#62-F-243C

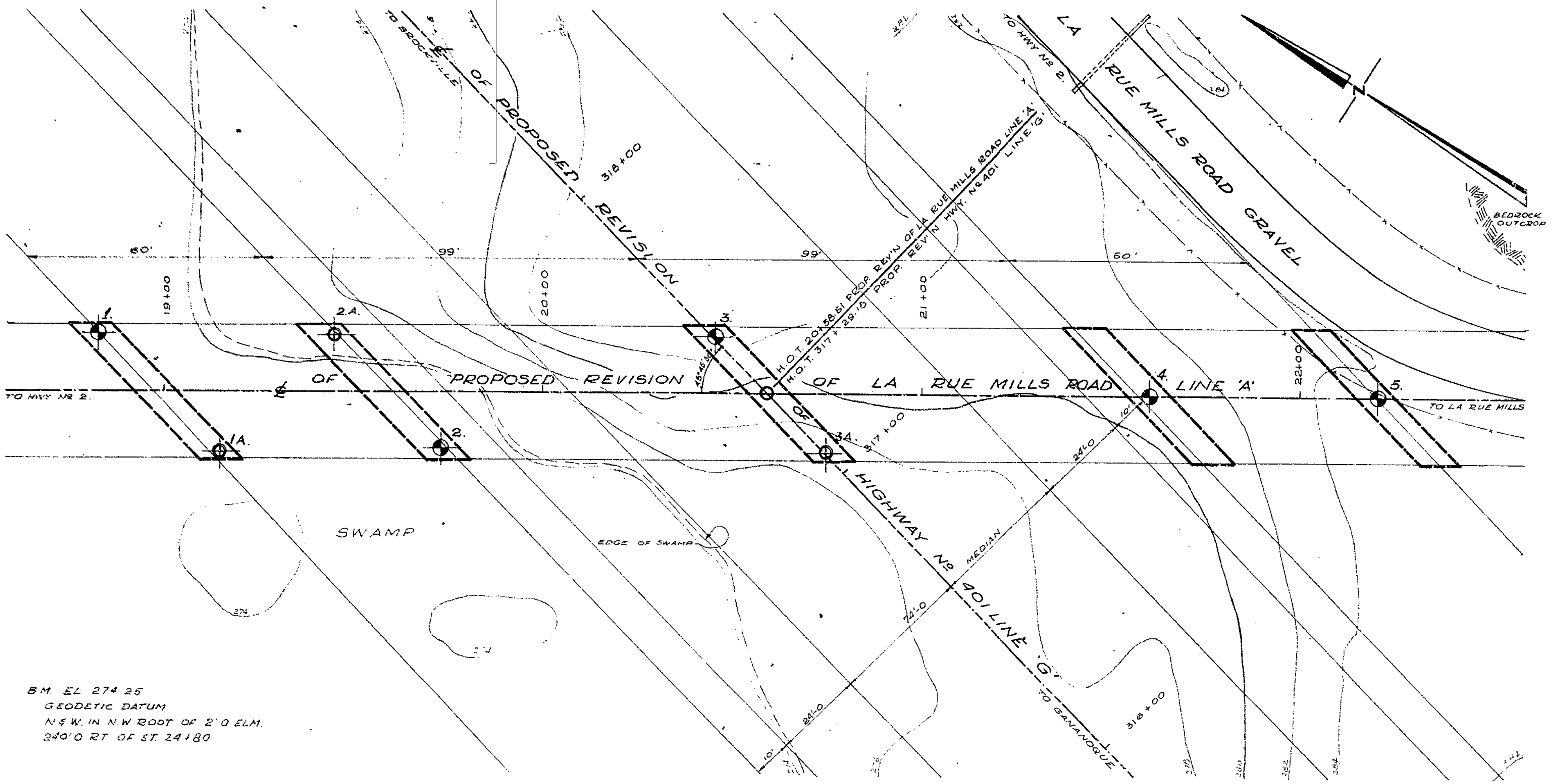
W.P. #175-61

Hwy #401

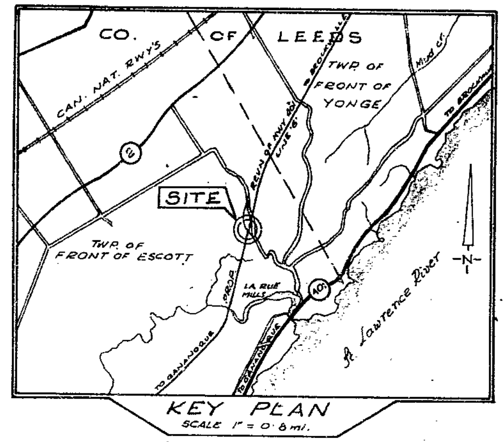
LA RUE MILLS

ROAD

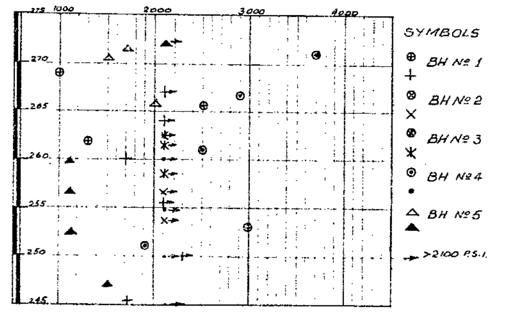
UNDERPASS



BORE HOLE LOCATION PLAN
SCALE 1" = 20'



KEY PLAN
SCALE 1" = 0.6 mi.



