

WILLIAM A. TROW AND ASSOCIATES LTD.

SIT. 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: 11

March 25, 1960.

BA 1026

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

Attention: Mr. L. G. Soderman, P. Eng.

Foundation Investigation
Proposed County Road Underpass
North of Ridgetown, WP 87-59.

Dear Sirs:

Enclosed herewith is our report on the soil conditions existing at the bridge site noted above.

Four borings were made at this location, two of which went down to or close to bedrock. The material encountered in these holes was found to consist for the most part of stiff to very stiff cohesive soil. Between depths of approximately 13 and 50 feet was a glacial deposit described as sandy silty clay with embedded gravel. Above and below was a stiff more plastic clay with partings or layers of very fine sand.

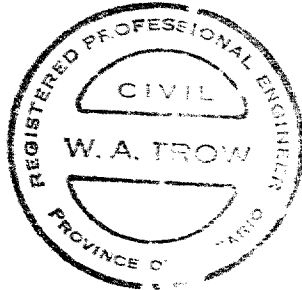
We have suggested the use of normal abutment footings bearing 13 feet below ground surface at this site. Because of the weight of the adjacent embankments a long term settlement of the order of 4 inches could occur with this arrangement. If a centre pier is to be used in this structure, the resulting differential movement may be unacceptable. If so, the abutments should be founded on H piles, end-bearing on bedrock, which lies about 90 feet below the surface.

In the estimation of settlement for this project, we have tended to generalize somewhat. It is possible that closer approximations could be obtained if the dimensions and weight of the structure were known more accurately and if numerous consolidation tests were performed in material from different depths and locations. We have felt that the additional time and expense involved

in the refinement of these calculations was not warranted, since some uncertainty would still remain after the additional work had been carried out. Please contact us if you wish to discuss this matter, or any other query arising out of this investigation.

Yours very truly,

WAT/lt
Encls.



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

WILLIAM A. TROW AND ASSOCIATES

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

FOUNDATION INVESTIGATION
PROPOSED COUNTY ROAD UNDERPASS
NORTH OF RIDGETOWN, ONT.:WP 87-59

Project: J477

William A. Trow & Associates Ltd.

March 25, 1960.

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FOUNDATION INVESTIGATION
PROPOSED COUNTY ROAD UNDERPASS
NORTH OF RIDGETOWN, ONT., W P 87 - 59

This report contains a description of the foundation conditions existing at the intersection of Hwy. 401 and the County road dividing Lots 6 and 7 of Concession 6, north of Ridgetown, Ont. A discussion of foundation requirements has been given. Details of field investigation methods are presented in the Appendix.

Site Description

The County road at this intersection lies in farmland which is flat with only minor changes in elevation of the ground surface. There are small wood lots about 1000 feet to the east and west, but generally, the fields are cultivated. Drainage ditches cut across the land to the north and south of the underpass location and very shallow ditches lie on each side of the County road.

The ground was snow-covered during the investigation period.

Description of Subsoil

The soil conditions at this site were determined by two borings taken to refusal 77 feet or more below ground surface and by two shallower holes. The soil types encountered at each test location are indicated in the borehole logs, Dwg. 2 to 5 of this report. The stratigraphical profile, shown in Dwg. 1, can be considered to be more or less representative of the materials that underlie the County road intersection. A brief description of each stratum follows.

Stiff clay with fine sand partings: This stratum extends to a depth of about 13 feet below present ground surface, or to Elev. 610 feet approximately. In all holes the main body of plastic clay begins about 6 to 8 feet below the surface and above this depth it is interbedded with fine brown water-bearing sand. The water table in the sand stabilized quickly about 2 feet below the ground surface at the time of the investigation. The moisture content of the clay lies a few percent above the plastic limit, but well below the liquid limit value. The Atterberg limit and moisture content measurement for this material, shown in Table 1, indicate considerable variation in plasticity. This is due to varying percentages of fine sand in the clay samples. The liquid and plastic limits of the sand-free portions of the clay are about 45 and 19 percent, respectively. The shear strength of this upper clay ranges from about 1150 to 1500 p.s.f.

Silty clay till: After a short transition from the plastic clay described above, the soil grades into a glacial drift deposit consisting of sandy silty clay with small gravel. This material exists in a very stiff condition and

its natural moisture content is very close to the plastic limit. The percentage of fine sand and gravel varies in this soil as evidenced by the different moisture content and Atterberg limit test results. Liquid and plastic limit values in the order of 38 and 17 percent respectively are probably representative of the plastic range of a large part of this material. The undrained shear strength, as determined by two triaxial shear tests is approximately 4000 p.s.f. Lower shear strength values of 2270 and 2000 p.s.f. were recorded at the bottom of the transition zone about 13 feet and 10 feet below the surface in Holes 1 and 3.

Stiff clay with fine sand partings: This stratum begins about 50 to 55 feet below the present ground surface and it extends to a depth of 77 feet. It is somewhat more plastic than the overlying till and, on the basis of shear strength and moisture content measurements, it appears to be more compressible. According to field vane tests, its shear strength is in the order of 1900 p.s.f., and its natural moisture content is well above the plastic limit.

Gravel over bedrock: Although not proven, a mixture of sand and gravel appears to overlie bedrock which is 90 feet below the ground surface. Refusal was encountered at the top level of this material at a depth of 77 feet, or Elev. 546 feet in Hole 1, and it was penetrated to refusal and assumed bedrock at 90 feet, or Elev. 532.6 in Hole 3.

It is of interest to note that bedrock was encountered at Elev. 547.3 feet at a site about 3 miles to the south-west and it was found at Elev. 542 feet at the Hwy. 21 intersection of Hwy. 401, approximately 3 miles to the north-east. The stratigraphy of the overlying soil also is in reasonable agreement with the conditions noted at these other sites.

A summary of some of the significant physical properties of the soil strata at this site is presented in Table 1. Typical stress strain curves for the undrained triaxial tests are shown in Dwg. 8. No consolidation tests were performed because the materials are identical to the soil types investigated at bridge sites designated WP 83-59 and WP 86-59, further to the west. Consolidation test data for these projects are included in this report.

Foundation Considerations

Since the subsoil conditions at this site generally are similar to those existing at other proposed County road crossings immediately to the west, the same foundation requirements and resulting long term behaviour should apply. At these other locations the support of bridge abutments on the natural soil at shallow depth below ground surface was suggested. Although the underlying soils were found to be in an over-consolidated condition, long term settlement was anticipated because the weight of the approach fill would affect a considerable depth of the slightly compressible soil below. This was felt to be of concern only if the design of the structure incorporated a centre pier. Differential settlement between the pier and the abutments could be avoided only by supporting the latter units on H-piles, end-bearing on rock.

