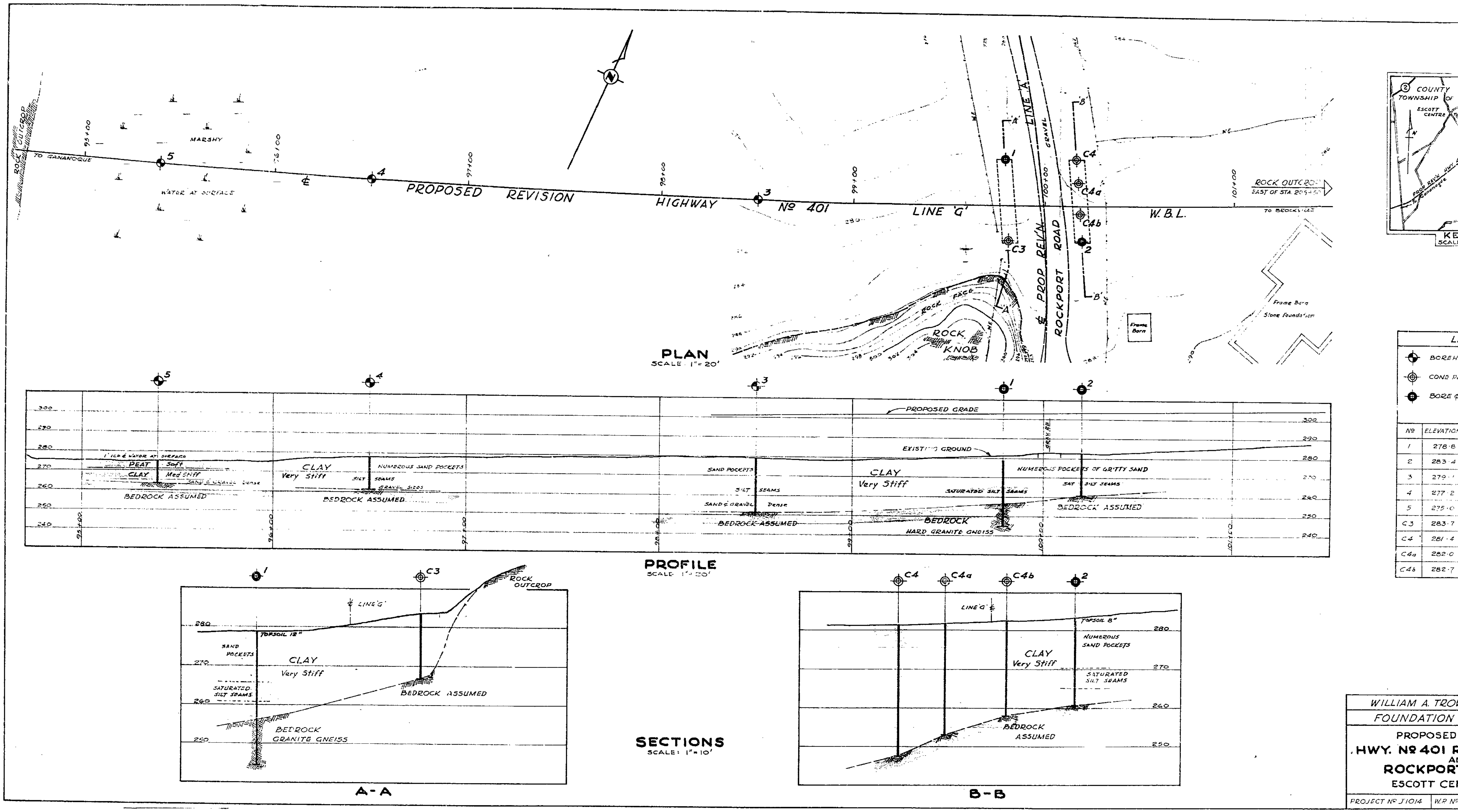
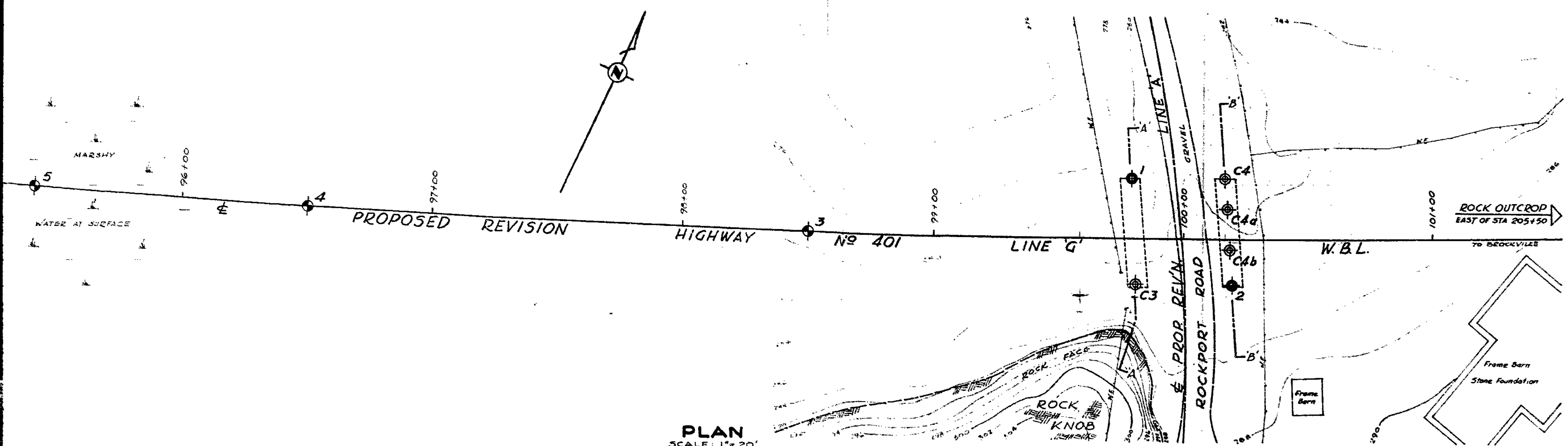
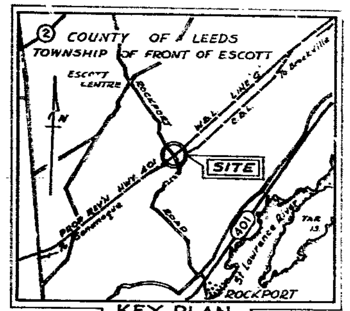
#63-F-213-C
W.P.#174-61
HWY.#401 &
ROCKPORT RD.



