will be of the order of one inch or less.

In view of the very high shear strength of the natural soil, the factor of safety as regards embankment stability will be high. Embankments can be placed at any convenient stage in the construction program. No significant reduction in settlement will result by placing the embankment fill before the bridge is built unless this part of the program precedes the bridge construction by several years. The full shear strength of the soil should be available to resist sliding of the base of the abutments after the embankment fill load is applied.

Summary of Comments and Conclusions

The comments of the foregoing sections can be summarized briefly as follows:

- 1) The soil at this underpass site consists for the most part of very stiff to hard clay glacial till. Bedrock lies about 87 feet below ground surface.

- 2) Footings for the bridge structure can be placed at a depth of 4 feet, at which level the recommended safe bearing value is 8000 p.s.f.
- 3) Because of the weight of embankment fill, the abutments will settle more than the interior pier. On the basis of very conservative assumptions, a long term settlement of the order of 10 inches has been computed. This movement should continue over a period of about 36 years. More probably, the settlement will be of the order of 4 to 5 inches during this period. A movement of the centre pier of one inch is estimated.
- 4) The soil is quite strong enough to support the weight of the embankment fill safely.

WAT/lt
March 18, 1960.
J 476



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

SUMMARY OF LABORATORY & FIELD TEST DATA

[illegible]

APPENDIXSETTLEMENT COMPUTATIONSAbutments:

Assume footings at depth of 4 feet, exerting an average net bearing pressure of 4000 p.s.f.; assume footings 40 feet long and 10 feet wide; assume load distribution into soil at 30° to vertical.

Consolidation characteristics for the soil as indicated in Dwg. 6.

Assume approach embankments 20 feet high, top width = 40 ft.; side slopes of $1\frac{3}{4}:1$; fill and soil weight = 125 p.c.f. above water table; soil weight below water table = 70 p.c.f.; water table at depth of 4 feet.

Distribution of fill pressure at abutment computed using Newmark diagram.

Computation of Coefficient of Compressibility M_v :

At depth of 20 feet, present in situ pressure approximately 1600 p.s.f.

Corresponding void ratio from Dwg. 6 = 0.864.

Pressure due to footing at 30° load spread = 940 p.s.f.

Pressure due to 20 feet of fill = 1060 p.s.f.

Total Increase in pressure = 2000 p.s.f.

Final pressure at 20 feet = 3600 p.s.f.

Corresponding void ratio = 0.841

Change in void ratio = .023.

$$M_v = \frac{.023}{2.0 \times 1.864} = 0.00622 \text{ sq.ft./kip.}$$

Assume this compressibility value effective for the full depth to bedrock. This assumption is probably conservative for the leaner clay till below 20 feet and may underestimate the compressibility of the deeper reddish grey clay found below 62 feet.

Computation of Settlement of Abutments

Depth below footing	Pressure due to Footings	Pressure due to fill	Total Pressure p	$S = H M_v p$ $= 120 \times .00622 p$
5	2260	1490	3750	2.80
15	1000	1150	2150	1.68
25	560	1000	1560	1.17
35	410	990	1400	1.05
45	300	950	1250	0.93
55	210	930	1140	0.85
65	160	910	1070	0.80
75	130	900	1030	0.77

10.0 inches approx.

Duration of Settlement:

Coefficient of consolidation $C_v = 0.123$ sq.ft.dy.

Time for 50% consolidation, or 5 inches settlement,
assuming drainage to surface and to bedrock =

$$\frac{0.197 \times 40^2}{0.123 \times 365} = 7 \text{ years.}$$

Time for 90% consolidation, or 9 inches settlement =

$$\frac{.848 \times 40^2}{0.123 \times 365} = 30 \text{ yrs.}$$

Settlement of Fill approximately 50 ft. from abutments:

Depth Below Ground	Pressure due to fill	$S = H M_v p$ $= 0.747 p$
5	2500	1.87
15	2350	1.75
25	2100	1.57
35	2000	1.49
45	1950	1.46
55	1900	1.42
65	1900	1.42
75	1850	<u>1.38</u>

12 inches approx.

FIELD INVESTIGATION METHODS

Four borings were made at this site. Two at the north-west and south-east corners of the proposed bridge were taken to bedrock; the other two were terminated at 28 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 most cases the ground was too stiff.

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

WILLIAM A. TROW & ASSOCIATES LTD.