RECORD OF UNDRAINED SHEAR STRENGTH WITH DEPTH

LEGEND

BORE HOLE

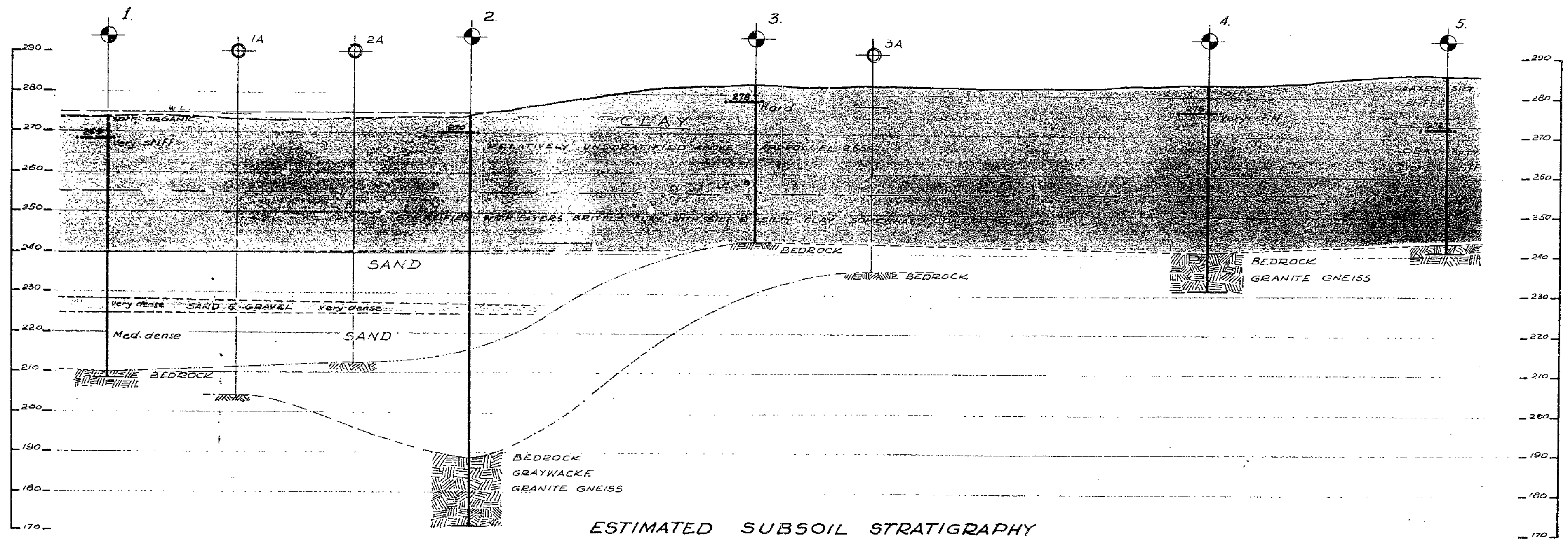
PROBE

ASSUMED OUTLINE OF BEDROCK 15 Ft RIGHT OF ℄

ASSUMED OUTLINE OF BEDROCK 15 FT LEFT OF ℄

ELEV. BEARING ELEV. WHERE TO APPLY 4 Ksf.

HOLE	ELEVATION	STATION	DIST FROM ℄
1	274·8	18 + 83	15 LT
1A.	274·8	19 + 15	15 RT
2.	274·8	19 + 74	15 RT
2A	275·5	19 + 45	15 LT
3.	283·0	20 + 45	15 LT
3A	276·9	20 + 74·5	15 RT
4.	283·0	21 + 60	℄
5.	285·1	22 + 20	℄