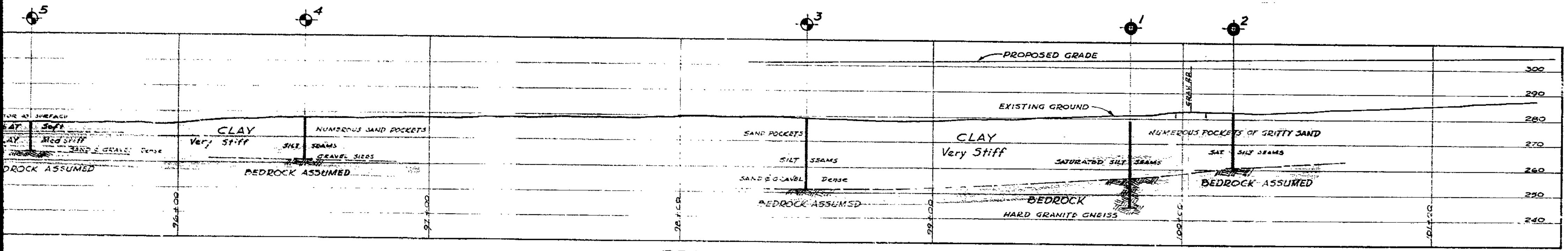
Nº	ELEVATION
1	278·8
2	283·4
3	279·[illegible]
4	277·2
5	275·0
C3	283·7
C4	281·4
C4a	282·0
C4b	282·7



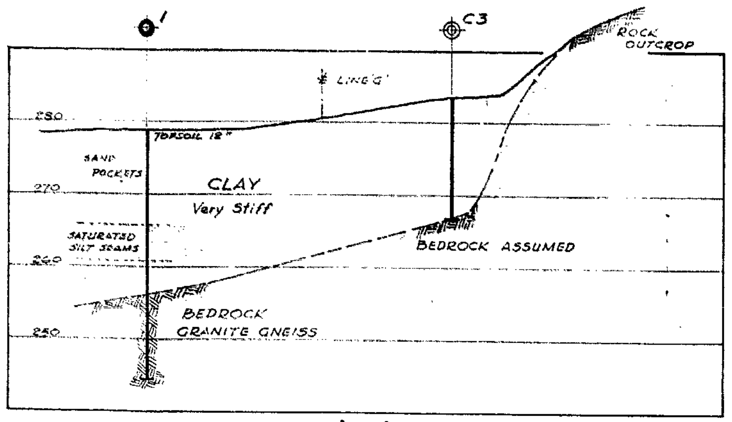
PLAN
SCALE: 1" = 20'



KEY PLAN
SCALE: 1 IN. = 0.8 MI.

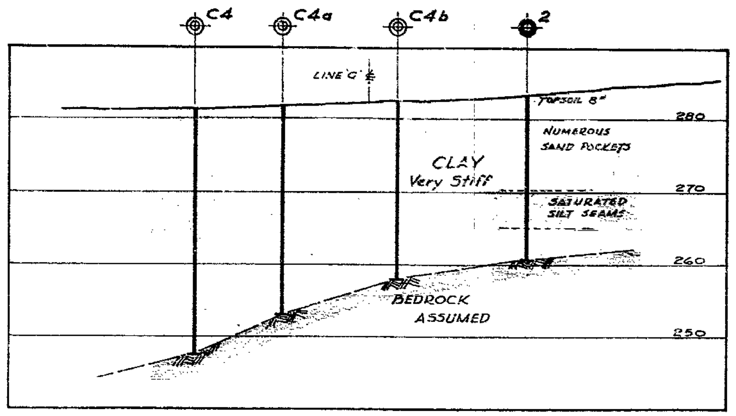


PROFILE
SCALE: 1" = 20'



A - A

SECTIONS
SCALE: 1" = 10'



B - B

LEGEND

- BOREHOLE
- CONE PENETRATION HOLE
- BORE & CONE HOLE

Nº	ELEVATION	STATION	OFFSET
1	278·8	99+79	24' LT.
2	283·4	100+20	18' RT.
3	279·1	98+50	£
4	277·2	96+50	£
5	275·0	95+40	£
C3	283·7	99+81	18' RT
C4	281·4	100+17	24' LT
C4a	282·0	100+18	12' LT
C4b	282·7	100+19	4' RT.

WILLIAM A. TROW & ASSOCIATES LTD.

FOUNDATION INVESTIGATION

PROPOSED CROSSING
HWY. Nº 401 REV'N. W.B.L. LINE 'G'
AND
ROCKPORT ROAD LINE 'A'
ESCOTT CENTRE, ONTARIO

PROJECT Nº J1014	W.P. Nº 174-61	DATE DEC. 62	DWG. 1

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. McCombie.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials and Research Division.

January 25, 1963.

FOUNDATION INVESTIGATION REPORT BY -
Wm. A. Trow & Associates, Limited,
W.B.L. Hwy. #401, Rockport Road,
W.P. 174-61 - Dist. #8.

Attached, we are forwarding to you the abovementioned report submitted by the Consultant, Wm. A. Trow and Associates, Ltd.

We are in agreement with the conclusions and recommendations contained in the Consultant's report, and believe this information will prove adequate for your future design work.

Should there be any queries in connection with this project, please do not hesitate to contact our Office.

AGS/MdeF
Attach.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
J. Ford
E. A. Cash
J. E. Gruspier
T. J. Kovich
J. Roy
E. R. Saint
F. Norman
A. Watt

Foundations Office
Gen. Files.

Materials and Research Division

December 6, 1962.

William A. Trow & Associates, Ltd.,
1950 Jane Street,
Weston, Ontario.

Attention: Mr. Wm. A. Trow.

Re: W.P. 174-61, Hwy. 401, Westbound Lane,
Underpass at Twp. Rd. to Rockport.

W.P. 60-62, Hwy. 2, Johnstown Creek at Johnstown.