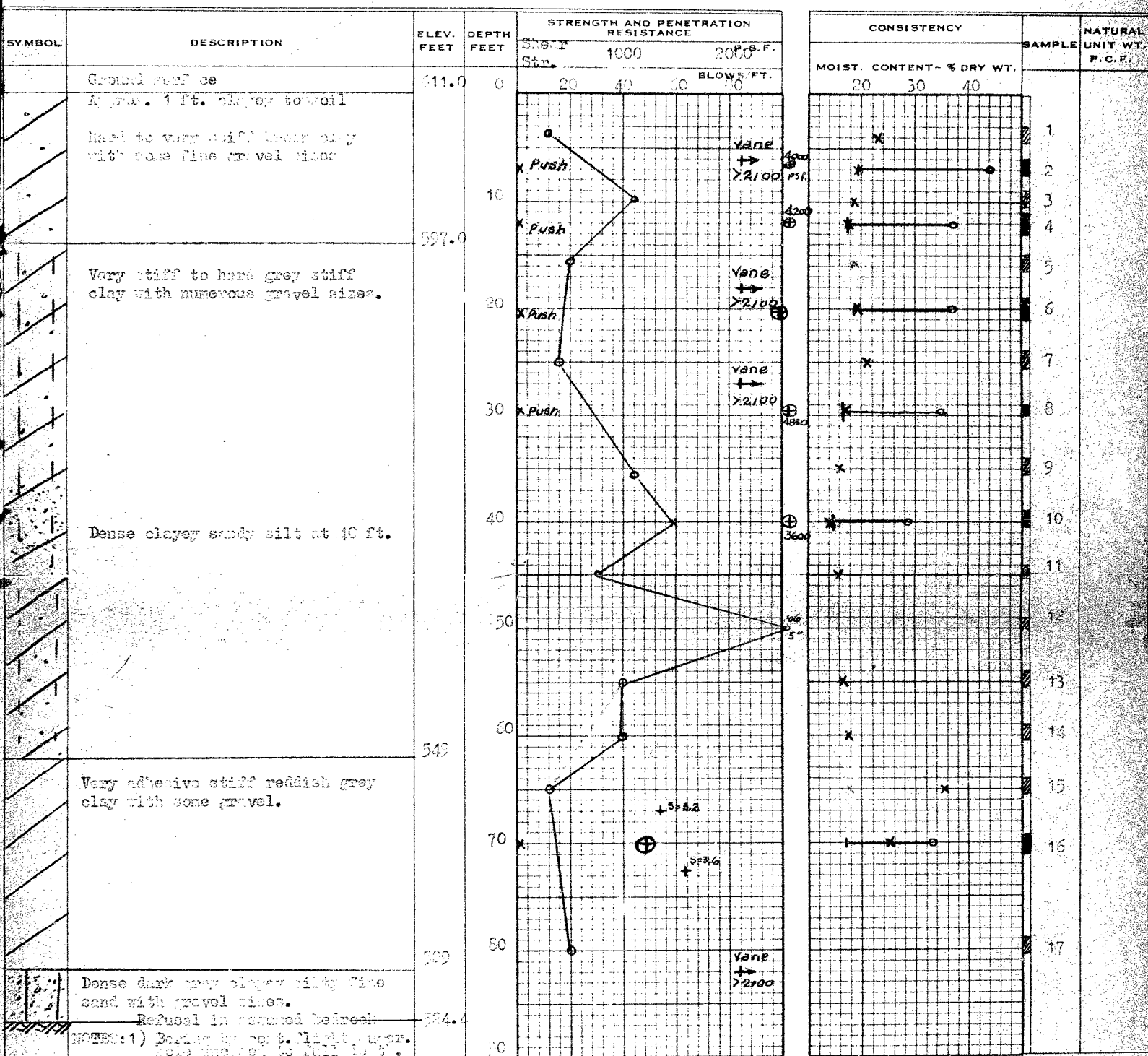
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT: Prosser County, Ga. - Columbus BP 53-55
 LOCATION: Hwy. 471 - North of Milledgeville
 HOLE LOCATION: Sec D .1
 HOLE ELEVATION AND DATUM: 511.5 BM - see Map. 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



PROJECT NO. J 476

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT: Preseal County, W. Virginia SP 83-70