Although the foregoing proposal appears applicable at this site, it is suggested that footings must be taken right down to the hard glacial till which exists below approximate Elev. 610 feet, or about 13 feet below ground surface. The shear strength of the overlying more plastic clay is somewhat variable and becomes as low as 1150 p.s.f. within the zone affected by shallow footings. As a consequence, the safe bearing pressure to apply at shallow depth would be much less than the 4000 p.s.f. value usually desired for abutment design. The net safe bearing pressure to apply to the underlying silty clay till at and below a depth of 13 feet is 6000 p.s.f. This pressure recommendation is based on the fact that the moisture content of the till is at or close to the plastic limit and that its shear strength is in the order of 2500 p.s.f. just below 13 feet.

As suggested in a foregoing paragraph, long term settlement of the abutments will still occur because of the compressing effect of the adjacent fill. In order to obtain an estimate of the magnitude of this long term movement, a modulus of compressibility of 0.0062 sq.ft. per kip will be assumed. This is the measurement obtained on a sample of clay from the WP 83-59 project north of Blenheim. As stated in the report for that site, the soil in place is probably much less compressible than this and the other test measurements would suggest. This discrepancy results, in part, from inevitable sample disturbances occurring during the recovery of the sample and its preparation for test. It is undoubtedly the case for the test on the glacial till since this sample contained so many gravel particles that some patching of it had to be carried out.

The computations involved in this estimate of settlement are presented in the Appendix. The loading conditions are assumed to be the same as for the WP 83-59 project. Reference to this information indicates that the estimated long term settlement of the abutments is about 8 inches. Immediately adjacent to the abutments, the fill will settle an additional $1\frac{1}{2}$ inches approximately because of the compression of the clay about Elev. 610 feet. Settlement of this upper material will occur at a relatively rapid rate because it contains numerous fine sand layers and partings.

For the reasons indicated above, this estimate of settlement is believed to be excessive and long term movements of about 4 inches are more probable. If the centre pier is used and it is loaded up to a pressure of 6000 p.s.f., it should be expected to settle about one inch. As a consequence, ultimately the differential movement between the abutments and the centre pier will be in the order of 3 inches. Conceivably, this could be the situation after a period of about 30 years. If this final differential movement cannot be tolerated, the abutments should be founded on H piles end-bearing to rock. In this instance, the soil pressure of the centre pier should be reduced in order to minimize the settlement of these units.

Some ground water will enter the excavations for the footings. This flow will be from the water-bearing sand noted above a depth of about 7 feet. However, after the excavations have been pumped two or three times, the quantity of seepage should be reduced considerably. This should be the case

also if construction work is undertaken during dry summer months. The walls of the excavation should stand vertically without the need for shoring.

The embankments should remain stable during and after placement since the underlying soil has sufficient shearing resistance to support its weight safely. Placement of the earth fill can be carried out at any stage in the construction program.

Summary of Comments and Conclusions

1) The subsoil at this site is quite similar to the material encountered at the other bridge sites along the route of Hwy. 401 in the Ridgetown area. The predominant soil type is a very stiff silty clay glacial till deposit which extends at most locations from a depth of 13 to 50 feet below ground surface. Above and below this material, the soil consists of a somewhat more plastic clay with partings of fine sand. The sand layers are larger and quite frequent in the first approximately 7 feet of subsoil depth.

Assumed bedrock lies about 90 feet below the ground surface and it is overlain by a deposit of dense gravel soil.

2) Bridge abutments and piers can be placed on the glacial till material at a depth of 13 feet, at which level the safe bearing value is 6000 p.s.f. It is estimated that the long term settlement of the abutments will be in the order of 4 inches, although a value of 8 inches was computed. The settlement of the centre pier should be one inch or less. Some ground water seepage will be experienced when excavating for the abutments.

3) If the suggested foundation depth and associated settlements, estimated above, are unacceptable, the abutments should be placed on end-bearing piles driven to bedrock.

4) No embankment stability problem exists at this site.

WAT/lt
March 25, 1960.
J 477



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

APPENDIXSETTLEMENT COMPUTATIONSAbutments

Assume footings placed at a depth of 15 feet, exerting an average net pressure of 4000 p.s.f.: assume footings 40 ft. long and 10 ft. wide: assume a load distribution into the soil at 30° to the vertical.

Assume coefficient of compressibility $M_v = 0.00622$ sq.ft. per kip to be applicable for entire depth of the soil. This value is believed to be conservative for the till even though a higher value of M_v was obtained for the glacial till in Dwg. 7, WP 86-59.

Assume approach embankments 30 ft. high, top width 40 ft.; side slopes $1\frac{1}{2}:1$; fill and soil weight = 125 pcf above the water table; soil weight below water table = 70 pcf; water table at 4 ft. (borings made in ditches below road level showed level at approximately 2 feet).

Distribution of fill pressure at abutments computed using Newmark diagram.

Computation of Settlement of Abutments

Depth below footing	Average Pressure due to footing	Average Pressure due to fill.	Total Press. P	$S = H M_v P$ $= 120 \times .00622 P$
5	2260	1170	3430	2.56
15	1000	1024	2024	1.51
25	560	985	1545	1.15
35	410	972	1382	1.03
45	300	970	1270	0.95
55	210	970	1180	0.88

Total 8 ins. approx.

Duration of Settlement

Coefficient of Consolidation $C_v = 0.123$ sq.ft./dy. (WP83-59)
Time for 50% Consolidation or 4 ins. settlement. Assuming drainage to surface and to gravel over bedrock:

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

$$\text{Time for 90\% consolidation: } \frac{0.848 \times 40^2}{0.123 \times 365} = 30 \text{ yrs.}$$

APPENDIXEstimated Additional Settlement of Clay above 14 feet

Increment of fill pressure at depth of 7 feet, close to the side of the abutment, approximately 2000 p.s.f.

Settlement of approx. 10 feet of clay in this upper stratum:

$$S = 120 \times .00622 \times 2000 = 1.5 \text{ ins.}$$

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 or refusal just above; 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 most cases the ground was too stiff.

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

[illegible]

PROJECT County Road Underpass RP 87-36

LOCATION North of Ridgeway, Ont.