W.P. 99-61, Hwy. 42, Glen Elbe Bridge, 2 Mi. W. of Portbton.
-- District No. 8, Kingston. --

Dear Sir:-

Please consider this your authority to carry out foundation investigations at the above sites. Plans and profiles were provided to your representative, Mr. Wm. Trow, on December 4, 1962.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Fourteen copies of each completed foundation report with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to January 15, 1963. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Charges for the work performed will be in accordance with your Schedule of Rates, dated November 19, 1962, and invoices to be addressed to the attention of the undersigned.

Note:- As Ottawa is the nearest recognized mobilization point, payment for mobilization will be from there, as discussed with your representative.

MDS/McP

Yours very truly,

[illegible]

[illegible]

cc: Messrs. [illegible] McCombie
J. Ford
[illegible] Cash
J. [illegible] Crumpler
M. D. Smith (2)
Mrs. T. Tate
Foundations Office
Gen. Files (

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

BA 1577

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

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

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

Project: J1014 January 21, 1963

Mr. A. Rutka, P.Eng.,
Materials & Research Engineer,
Materials & Research Section,
Department of Highways of Ontario,
Parliament Buildings, Toronto

W.P. 174-61-2

Attention: Mr. A.G. Stermac

Re: Foundation Conditions - Proposed Overpass
Hwy. 401 W.B.L. - Rockport Road

Dear Sirs:

We have completed an investigation of subsoil and bedrock conditions at this crossing of Rockport Road. The field exploratory program included borings made both at the bridge site and at representative locations along the route of the western embankment approaches to this bridge. Since no foundation or general soil mechanics problems were revealed by this study, we shall take the liberty to be brief in our submission to you.

Our observations and conclusions arising out of this survey briefly are as follows:

1) The subsoil along this route consists, for the most part, of very stiff to hard marine clay, which is quite competent to support the weight of the bridge and its approach fill in an economical manner. At deeper levels it contains wet silt seams and this material tends to reduce the strength of the clay to some extent.

2) Bedrock, consisting of very hard granite gneiss, underlies the clay at depths ranging from 17 to 34 feet below the existing ground surface. Bedrock outcrops in the form of a rock knob a few feet to the south of the proposed bridge and also 500 feet to the west and 300 feet to the east along centre line. Water, under an artesian head of about 4½ feet, was encountered at the rock contact in hole 1 at the northwest corner of the bridge. This hole was sealed off just above this level with bentonite. Some sand and gravel overlies bedrock along the highway route west of the bridge.

3) Support of the bridge structure on simple footings bearing at a minimum depth of 3 feet below the present ground surface is recommended. A frost cover of at least 4 feet of soil must, of course, be provided. The safe net bearing value to apply at this level is 8000 psf and the settlement resulting from this footing stress will be much less than 1 inch. The alternative foundation proposal is to use piles end-bearing on the hard bedrock. The safe loading for a pile will equal its permissible structural capacity when considered as a short column. The footing scheme is to be preferred, however, since it essentially eliminates the possibility of differential settlement between the bridge and the adjacent fill.

4) Any settlement at the bridge structure will be due almost entirely to the weight of the adjacent highway fill. According to computations in the report, the movement will be of a very small order, and certainly of a magnitude less than 2½ inches.

5) The clay under the embankment approaches to the bridge will remain quite stable under the weight of the approximately 22 feet of fill. However, in a marshy area between Sta 95+00 and 96+00 about 3 feet of very soft peat must be removed and, of course, the ground must be drained. There is a thin stratum of soft to firm clay, well interbedded with sand and silt seams, underlying the peat in this marshy area. The measured undrained shear strength of this deposit was about 680 psf, which value is becoming borderline with regard to stability under an embankment height approaching 22 feet. This matter is discussed in the report, and it is concluded that the clay is strong enough to support the proposed loading safely.

6) A sluggish, very shallow creek passes across the highway route in the vicinity of Sta 95+00 to Sta 96+00. It will be necessary to provide for this water by the installation of a culvert.

SITE AND PROJECT

The site of this crossing lies a few hundred feet to the north of the proposed bridge for the east bound lane of Hwy. 401, which was considered in our report of November 22nd, 1962. The terrain consists of relatively flat and almost marshy ground with abrupt outcrops of bedrock, either in the form of isolated knobs or of rugged, more continuous rock forms. The bridge site lies near the east end of a long stretch of low-lying ground which is marshy and covered with about 1 foot of water in a low section about 450 feet to the west of the road crossing. This low area lies in the path of a shallow sluggish creek which drains this marshy land.

The west bound lane of Hwy. 401 crosses this poorly drained area in the form of an earth embankment approximately 22 feet high. The bridge structure will have a span of 38 feet, and therefore a simple, single span bridge probably will be utilized at this crossing.

SUBSOIL AND INVESTIGATION PROCEDURE

The descriptions of the subsoil at each test location are given in the borehole logs, Dwgs. 2 to 7 of this report, and in the interpreted stratigraphies of Dwg. 1.

As indicated in the opening paragraphs of this report, the predominant soil type along the highway route is a very stiff to hard marine clay. Some undrained shear strength measurements were performed on this material in order to obtain some basis for estimating safe bearing values and for appraising embankment stability. In the bridge area, the clay within the significant depth below footing level was found to have an undrained strength ranging from 3200 to 5700 psf, and its moisture content lies very close to the plastic limit. Tests made in the lower, weaker levels in holes 3 and 4 to the west of the bridge, indicate strengths of 2300 psf and 1300 psf respectively. A much lower strength of 680 psf was measured on a sample of clay from hole 5, but this result is believed to be too low, because the soil contained many sand and silt seams. In any event, the clay layer at this location is only 6 feet thick, and it is underlain by dense, medium to coarse sand.

Some layers of medium to coarse sand above bedrock were also noted in holes 4 and 5 to the west of the bridge. Bedrock consists of very hard granite gneiss and drilling into it was very difficult. A slight artesian flow was noted in hole 1 after drilling into rock. It is felt that this water originates at the contact between the clay and bedrock.

The borings were made using wet sampling methods and the hole was cased with BX pipe to full depth. In most instances the samples were recovered by driving a 2 inch O.D. split spoon into the ground, but at some levels undisturbed shelby tubes were obtained. The energy used in driving the sampling equipment was 350 ft. lbs. per blow.

FOUNDATIONS

As indicated in the opening remarks, simple footings would seem to be the most suitable foundation medium for this structure, considering the small span involved, the relatively heavy weight of approach fill and the high strength of the clay. The safe net bearing value to apply to a clay soil is computed from the expression:

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

where C is the undrained shear strength; an average value of at least 4000 psf within the significant footing depth is suggested from the tests in hole 1.