LOCATION: Hwy. 401 - N. of Blain

HOLE LOCATION: See Dwg. 1

HOLE ELEVATION AND DATUM: 511.0 See Dwg. 1

BOREHOLE NO. 2

FIELD SUPERVISOR

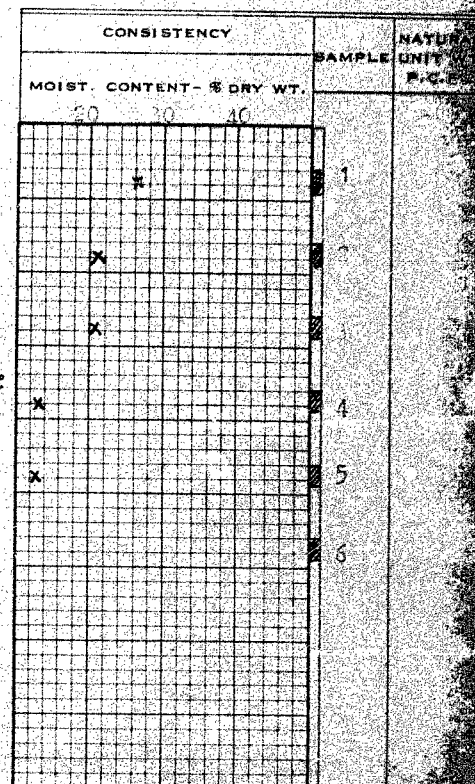
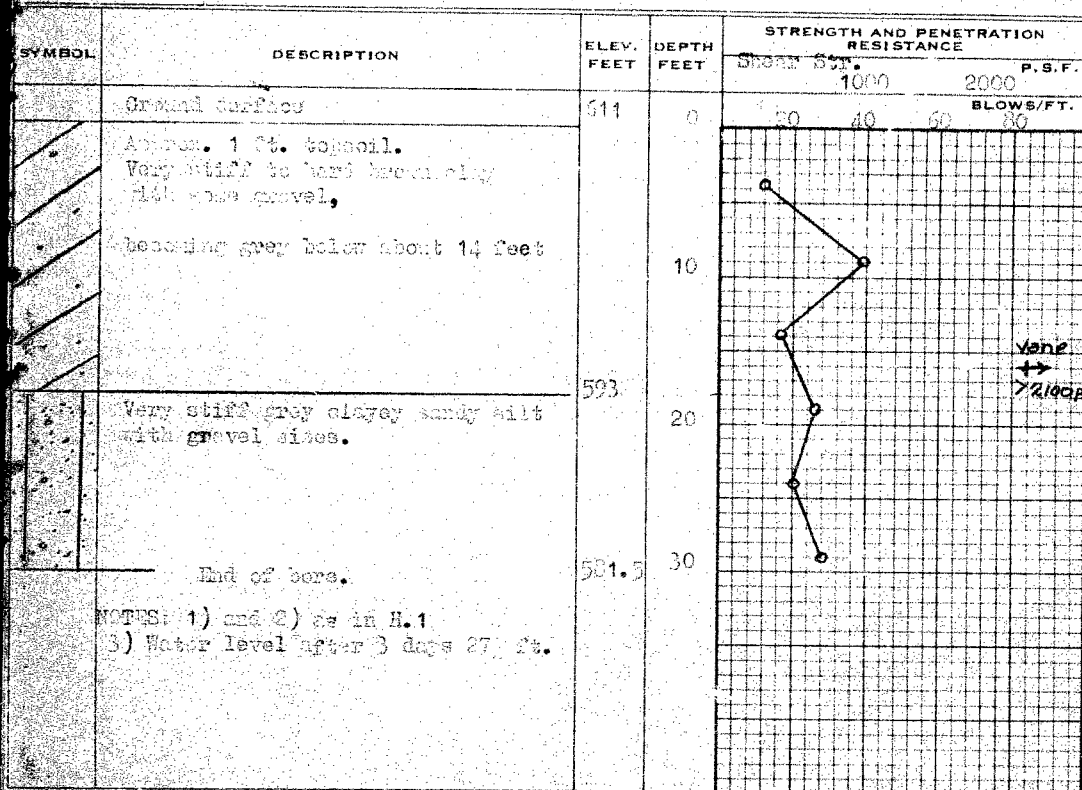
DRILLER

PREP

DRAWING NO.