HOLE LOCATION See Inv. 1

HOLE ELEVATION AND DATUM: 623.2 BM see Pwr. 1

BOREHOLE NO. 1

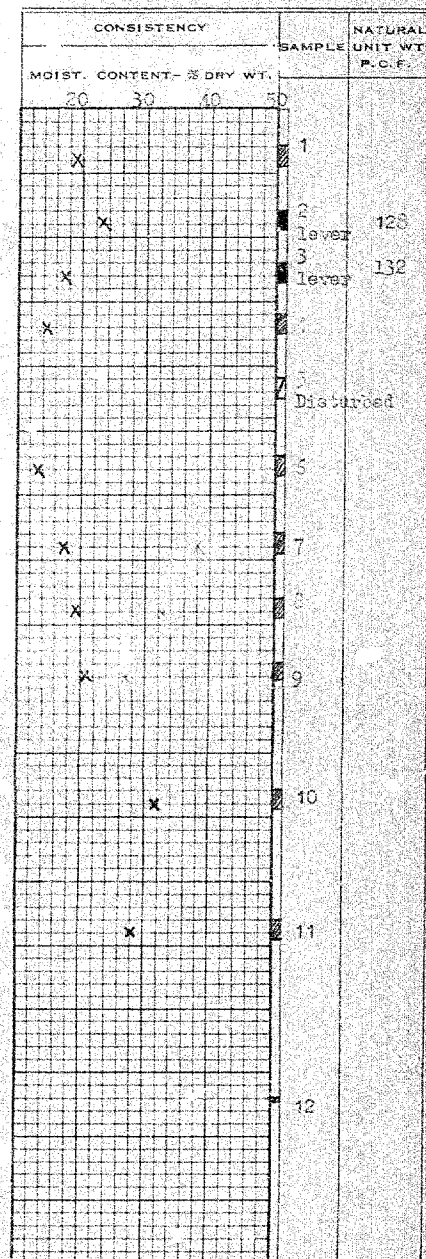
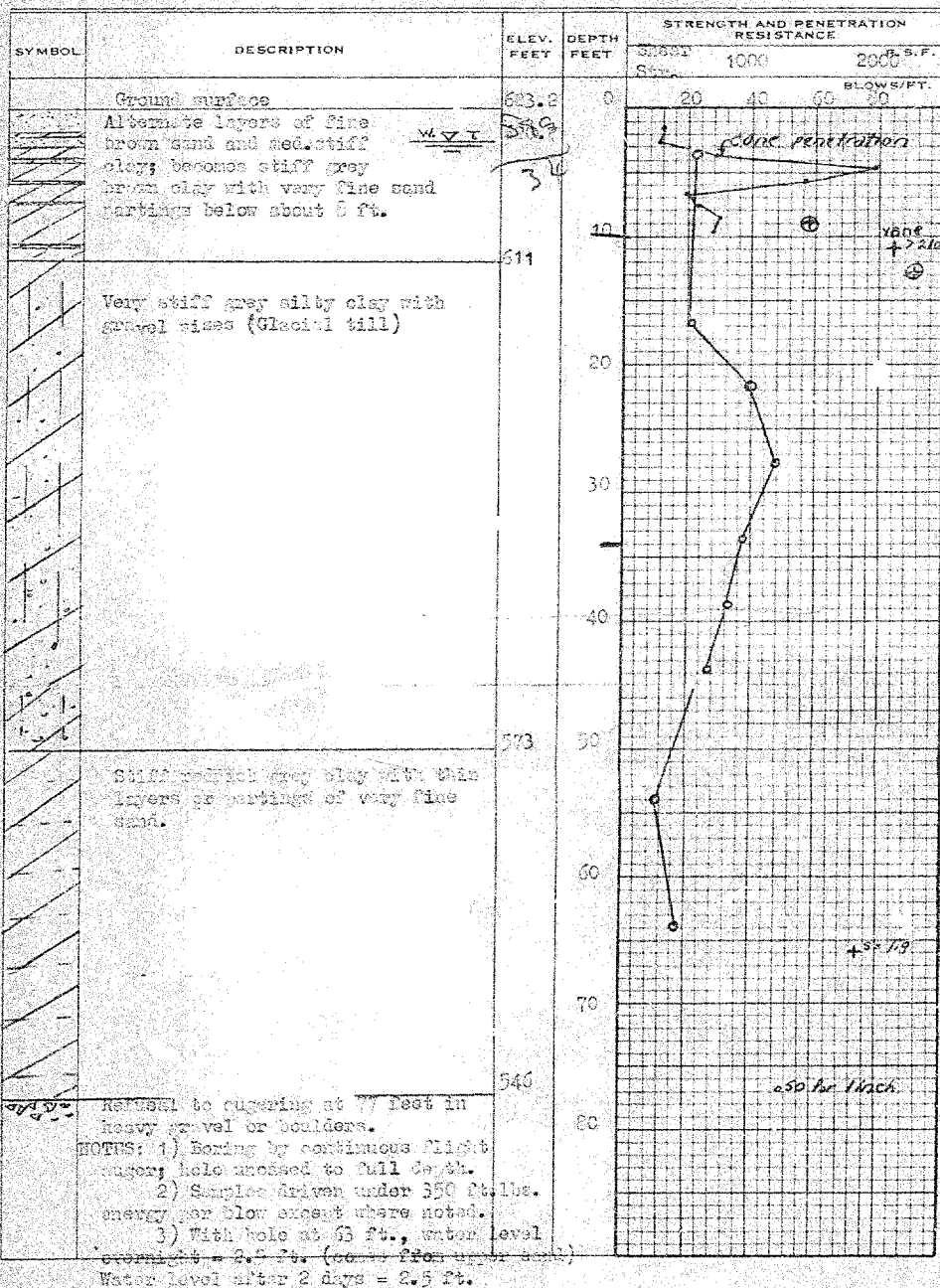
FIELD SUPERVISOR

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



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

PROJECT NO. J 477

DRAWING NO. 3

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT County of Delaware, DE-2

BOREHOLE NO. 2

LOCATION North of Elizabethtown, Del.