N is a bearing capacity factor approximately equal to 6 for the footing shape and depth applicable for this structure

F=3 is the suggested factor of safety to keep settlement within tolerable limits. This safety factor for controlling settlement is usually a more important consideration in the design of building footings than it is for simple span bridges bounded by high fill.

Solving in this expression for this project, q, is determined to be equal to 8000 psf. This pressure can be applied at any level between depths of 3 and 7 feet from the present ground surface.

No consolidation tests have been performed for this project, since very little settlement is expected in the very stiff clay under this site. However, some estimate or forecast of foundation movement will be useful in order to determine if a problem exists. This can be obtained from the expression for consolidation settlement:

$$S = H \, Mv \; p$$

where: H is the thickness of compressible clay below footing level. The depth of clay varies at this site, as indicated in Table 1.

Mv is the modulus of compressibility which must be determined from a consolidation test. A value of 0.005 sq. ft. per kip is estimated to be quite conservative for this clay

p is the effective fill pressure transmitted into the clay

For a fill height of 21 feet, p will be approximately 2700 psf. At the bridge abutment, it can be shown that little more than half of this pressure will be effective below footing level. Conservatively taking an average pressure of 1500 psf, the magnitude of consolidation settlement has been computed in Table 1 assuming footings bearing at El 276 feet, which is 3 feet below the surface at the hole 1 location. Reference to this table shows that the settlements will range from 1 to 2½ inches, depending upon the depth of clay. Elastic compression will increase this estimate to some extent.

These computations are considered to be conservative, principally because the chosen value of Mv is believed to be much too high and the very stiff clay crust will tend to spread load out more than is assumed in the estimate for p. Nevertheless, there will be a tendency for some slight tilting of the bridge toward the north since the clay thickness increases in that direction. This minor differential movement can be reduced to some extent by adjustments in footing level to decrease the differential thickness of underlying clay. It is unlikely that this movement will be noticed if the bridge is supported on footings.

In view of the very high strength of the clay, no stability problem exists at this site. Theoretically, the shear strength of the clay is borderline in the marshy area between Sta 95+00 and 96+00, but this test result is believed to err on the low side and, in any event, the numerous sand and silt seams interbedded in it should permit rapid drainage and consolidation of the clay to a higher strength.

Assuming that relatively rigid abutments will be utilized in this bridge, and that the backfill, immediately behind it, will consist of granular material, the horizontal pressure exerted against the abutment walls will be the "at rest" earth pressure. The appropriate earth pressure coefficient will be 0.4 approximately. No differential settlement should occur between the fill and the abutment, and therefore no wall friction should be generated.

The earth pressure will be resisted in varying degree by the bridge deck, depending upon the rigidity of the connection to this member. The full undrained strength of the very stiff clay should be generated under the base of the footing immediately after embankment load is applied against the abutment. The long term sliding resistance under the footing will be equal approximately to $N \tan \phi'$ where N is the normal abutment load on the soil and ϕ' is the effective angle of internal friction of the clay. It is conservative to neglect any effective cohesion under the footing and to assume a value for $\tan \phi' = 0.35$. If the sliding resistance is insufficient with these conservative assumptions, a key can be dug into the clay to increase the support. If it is considered necessary or critical, this earth pressure problem could be examined in more detail after specific design proposals have been prepared.

We shall be pleased to discuss any aspect of this report, if you consider this to be desirable after you have reviewed the foregoing information.

Yours very truly,

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

William A. Trow, P.Eng.

WAT/gc
Encls.

TABLE 1

COMPUTATION OF SETTLEMENTS OF BRIDGE STRUCTURE

Hole No.	1	Cone 3	2	Cone 4
Approx. Thickness of Clay in Inches	240	120	180	342
Settlement = .0075 H	1.8	0.9	1.35	2.5

<u>Assumptions:</u>

1) Footings at El 276 feet.

2) Modulus of compressibility Mv = .005 sq. ft./kip - (conservative).

3) Fill height = 21 feet; average pressure p in clay = 1500 psf (conservative)

4) Settlement S = H Mv p

where H = thickness of clay below El 276 ft. or footing level.

TABLE 2

SUMMARY OF CONE TEST RESULTS IN BRIDGE AREA

Cone No.	C3	C4	4a	4b
Surface El	283.7	281.4	282.0	282.7
Depth		Penetration Resistance Blows/ft. 350 ft.lbs. Energy		
0-1	9	2	Cone point only driven	Cone point only driven
2	4	2		
3	5	4		
4	5	9	Refusal and bounce at 29.1 ft.	Refusal and bounce at 25.0 ft.
5	13	15		
6	14	11		
7	16	12		
8	16	16		
9	20	16		
10	29	17		
11	48	21		
12	42	21		
13	53	21		
14	56	19		
15	57	22		
16	54	21		
17	53 for 10"	24		
18	(bounce)	21		
19		17		
20		14		
21		15		
22		15		
23		15		
24		16		
25		15		
26		21		
27		21		
28		19		
29		18		
30		27		
31		47		
32		53		
33		29		
34		38		
		5 for 1" (bounce)		

Looking East Along C.L., Drill on B.H. 4

Looking West Along C.L., Drill on B.H. 4

Looking East Along C.L., Drill on B.H. 4

Looking West Along C.L., Drill on B.H. 4

Looking West Along C.L., Drills on
B.H. 1 (right) and B.H. 3 (centre background)

Looking South Along Rockport Rd.
Drill on B.H. 3

Looking West Along C.L., Drills on
B.H. 1 (right) and B.H. 3 (centre background)

Looking South Along Rockport Rd.
Drill on B.H. 3

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No 3

PROJECT No J1014

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—x—
- 2" DIA. CONE ———

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) +S

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

ATTERBERG LIMITS
- LIQUID LIMIT —o
- PLASTIC LIMIT ⊢

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE No. 1

PROJECT Proposed Crossing, Rockport Rd. 'Line A' & Prop. Hwy.401, WBL, 'Line G'

LOCATION Rockport, Ont.