LEGEND

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



WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

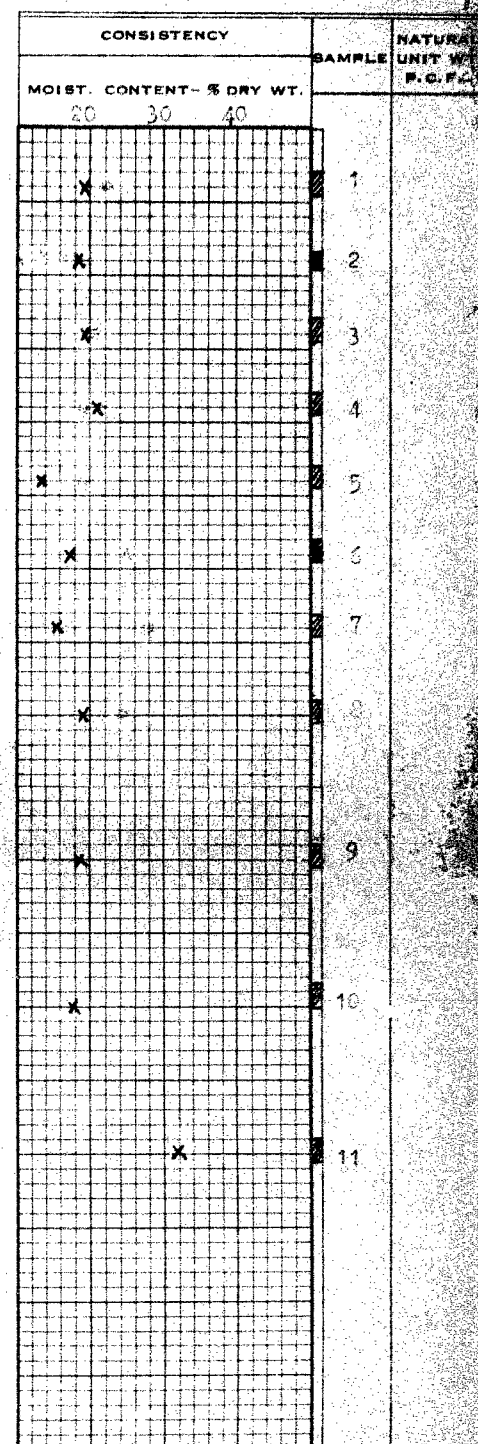
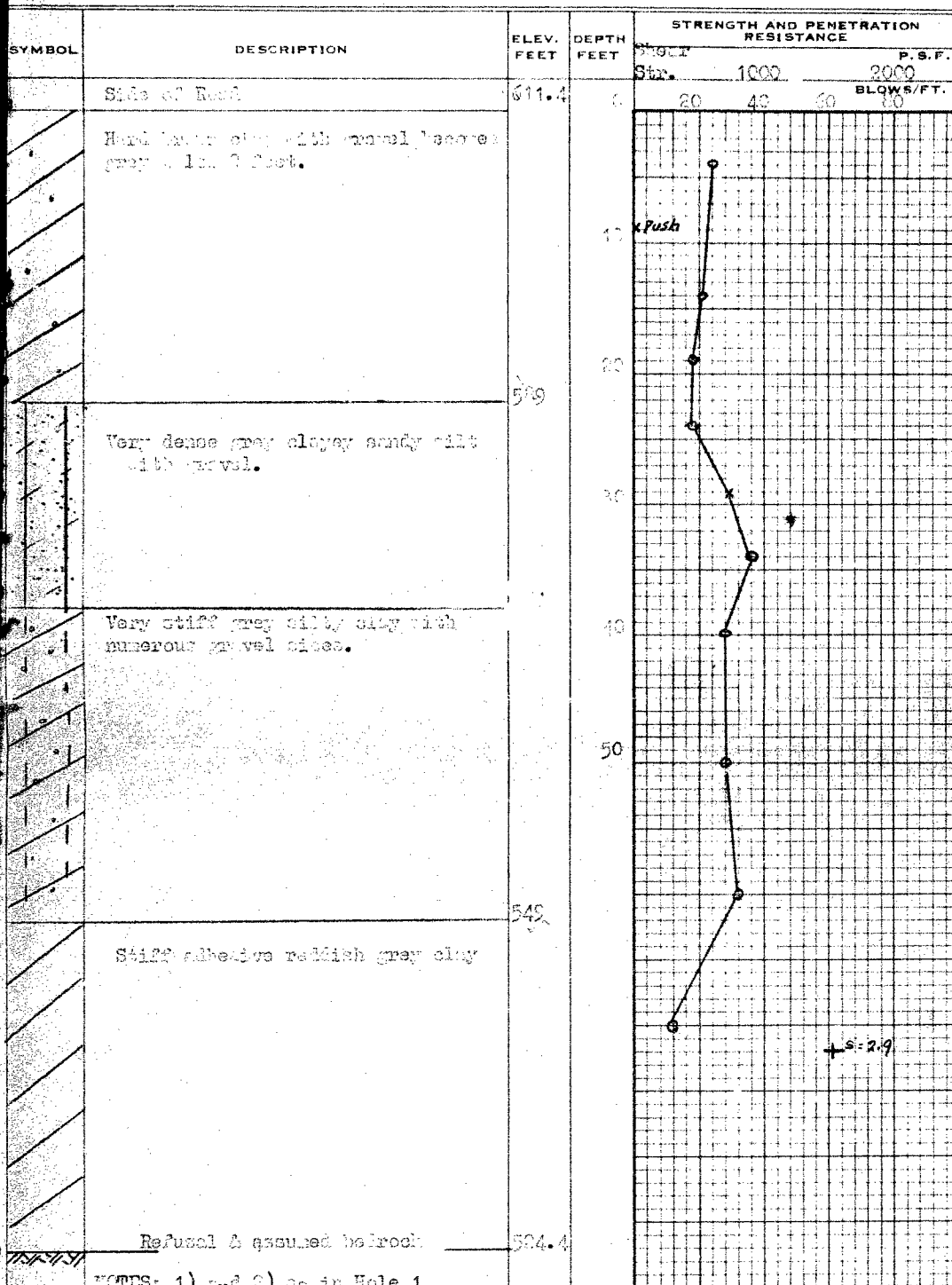
DRAWING NO. 4

LEGEND

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

PROJECT Proposed Concrete Sl. Interpass WP-3-50
 LOCATION 101- South of Blenheim
 HOLE LOCATION Site No. 1
 HOLE ELEVATION AND DATUM 611.4 25 - 200 Dag. 1

BOREHOLE NO. 3
 FIELD SUPERVISOR
 DRILLER
 PREP.



J 453

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

5.

1. The first step is to identify the problem.

PROJECT

LOCATION 7-6 101 7 6 1-1050

HOLE LOCATION

HOLE ELEVATION AND DATUM

BOREHOLE NO.

FIELD SUPERVISOR

DRILLER _____

PREP.

DRAWING NO.