FIELD SUPERVISOR

HOLE LOCATION See Dwg. 1

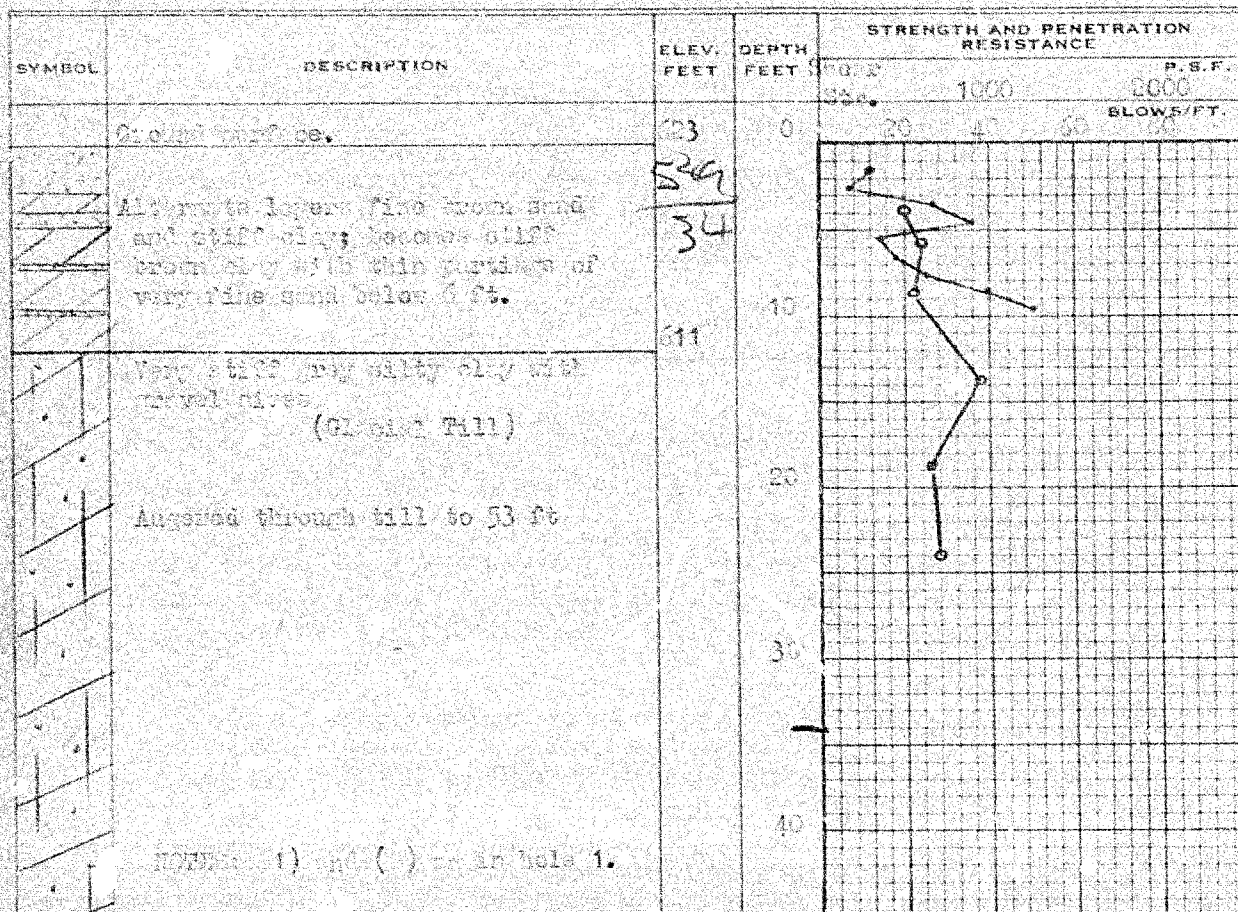
DRILLER

HOLE ELEVATION AND DATUM 33.0 H 200 P 1.1

PREP.

LEGEND

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



CONSISTENCY				SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.					
50	30	10	50		
				1	
	X			2	
	X			3	
	X			4	
X				5	
				6	
	X			7	
X				8	
				9	
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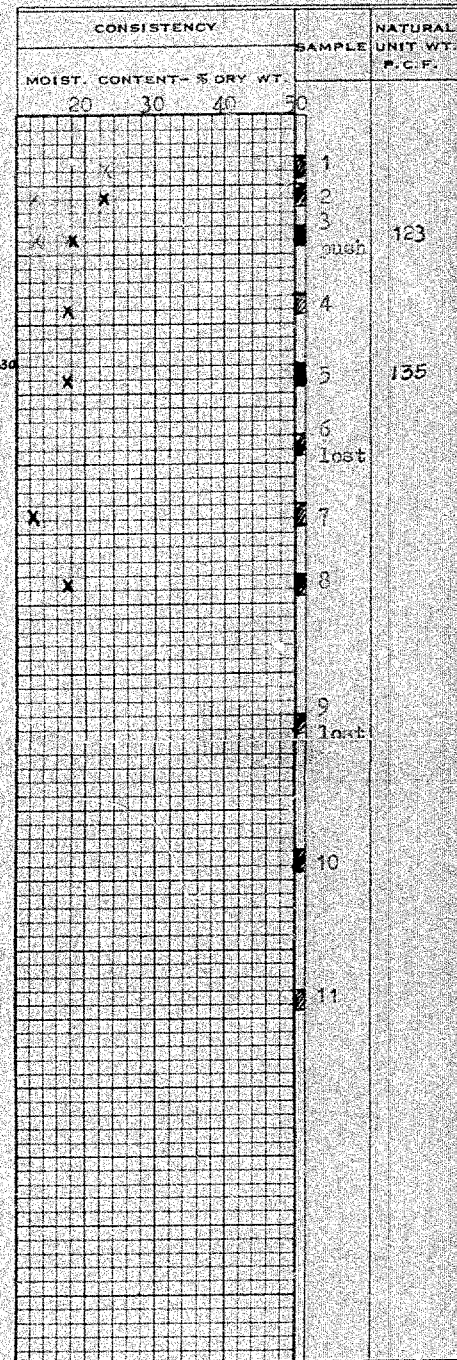
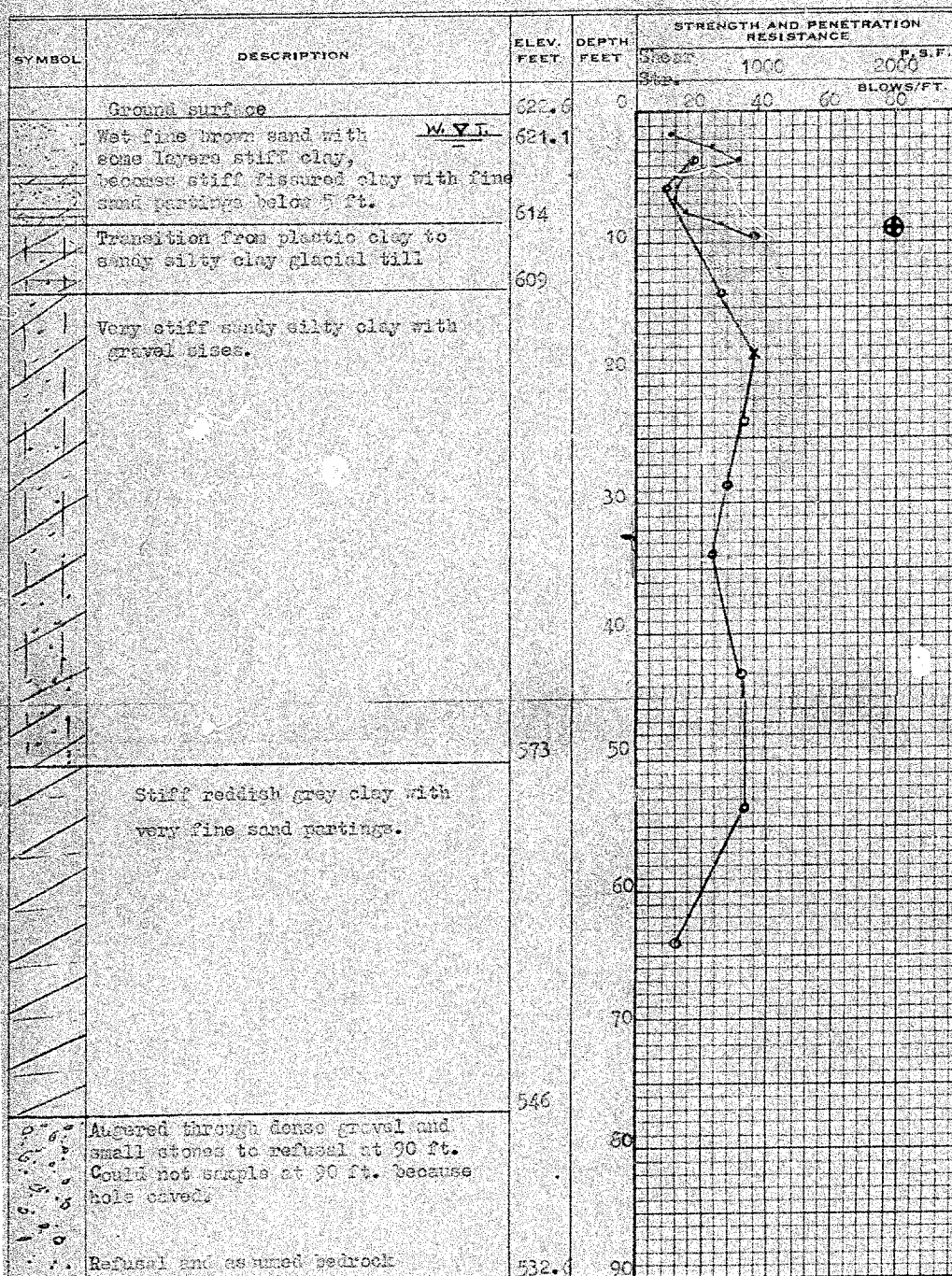
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT County Road Underpass WP 51-59
 LOCATION North of Ridgeway
 HOLE LOCATION Sess Dr. 1
 HOLE ELEVATION AND DATUM 622.6