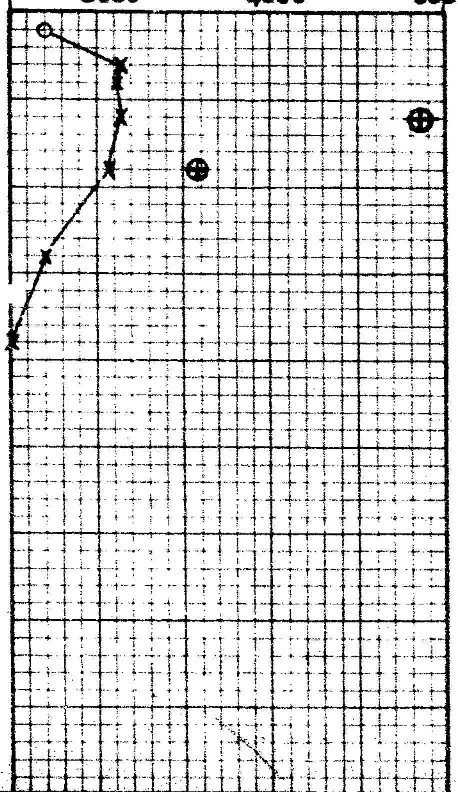
HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 278.8 ft.

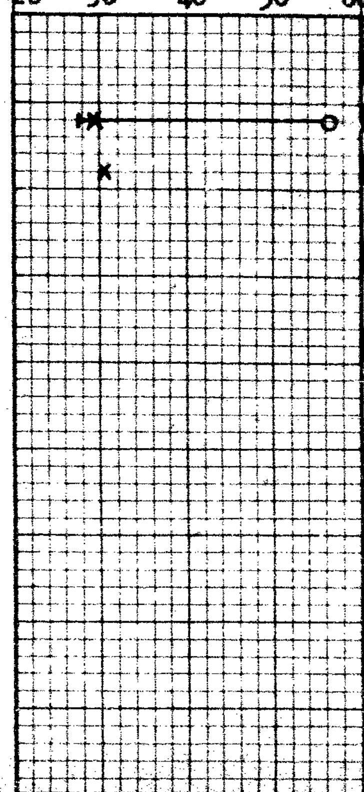
DATUM See Dwg. 1.

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	12 ins. topsoil	278.8	0	SS1	
	MARINE CLAY-very stiff, brown, with occas. sulphide specks & numerous tiny pockets of gritty sand.			TW2	
				TW3	122
				TW4	121
	-wet saturated silt seams 13 ft. to 18 ft.		10	TW5	
				TW6	Levered
			20	L.	
	BEDROCK - Hard granite gneiss. 100% recovery	256.1			
			30		
	End of Hole	244.6			
			40		

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT: 20, 40, 60, 80
SHEAR STRENGTH P.S.F.: 2000, 4000, 6000



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT: 20, 30, 40, 50, 60



Notes:
1) Boring cased with BX pipe to full depth.
2) Water overflowing casing at El 279.3 ft. after boring completed. (No evidence of water in clay). Water head = El 283.1 Hole sealed with bentonite just above rock.
3) Vane strength in excess of 2100 psf at 16 & 21 ft. No test attempted above these levels.

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SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

DRAWING No 3

PROJECT No J1014

LEGEND

BOREHOLE No. 2

PROJECT Proposed Crossing, Rockport Rd. 'Line A' & Prop. Hwy.401,WBL,'Line C'

LOCATION Rockport, Ont.

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 283.4 ft.

DATUM See Dwg. 1.

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—
- 2" DIA. CONE ————

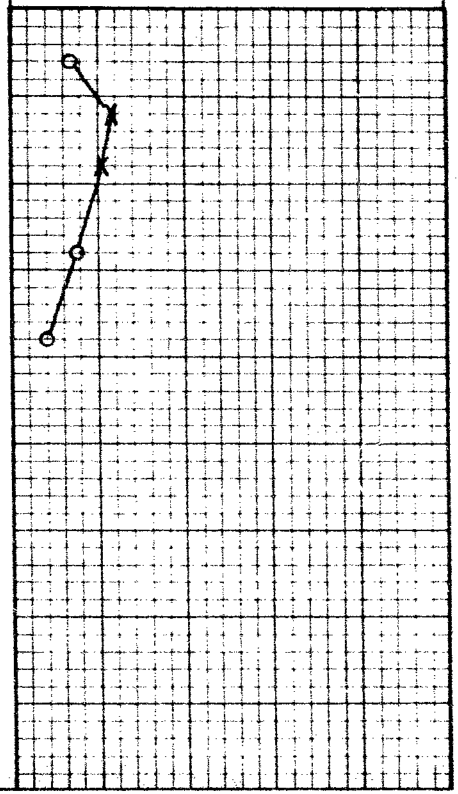
SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) $+^S$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS
- LIQUID LIMIT —o
- PLASTIC LIMIT ⊢

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB BLOWS/FT (20, 40, 60, 80) / SHEAR STRENGTH PSF	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	8 ins. topsoil	283.4	0				
	MARINE CLAY-very stiff, brown, occas. sulphide specks, numerous tiny pockets of gritty sand.					SS1	
						TW2	
			10			TW3	
	-wet silt seams from 13 ft. to 18 ft. approx. Irregular stratification of light grey silty clay & dark grey fissured clay in sample 5.					SS4	
	ASSUMED BEDROCK CONTACT (Wash rods bounce)		20			SS5	
	End of Hole	260.7					
			30				
			40				



Notes:
1) As in hole 1.
2) W.L. after casing withdrawn = 0.6 ft. After 18 hrs. = 1.6 ft. = 281.8 ft.
3) Vane strength in excess of 2100 psf at 20 ft.