LEGEND

2nd DIA. SPLIT TUBE

2¹¹ SHELBY TUBE

2¹¹ SPLIT TUBE

2¹¹ DIA. CONE

CASING

211 SHELBY

1/2 UNCONFINED COMPRESSION (Qu)

VANE TEST [C] AND SENSITIVITY [S]

NATURAL MOISTURE AND

LIQUIDITY INDEX

LIQUID LIMIT

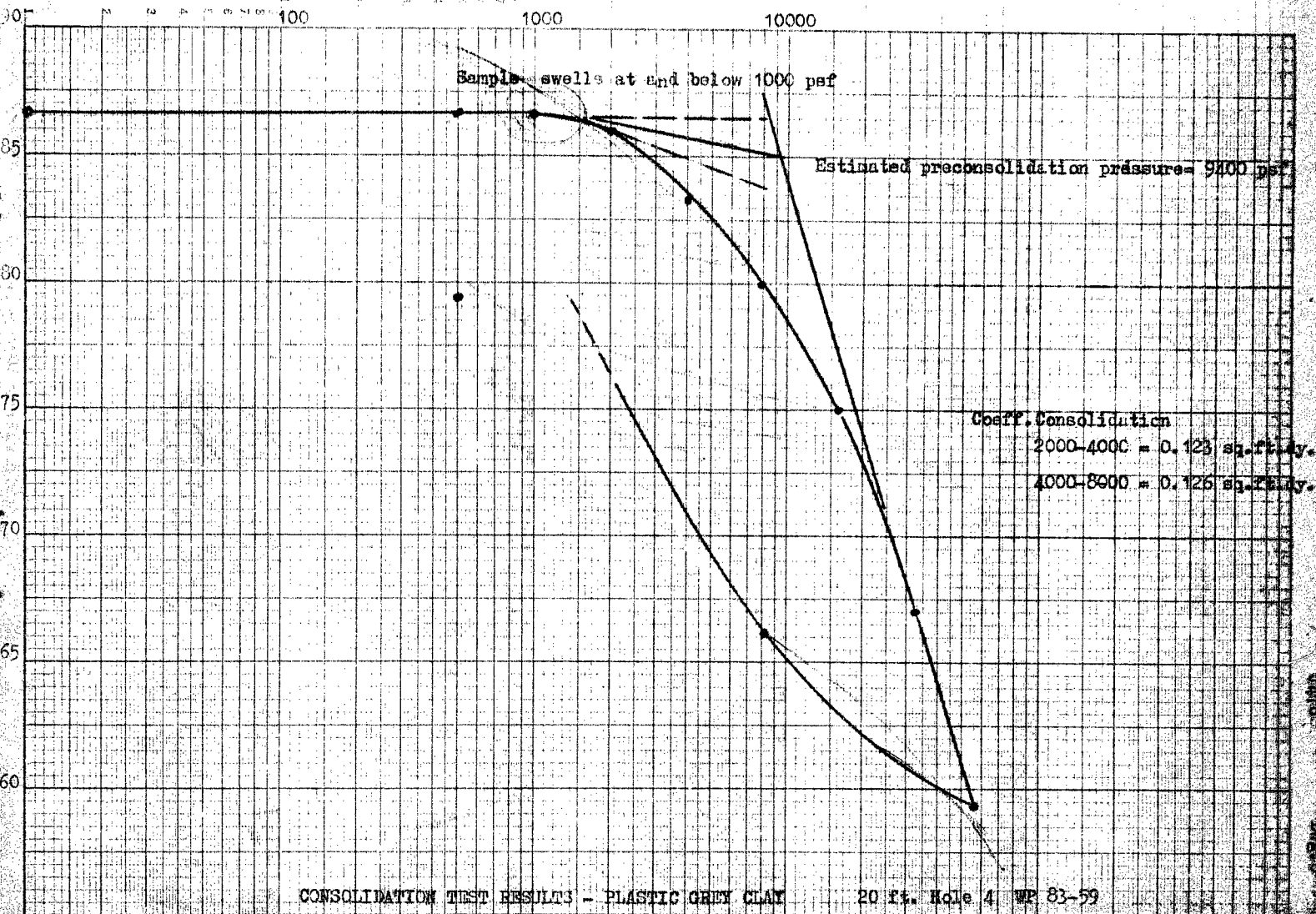
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				STRENGTH P.S.F.	100'S	200'S	BLOWS/FT.
	Top of Bed	111.2	0				
	Bed of sand	107.2	4				
	Very loose to medium sand, with some gravel.		10				
	Sand becomes very fine below below about 10 feet - more gravel at this level		20				
	Very loose to very fine sand with gravel.		30				
	End of bore.	104.0	36				