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



NOTES: (1) and (2) as in H. 1
 (3) With hole at 63 ft., over night WL = 1.5 ft.

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

PROJECT NO. J-47

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

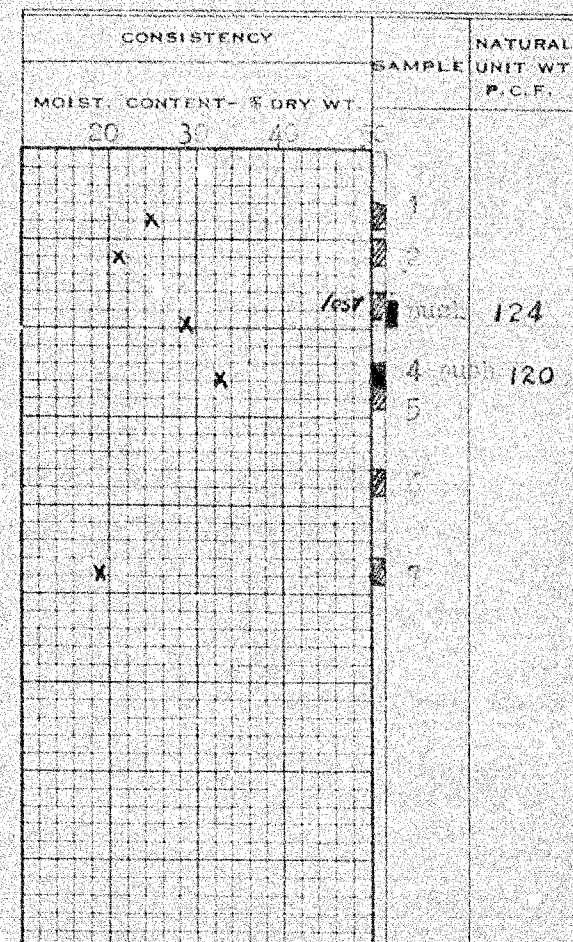
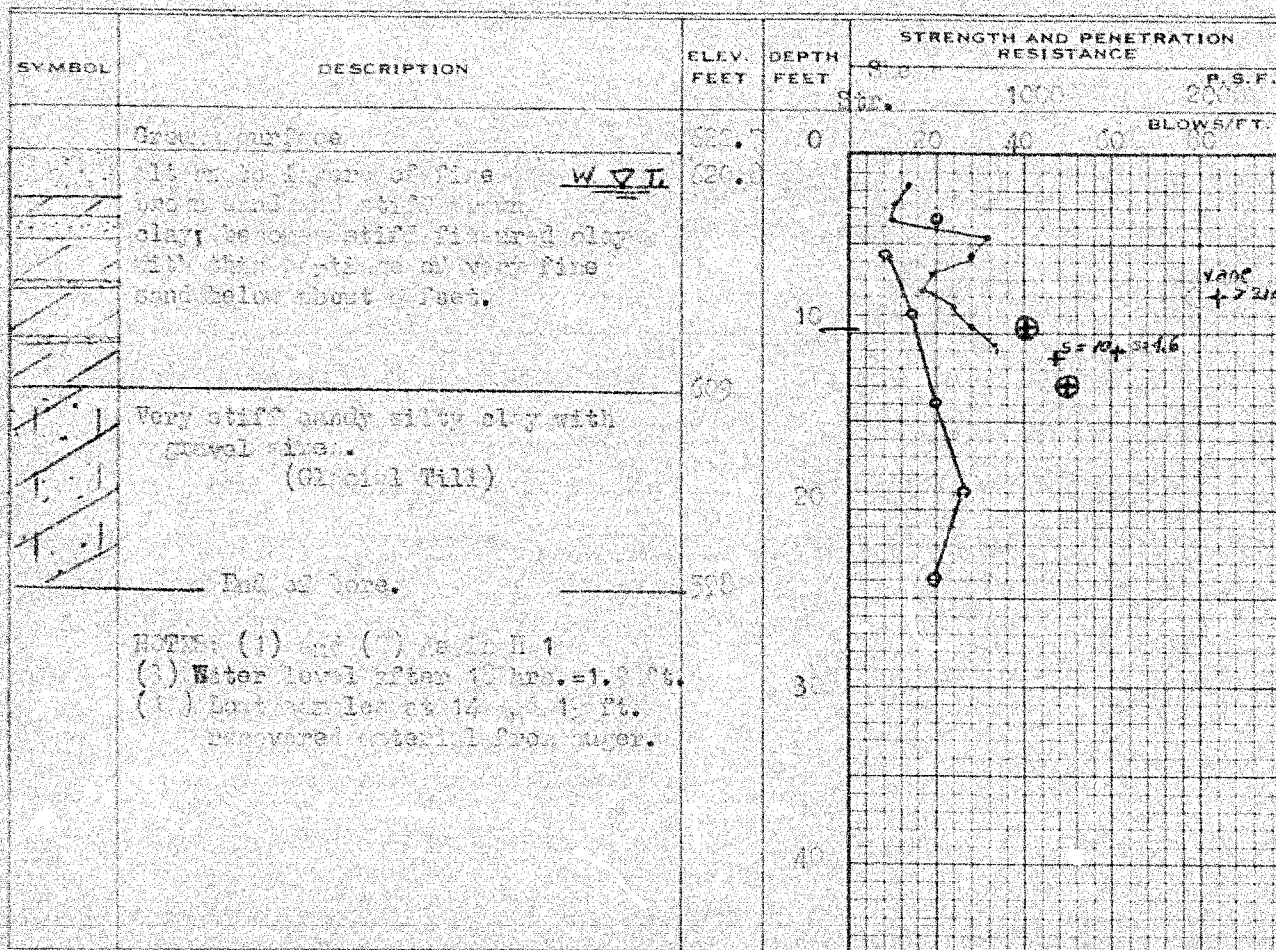
PROJECT: CONCRETE PILE FOUNDATION
LOCATION: 1000 S. 10th St., S.W., ALBANY, N.Y.
HOLE LOCATION: See Plan
HOLE ELEVATION AND DATUM: 120.5