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SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

DRAWING No. 4

PROJECT No. J1014

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—
- 2" DIA. CONE ————

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS
- LIQUID LIMIT —o
- PLASTIC LIMIT ⊢—

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE No. 3

PROJECT Proposed Crossing, Rockport Rd. 'Line A'

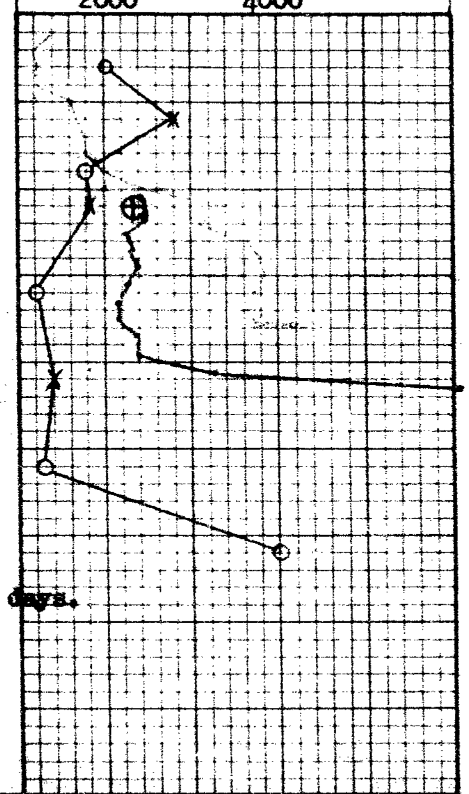
LOCATION Rockport, Ont. & Prop. Hwy.401,WBL,'Line G'

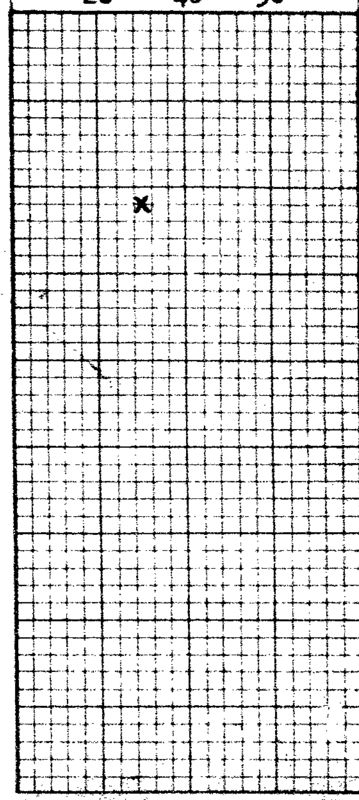
HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 279.1 ft.

DATUM See Dwg. 1.

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT (20, 40, 60, 80) / SHEAR STRENGTH P.S.F. (2000, 4000)	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT (20, 40, 50)	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
	6 ins. topsoil	279.1	0				
	MARINE CLAY-very stiff, brown, occas. sulphide specks, numerous tiny pockets of silty sand. Silt seams below about 13 ft. Irregular stratification below 20 ft. as in hole 2. Large gravel deflects split spoon and casing below 28 ft.		10, 20			SS1, TW2, SS3, TW4, SS5, TW6	118.3
	SAND & GRAVEL-dense brown - split spoon bent in withdrawal at 31½ ft. - possibly slid down side of bedrock slope beginning at 28 ft. (Bedrock at El 251 ft. estimated)	251	30			SS7, SS8	





Notes: 1) As in hole 1.
2) With casing at 11 ft.,W.L. = 5.5 ft.after 5 days.
W.L. in casing at end of bore = 3.5 ft.
After withdrawing casing,W.L. very slowly began to rise in hole.
Bentonite seal installed in hole at depth of approx. 10 ft.
3) Vane strength in excess of 2100 psf at 17, 23 and 27 ft.
4) Cone driven 14 ft. to west of hole; refusal at 21.9 ft.; cone rod badly bent on withdrawal.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No 5

PROJECT No J1014

LEGEND

BOREHOLE No. 4

PROJECT Proposed Crossing, Rockport Rd. 'Line A' & Propo Hwy.401,WBL,'Line G'

LOCATION Rockport, Ont.

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 277.2 ft.

DATUM See Dwg. 1.

PENETRATION RESISTANCE

- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—
- 2" DIA. CONE ———

SHEAR STRENGTH

- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS

- LIQUID LIMIT —o
- PLASTIC LIMIT ⊢—

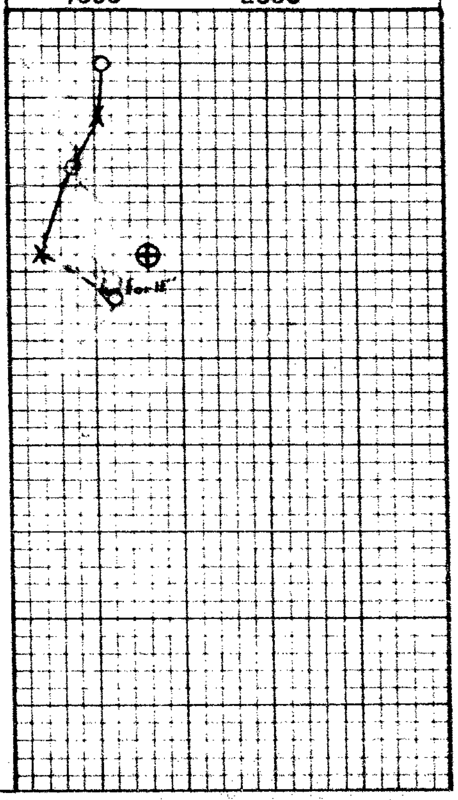
SAMPLE TYPE

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

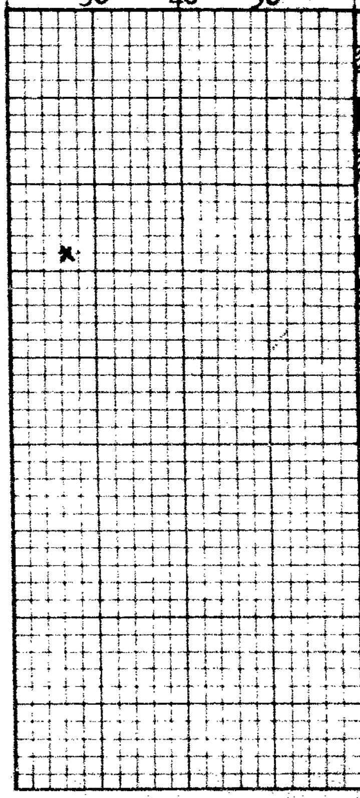
SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F
		277.2	0		
	MARINE CLAY-very stiff, brown, numerous tiny sand pockets, silt layers and med. to coarse sand partings below 8 ft., gravel sizes below about 14 ft.			SS1	
				TW2	
			10	SS3	
				TW4	127
				SS5	
	End of Bore	261.1			
			20		
			30		
			40		

Notes: 1) As in hole 1.