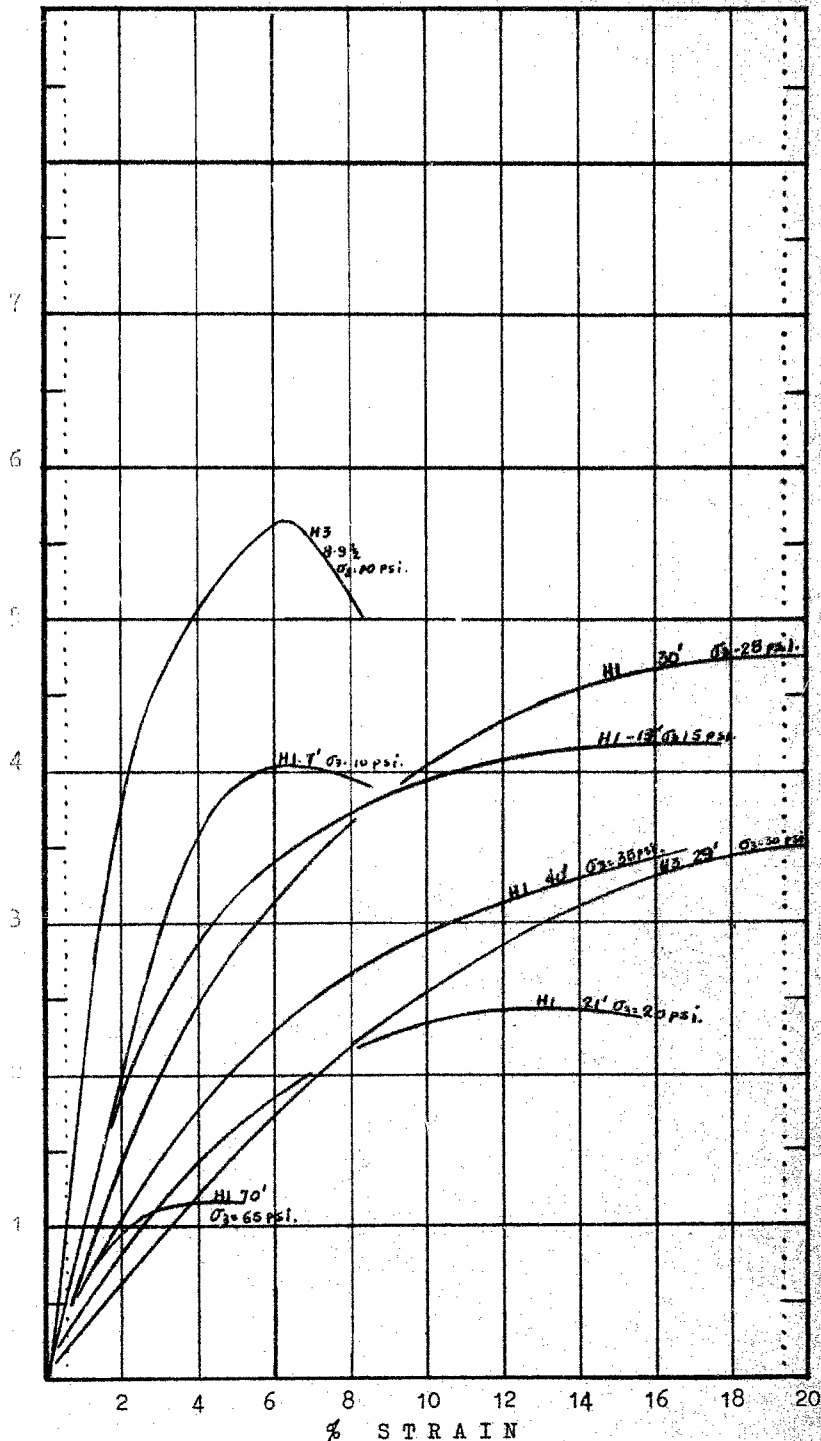
NOTES: 1) and 2) go in Hole 1.

CONSISTENCY		SAMPLE	NATURAL UNIT WT. P.C.P.
MOIST. CONTENT - % DRY WT.			
20	30	40	
		1	
		2	
		3	
		4	
		5	
		6	1st 5" tube
		7	

Pressure psf

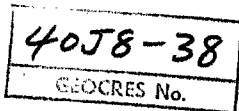


SHEAR STRESS ksf



STRESS STRAIN CURVES - UNDRAINED TRIAXIAL TEST RESULTS

WILLIAM A. TROW AND ASSOCIATES HOLES 1 and 3 WP 23-59



DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

D.M.O.
TORONTO
RECEIVED
JAN 1967
BRIDGE
OFFICE

To: Mr. C. S. Grebski,
Bridge Design Engineer,
Bridge Division.

From: Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Attention: Mr. C. Bassi

DATE: January 5, 1967

OUR FILE REF.

IN REPLY TO:

SUBJECT: Re: (1) W.P. 83-59 - Centreline Rd. Underpass, Hwy. #401,
District #1 (Chatham).
(2) W.P. 81-59 - Raleigh Twp. Bridge #13, Hwy. #401,
District #1 (Chatham).

As requested by you, we have reviewed the subsoil conditions at the above mentioned proposed structure sites with regard to the possibility of constructing the bridges with perched abutments. Our recommendations are as follows:

(1) W.P. 83-59 -

Piers may be supported on spread footings founded at or below el. 607.0, utilizing a safe pressure of up to 4.0 t.s.f. Abutments may be constructed within the approach fills and be supported on 12-3/4-inch O.D. steel tube piles driven to (but not below) el. 602. A design capacity of 30 tons/pile may be assumed.

(2) W.P. 81-59 -

Piers may be supported on spread footings founded at or below el. 579.0. A safe pressure of 3.0 t.s.f. may be assumed for design purposes. Abutments may be constructed within the approach fills in which event they may be supported on 12-3/4-inch O.D. steel tube piles driven to (but not below) el. 575.0. A safe capacity of 25 tons/pile may be assumed for design purposes.