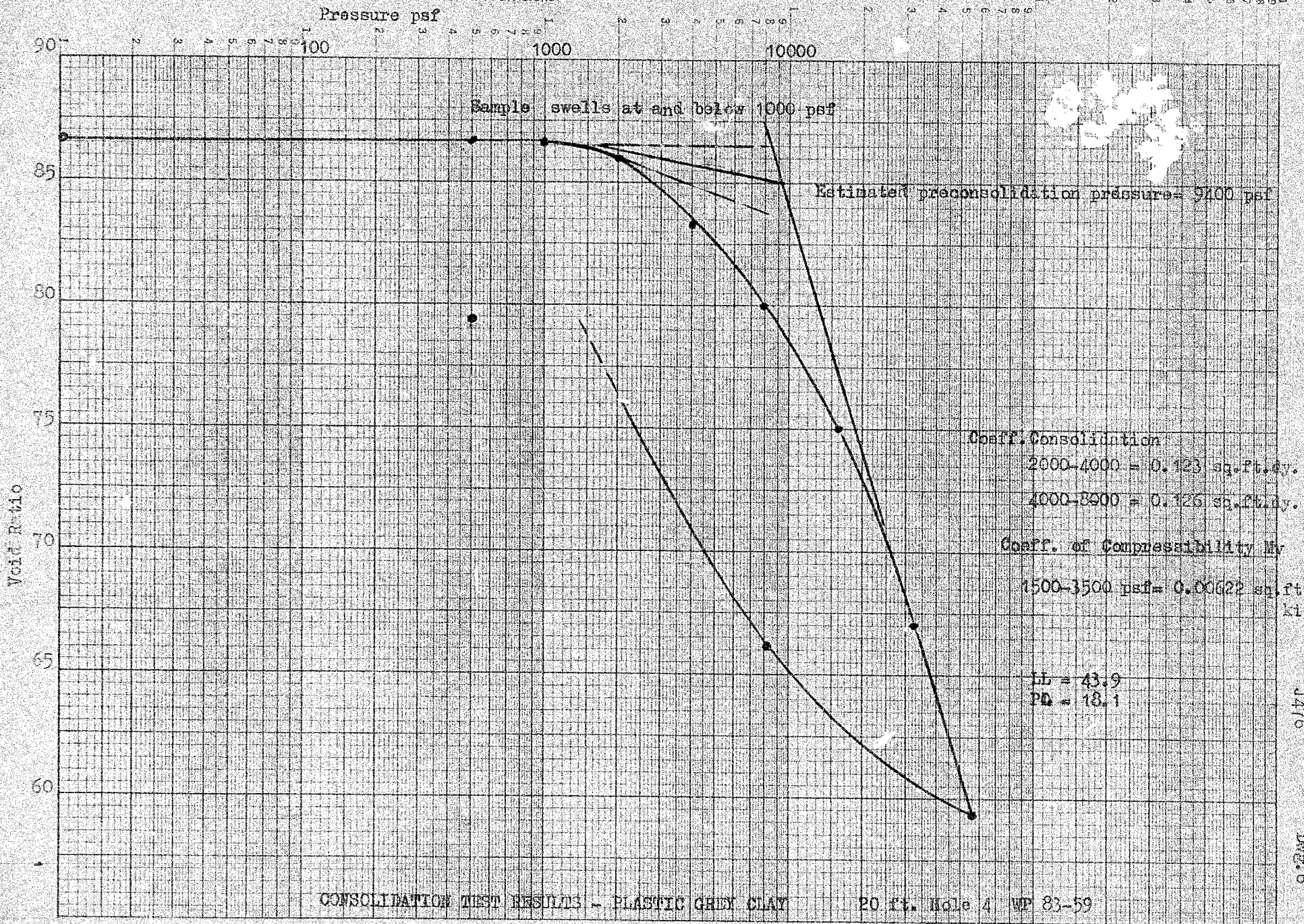
BOREHOLE NO. 1
FIELD SUPERVISOR: _____
DRILLER: _____
PREP: _____

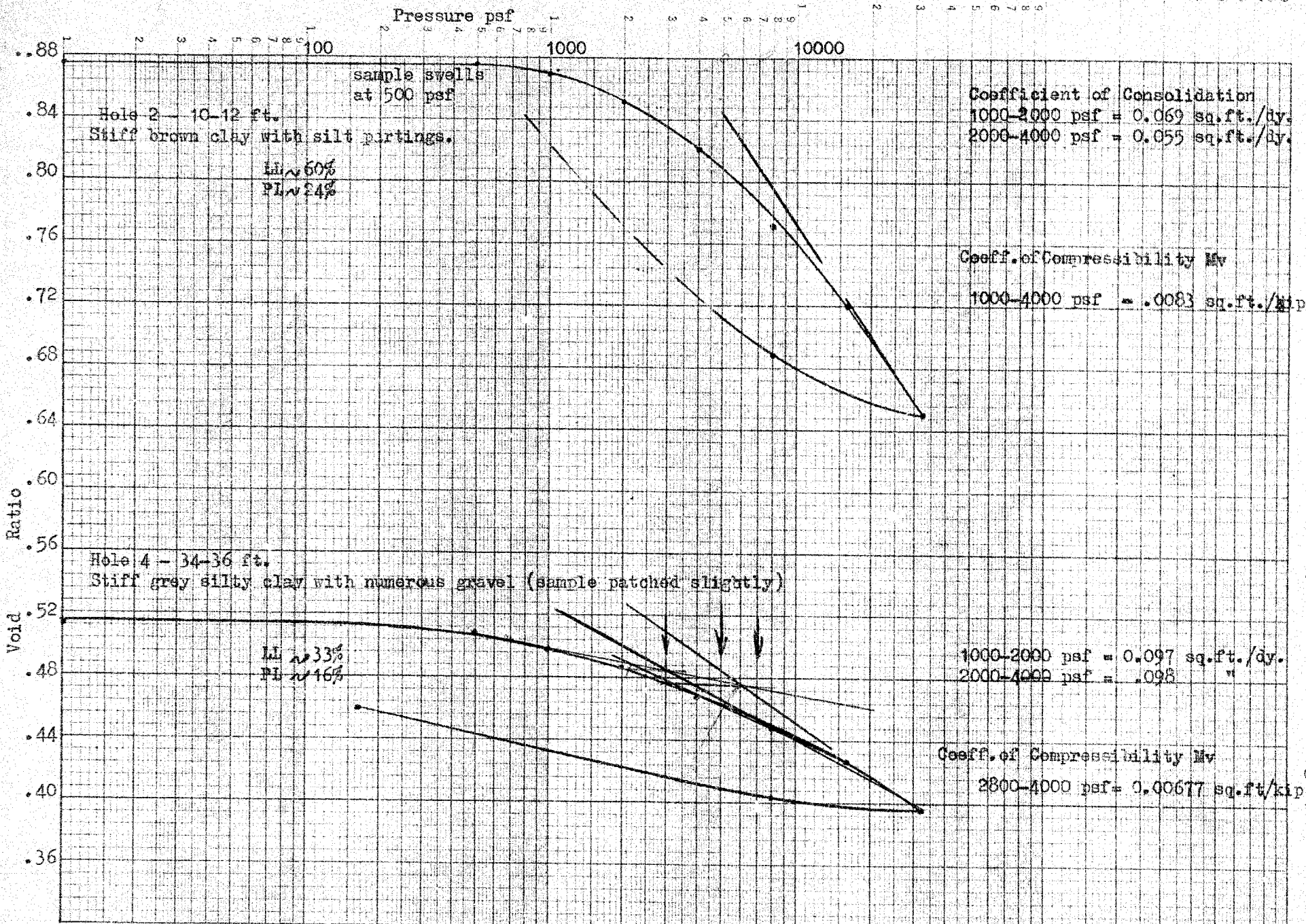
DRAWING NO. _____

LEGEND

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



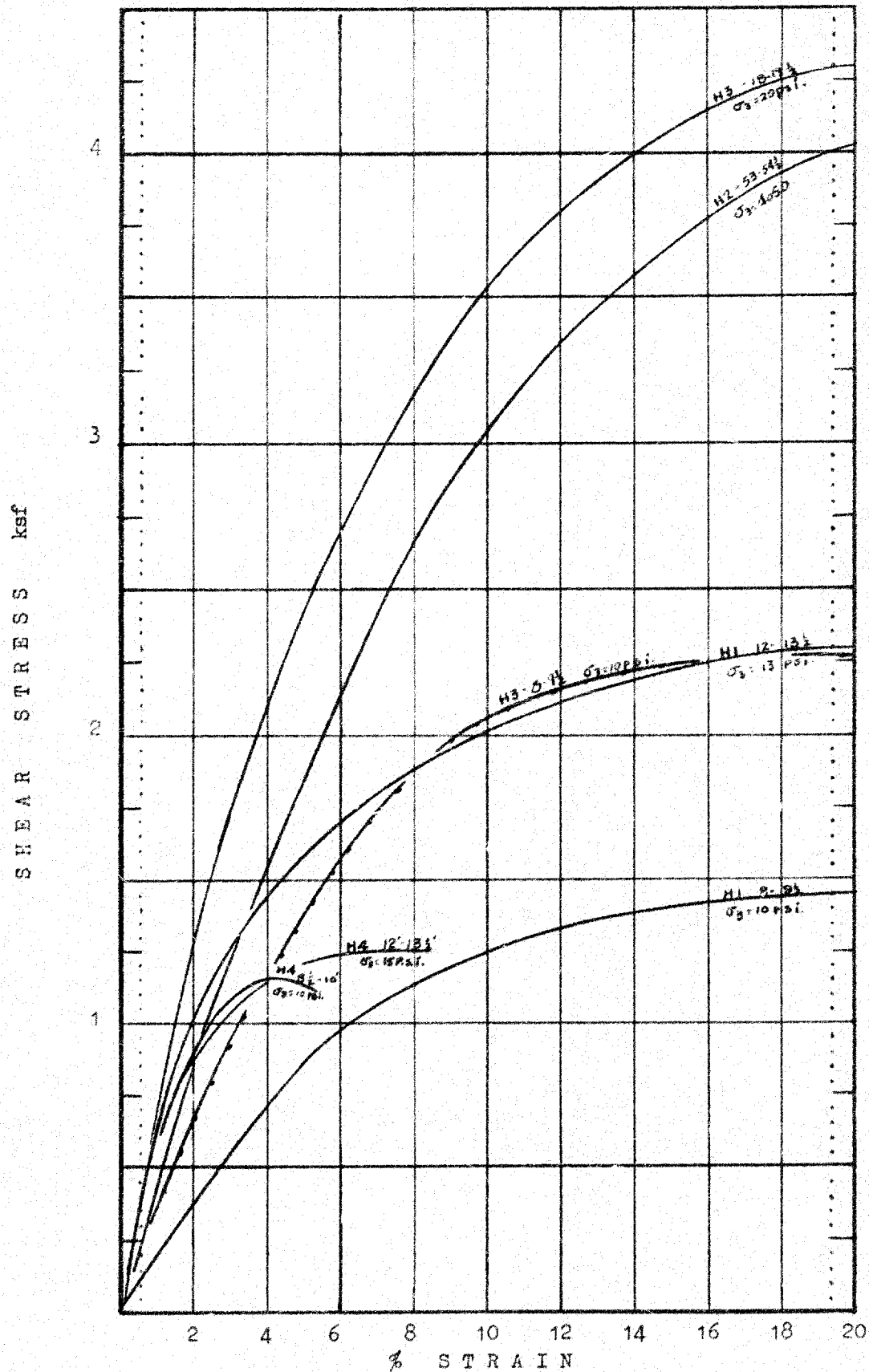




CONSOLIDATION TEST RESULTS HOLES 2 and 4 WF 86-59

Project 3470

Dwg. 7



STRESS STRAIN CURVES UNDRAINED TRIAXIAL TESTS WP 87-59



ONTARIO

DEPARTMENT OF HIGHWAYS

335 Saskatoon Street,
LONDON, Ontario,
July 28, 1960.

MEMORANDUM FOR-
Mr. A. Toye,
Bridge Engineer,
Department of Highways,
Parliament Buildings,
TORONTO, Ontario.

Attention: Mr. S. McCombie,

RE; W.P. 87-59 Howard Twp.
Bridge #6 Highway #401.

We have received the preliminary bridge drawing D-4627-P1
from G. Scott and have the following comments:-

1. Alignment is correct.
2. Grade has been raised 1 foot and Road Design Office has changed the "F" profile accordingly.
3. Two 24' pavements, 10' side clearance and 50' median provided on Highway #401 is correct.
4. 28' driving surface provided on the county road is adequate.
5. Earth grading, granular and paving of the approaches will be included with the structure contract.
6. Drainage of the bridge deck will be handled by curb and gutter on the approaches.
7. Drainage on Highway #401 will flow both ways from the structure.

J.A. KNOWLES,
Project Design Engineer.

JAK-nc



Bridge Division,
August 12, 1960.

MEMORANDUM TO:

Mr. L. G. Soderman,
Principal Soils & Foundations Engineer,
Room #121,
Downsview, Ontario.

To confirm our conversation from August 11, 1960,
we are continuing designs of Howard 3 & 6 Twp. Bridges
on the following bases:

Howard 3 W.P. 88-59 (BA 1023)

Ftng. bottom EL. of pier B was lowered to EL. 618

Howard 6 W.P. 87-59 (BA 1026)

Ftng. bottom EL. of piers A & C are as EL 617 and no piles used; pressure not to exceed 2 tons/ sq. ft.