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT: 20, 40, 60, 80; SHEAR STRENGTH P S F: 1000, 2000



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT: 30, 40, 50



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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No 6

PROJECT No. J1014

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 2" DIA. CONE

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) +S

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE No. 5

PROJECT Proposed Crossing, Rockport Rd. 'Line A' & Prop. Hwy.401, WBL, 'Line G'

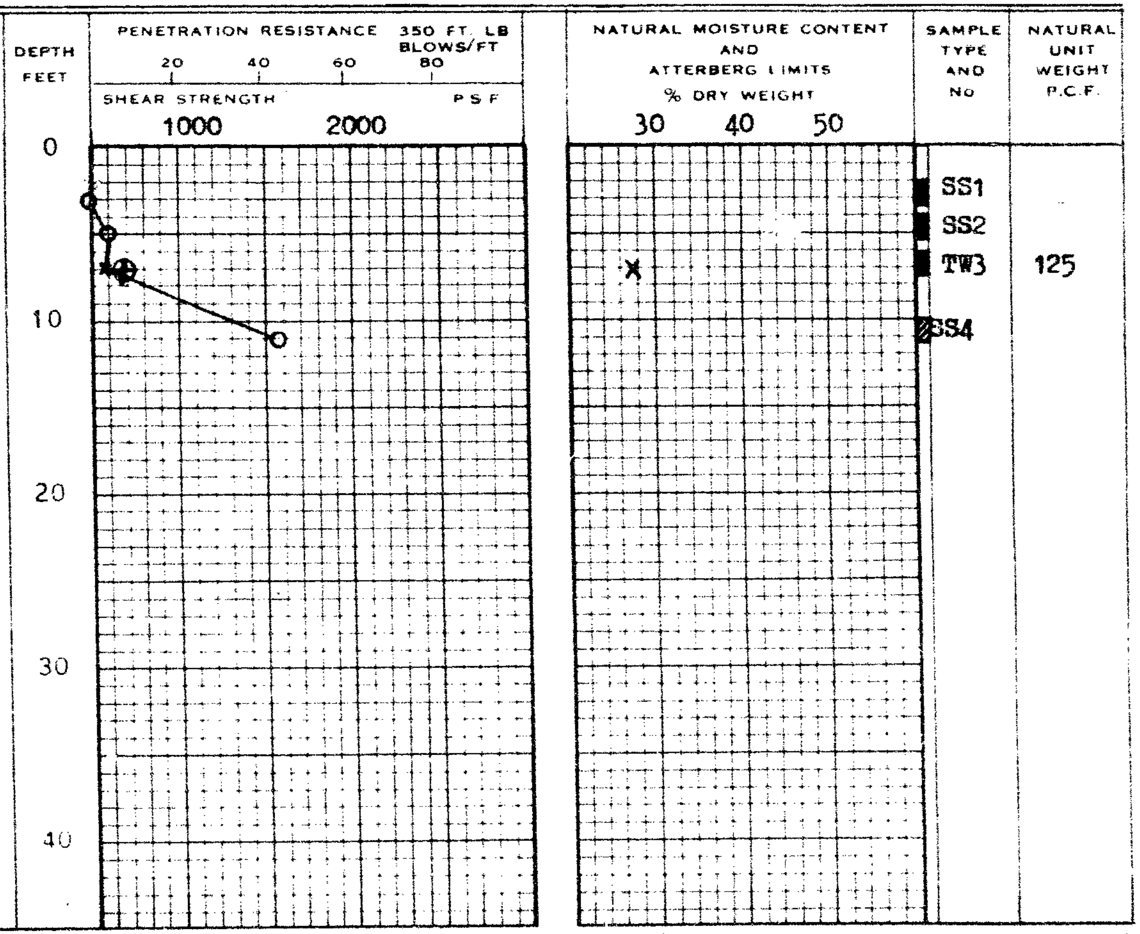
LOCATION Rockport, Ont.

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 275.0 ft.

DATUM See Dwg. 1.

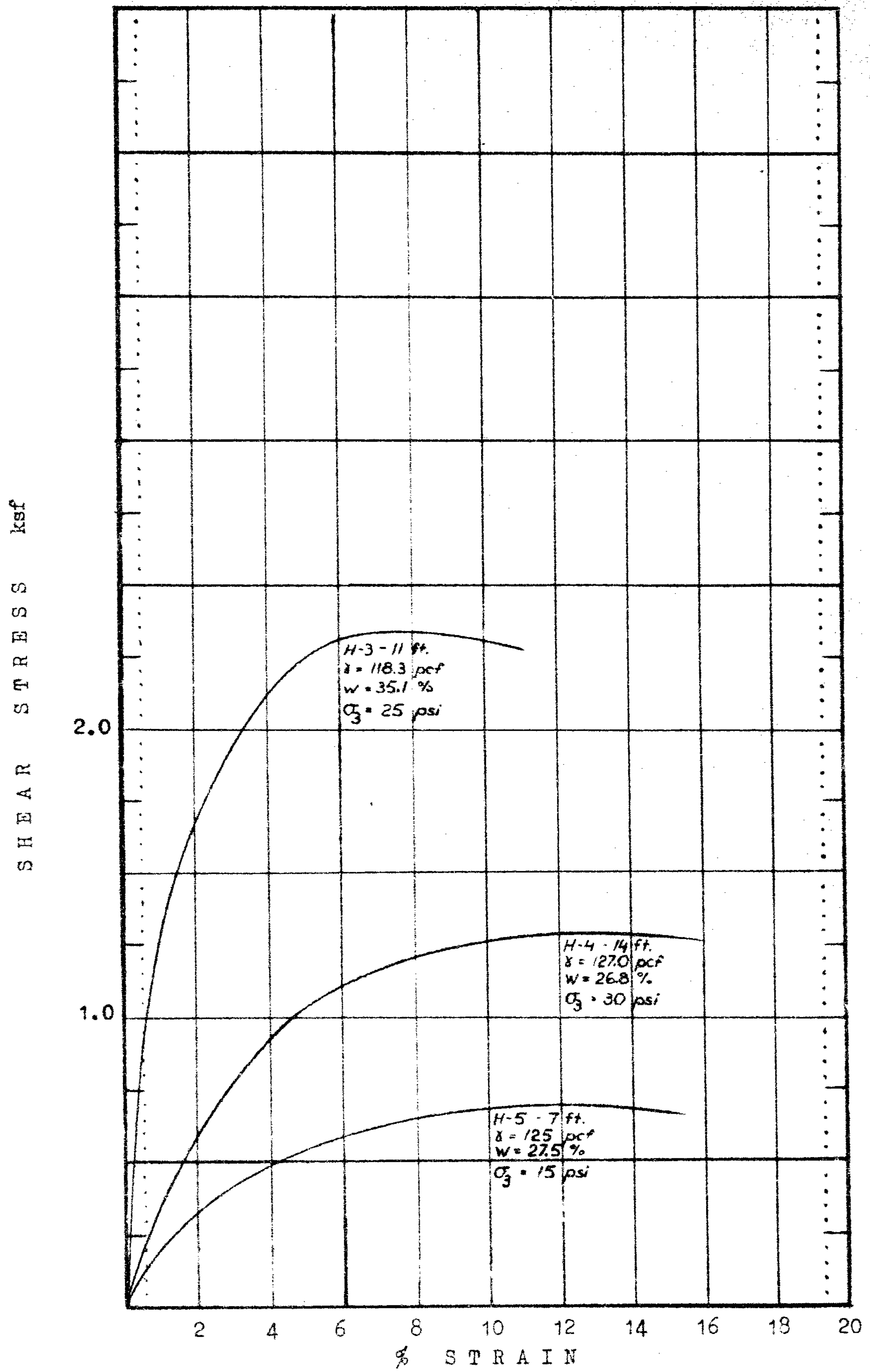
SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	In swamp	275.0	0		
	Ice and water	274.0			
	PEAT-soft, compressible	271.0		SS1	
	CLAY-med. stiff, grey, numerous silt layers, medium to coarse sand partings.	265		SS2	
				TW3	125
	SAND & GRAVEL-med. to coarse, dense, split spoon bent on inclined rock at 11½ ft.	263.5	10	SS4	
	End of Hole				