For both structures it is recommended that granular type fill be placed up to the level of the underside of the footings at the abutment locations. This fill need only be placed over an area about twice as large as the area covered by the footings and should be well compacted.

Difficulties have arisen in the past when tube piles have been driven through compacted fill. It is believed that most of these difficulties have been due to the presence of cobbles or small boulders. We, therefore, strongly recommend that all grain sizes larger than 3 inches, be screened out prior to placing the above

cont'd. /2 ...

Mr. C. S. Grebski,
Bridge Division -
Attn: Mr. C. Bassi.

- 2 -

January 5, 1967

mentioned granular fill.

It is important that the piles be driven no deeper than the elevations recommended above, since the strength of the subsoil decreases considerably with depth.

If the above recommendations are followed, it is believed that differential settlements between the abutment and pier footings will be of a negligible order.

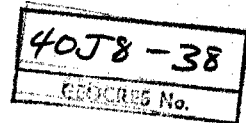
If you have any further queries concerning these projects, please contact this Office.

KGS/MdeF

cc: Messrs. A. P. Watt
J. Keen
Foundations Office
Gen. Files

K. G. Selby
K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

MEMORANDUM



To: Mr. C. S. Grebski
Bridge Design Engineer
Bridge Office
D O W N S V I E

FROM: A. P. Watt

DATE: February 17, 1967

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 83-59, Bridge Site 13-268,
Centreline Road Underpass,
9.1 miles west of Hwy. 21,
Highway 401,
District 1, Chatham.

D.H.O.
TORONTO
RECEIVED
FEB 20 1967
BRIDGE
OFFICE

Please be advised that the parties interested in the preliminary plan D-6129-P1 for the above noted structure have been contacted.

Attached please find correspondence from Road Design, South Western Region, the Construction Engineer, Chatham District and the Regional Traffic Engineer. The Bell Telephone Company of Canada through Mr. Murray Williams have indicated that they do not require any conduit on this bridge and a letter confirming this will be sent at a future date.

Please note that $3\frac{1}{4}$ " of asphalt will be approaching the bridge therefore the note reading "finished crown of pavement = 0.75' above profile grade" should be changed to "finished crown of pavement = 0.27' above revised profile grade". The profile grade P.I. should also be changed to read EL. 643.23.

If in the preparations of the final bridge drawings, the comments in the above correspondence have been considered, then the preliminary plan D-6129-P1 is satisfactory to Regional Bridge Planning.

A handwritten signature in cursive script, appearing to read "A. P. Watt".

A. P. WATT
REGIONAL BRIDGE LOCATION ENGINEER

APW:gf
ATT'D

c.c. Mr. S. McCombie
Mr. R. Forrest

John Bond ✓

Lender ✓
Material ✓
E

LOND CHAT 5 FEB 6/67 4:36

REP WATT BRIDGE

PP 83-59 SITE 13-258 HWY 401 CENTRELINE ROAD

YOUR MEMO AND PLAN OF FEB 3. PROPOSAL AGREED. YOUR COST LIKES A BIT
OUT AND MAYBE A TYPO ERROR. SHOULD IT BE \$193,000. EQUIVILENT JOBS
HAVE BEEN RUNNING CONT 64-148 ^{Parry} \$215,000, 66-98 \$200,000, ✓
66-172 \$255,000. 2 structure

P H PEACOCK CONST ENGR

Called Mr. P. H. Peacock on Feb. 8/67 and explained that
our figure is only bridge costs. He suggests to ignore his T T
Apr.

T
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P
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MEMORANDUM

To: Mr. A.P. Watt,
Regional Bridge Location Engineer,
Department of Highways,
LONDON REGIONAL OFFICE.

FROM: Mr. D.D. Murray,
Sr. Project Design Engineer,
Road Design Section,
LONDON REGIONAL OFFICE.

DATE: February 7th, 1967

Our File Ref.

IN REPLY TO

SUBJECT:

W.P. 83-58 - Hwy. #401 Underpass
Centre Line Road, Twp. of Harwich

In reply to your memo of February 3rd, 1967, the preliminary plan shows a slight error in the elevation of the profile grade.

The P.I. of the profile grade should be changed from EL. 642.75 to EL. 643.23 and the finished crown of pavement should be shown as "0.27 above revised profile grade".

These changes are shown in red on the attached copy of the preliminary plan, Dwg. No. D 6128-P1.

DDM:jo


D.D. Murray,
Sr. Project Design Engineer

RECEIVED

FEB 8 1967

REGIONAL OFFICE

MEMORANDUM

To: Mr. A. P. Watt,
Reg. Bridge Location Engineer,
Department of Highways,
LONDON REGIONAL OFFICE.

From: Mr. R. A. Shannon,
Regional Traffic Engineer,
LONDON, Ontario.

Date: February 7, 1967.

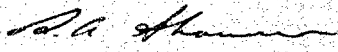
Our File Ref.