We use tube piles for the Abutments Footings, with design load of 20 tons, which should go to EL. 604.

FG/dd

F. Gormek
F. Gormek,
Supervisory Engineer,
Bridge Office.

Handwritten: Handwritten
Please get photo of
this and file in
copy and report
FBI report
Handwritten: Handwritten
F. Gormek
Supervisor
Bridge Of

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

April 4, 1960.

FOUNDATION INVESTIGATION - by
W. A. Trow & Associates, Ltd.

Attention: Mr. S. McCombie.

Re: Proposed County Road Underpass
North of Widgetown, District 1,
W.P. 87-59.

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. It is also our opinion that the computed settlements are based on a conservative value of the coefficient of compressibility and will therefore most probably be smaller than outlined in the 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:

A. Sternac
(A. Sternac,
FOUNDATIONS OFFICE ENGR.)

AS/MdeP
Attach.

cc: Messrs. A. M. Toye (2)
B. A. Tregaskes
D. C. Ramsay
A. Cater
G. U. Howell
J. Roy
A. Watt
Foundations Office
Gen. Files.

#60-F-273C

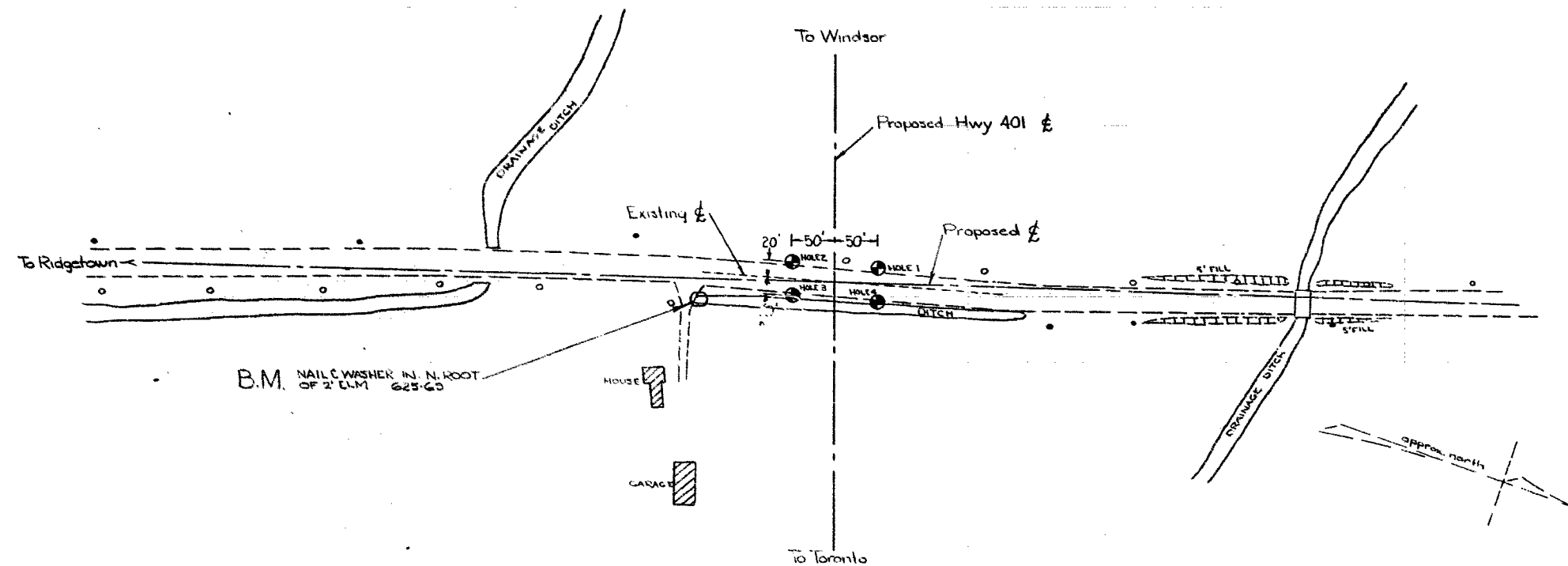
W.P. # 87-59

HWY. # 401

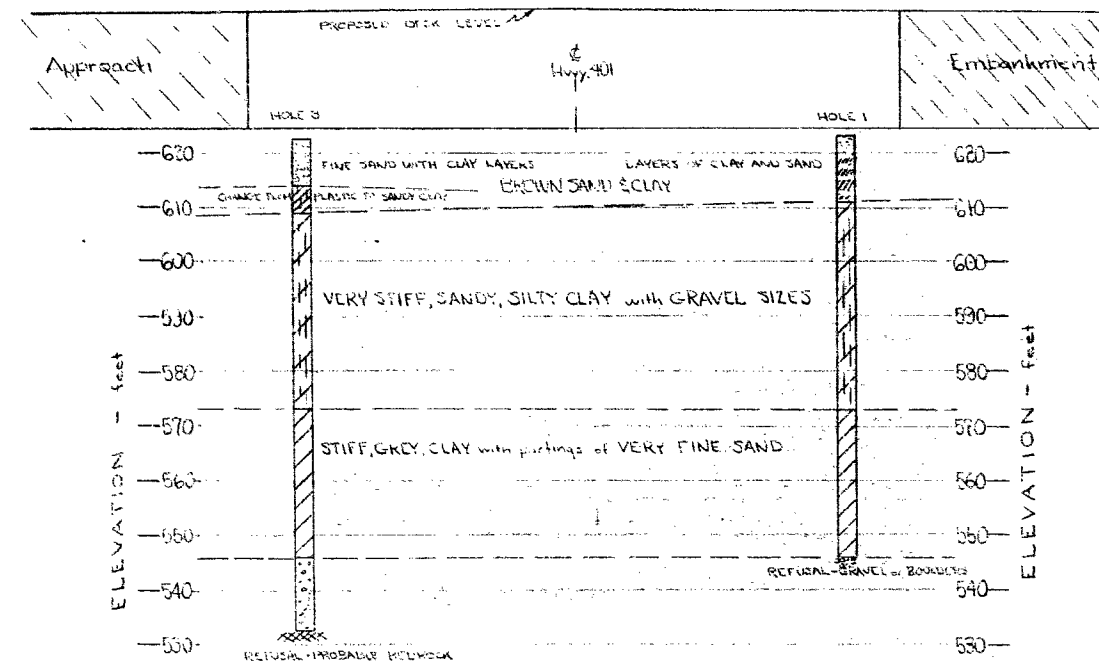
PROPOSED COUNTY

RD. UNDERPASS

RIDGETOWN.



BOREHOLE LOCATION PLAN SCALE 1"=100'



ESTIMATED STRATIGRAPHY SCALE 1"=20'

PROPOSED UNDERPASS
W.P. 87-59

FOUNDATION INVESTIGATION
WM. A. TROY & ASSOCIATES LTD.