Notes: 1) As in hole 1.

2) Undrained triaxial test result performed on clay from this hole believed to be unduly conservative because of numerous wet sand seams and pockets in the soil.

PROJECT NO. J1014 DRAWING NO. 7



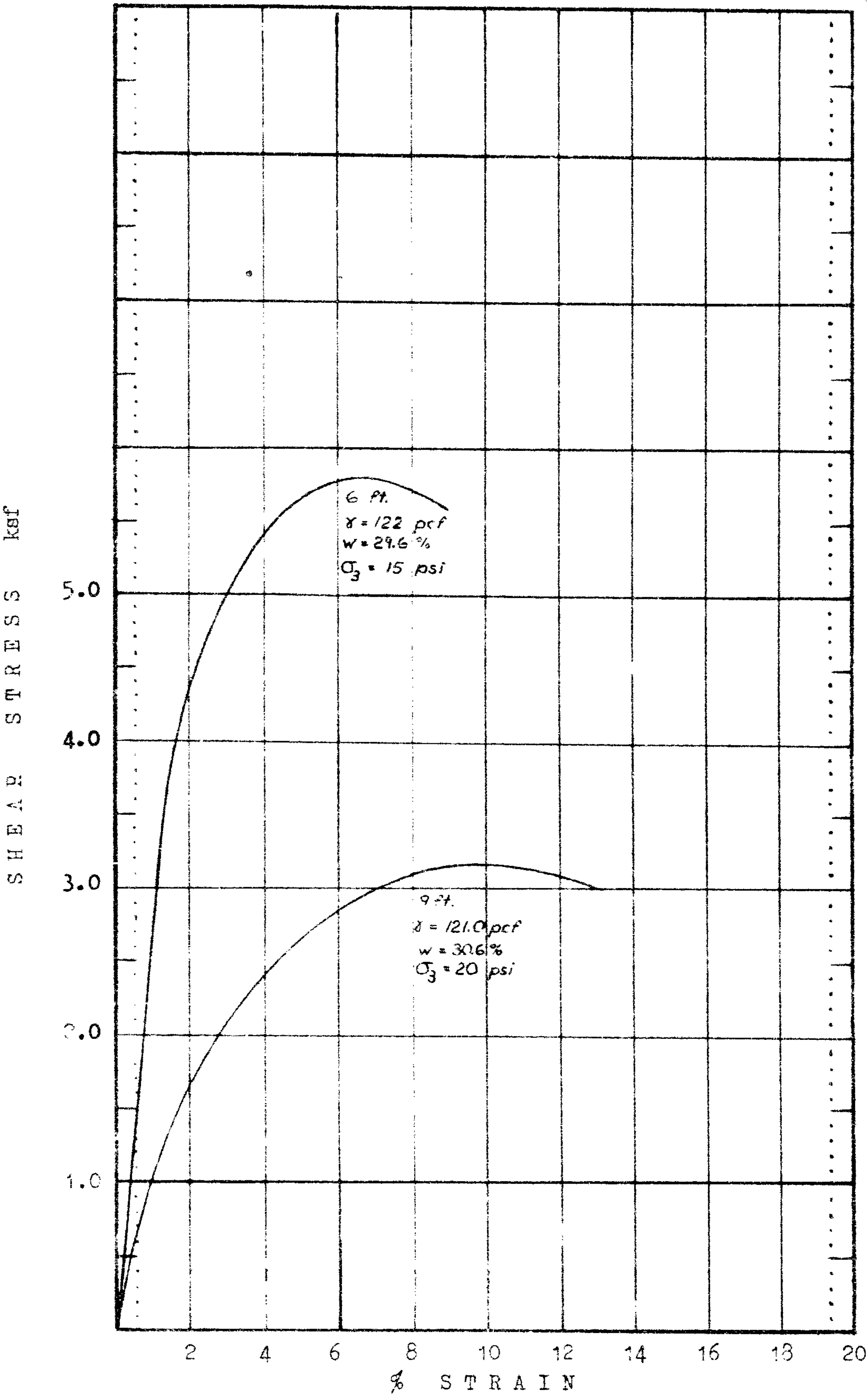
UNDRAINED TRIAXIAL TEST RESULTS

HOLES 3, 4 & 5

WILLIAM A. TROW AND ASSOCIATES ROCKPORT OVERPASS

PROJECT NO. J1014

DRAWING NO. 8



UNDRAINED TRIAXIAL TEST RESULTS

WILLIAM A. TROW AND ASSOCIATES

HOLE 1
ROCKPORT OVERPASS