IN REPLY TO

SUBJECT: Re: W. P. 83-59, Bridge Site 13-268,
Centreline Road Underpass,
9.1 miles West of Highway #21,
Highway #401, District 1, Chatham

This will acknowledge receipt of your memo of February 3rd in which you inquired if illumination is required on the above structure.

This structure is located in a rural area and will not serve as a cloverleaf, therefore, illumination is not required at present nor any provision for future illumination.


R. A. Shannon,
Regional Traffic Engineer

RAS:aa

RECEIVED
FEB 14 1967
REGIONAL OFFICE

FORM
SB-OS-62
66-1593

DEPARTMENT OF HIGHWAYS ONTARIO

ACTION SLIP

DATE

Feb. 21/67

TO

*Mr. L. Dzielski, Bridge Design Engineer
Bridge Office, Downsview*

FROM

A. P. Watt

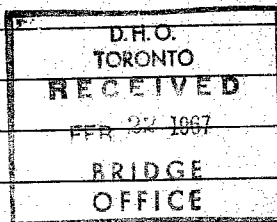
- ☐ NOTE AND FILE
- ☐ NOTE AND RETURN TO ME
- ☐ RETURN WITH MORE DETAILS
- ☐ NOTE AND SEE ME
- ☐ PLEASE ANSWER
- ☐ FOR YOUR APPROVAL
- ☐ RETURN WITH YOUR COMMENTS

- ☐ PREPARE REPLY FOR MY SIGNATURE
- ☐ TAKE APPROPRIATE ACTION
- ☐ PER YOUR REQUEST
- ☐ FOR YOUR SIGNATURE
- ☒ FOR YOUR INFORMATION
- ☐ INVESTIGATE AND REPORT

COMMENTS

Re: WP 83-59 Bridge Site 13-268

Centreline Road Underpass



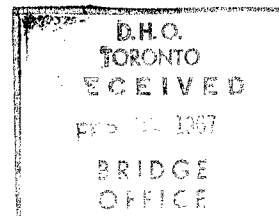
SUBJECT

YOUR FILE
OUR FILE

THE BELL TELEPHONE COMPANY OF CANADA

55 Sixth Street,
Chatham, Ontario,
February 17, 1967.

Department of Highways of Ontario,
Mr. A.P. Wett, P. Eng.,
Regional Bridge Location Engineer,
Box 4544 Postal Station "C",
London, Ontario.



Dear Sir:

Re: W.P. 83-59, Bridge Site 13-268
Centre Line Road Underpass,
9.1 miles west of Highway #21
Highway 401,
District 1, Chatham

This is in reply to your letter of February 3, 1967, regarding the conduit requirements of the Bell Telephone Company of Canada at the above site.

After careful consideration of the area involved, we do not see any requirement for conduit in the proposed bridge structure.

We appreciate your co-operation in this matter.

Yours truly,

A handwritten signature in cursive script, appearing to read "W. H. Miller".

for Assistant Chief Engineer

EMW:dm

c.c. file
J.V.H.

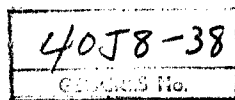
RECEIVED
FEB 20 1967

REGIONAL OFFICE



ONTARIO

DEPARTMENT OF HIGHWAYS



Memo to Mr. A. M. Toye, **Date** April 19, 1960.
Bridge Engineer. **Subject** FOUNDATION INVESTIGATION -- by
From Materials & Research Section. William A. Trow & Associates.

Attention: Mr. S. McCombie.

Re: Proposed County Road Underpass,
Hwy. 401 - Ridgetown, Ontario.
W.P. 83-59 -- District No. 1

We have reviewed the Report on the foundation investigation at the above site, submitted by W. A. Trow & Associates, Ltd.

We would agree with the conclusions and recommendations made in the above-mentioned report, except that we think that some difficulties could arise from the fact that a more compressible layer has been found in one place - i.e., in B.H. No. 4. Although the compression curve of this material, when used for settlement computations, gives more favourable results, it is our opinion that this material is more compressible than the other materials. A much higher water content and a smaller value of the relative consistency are the basis of our belief. The presence of a probably more compressible layer is going to be investigated in the very near future. For the time being, we would recommend for you to follow the recommendations given in the above mentioned report. For your convenience, we give you here, the summary of these recommendations:-

1. The soil at the proposed underpass consists, for the most part, of **very** stiff to hard glacial till material. Bedrock lies about 87 feet below ground surface.
2. Footings for the bridge structure can be placed at a depth of 4 ft., at which level the recommended safe bearing value is 8,000 p.s.f.
3. The settlements will be somewhere in the order of 4 - 5 inches. This movement should continue over a period of 36 years. These settlements refer to the abutments, while the settlement of the centre pier is estimated as approx. 1 inch.

cont'd. /2 ...

Recommendations: (cont'd.) ...

4. There are no stability problems for the approach embankment fill.

The above recommendations should be changed only if the additional investigation shows a distinct difference in soil properties. In this case, we will let you know about the changes.

If there are any details you would like to discuss, please feel free to call on us.

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

Per:



(A. Stermac,
FOUNDATION OFFICE ENGR.)

AS/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.