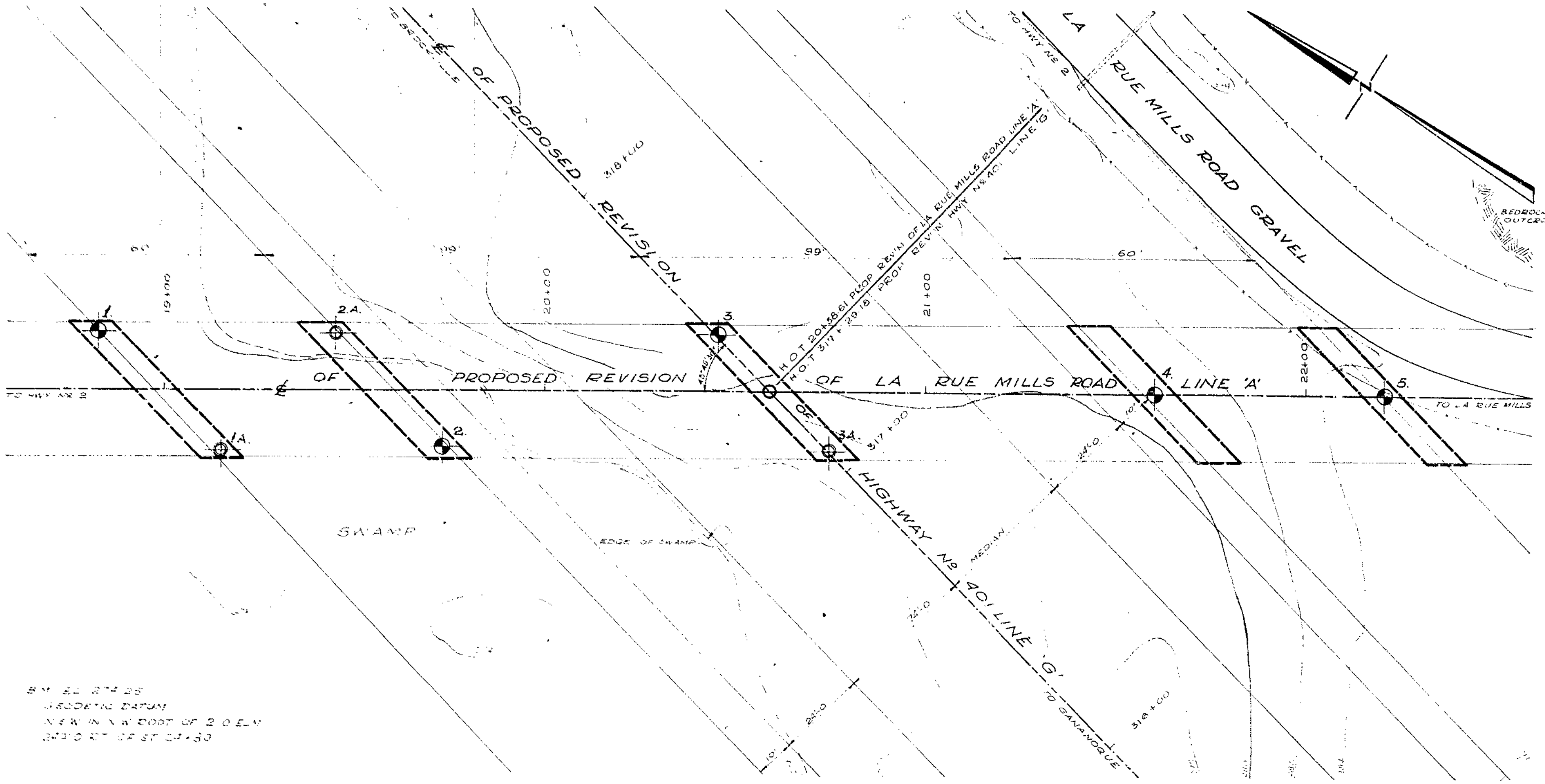
ESTIMATED SUBSOIL STRATIGRAPHY
SCALE 1" = 20'

W. A. TROW & ASSOC. LTD.

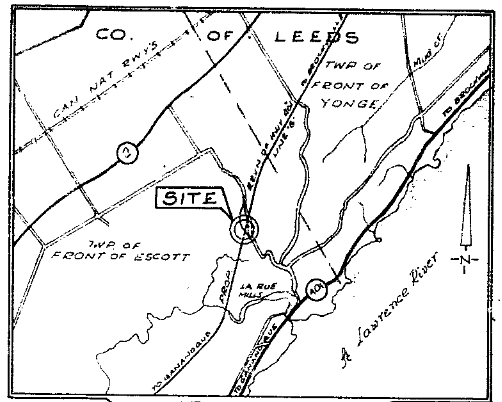
FOUNDATION INVESTIGATION

PROPOSED CROSSING AT HIGHWAY № 401 LINE 'G' & LA RUE MILLS ROAD LINE 'A'

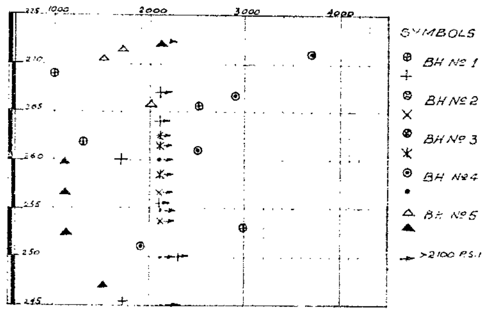
PROJECT № T. 968	W.P. № 175-61.	DATE NOV. 1962	DWG. 1.



BORE HOLE LOCATION PLAN
SCALE 1" = 20'



KEY PLAN
SCALE 1" = 0.6 mi.



RECORD OF UNDRAINED SHEAR STRENGTH WITH DEPTH

LEGEND

BORE HOLE

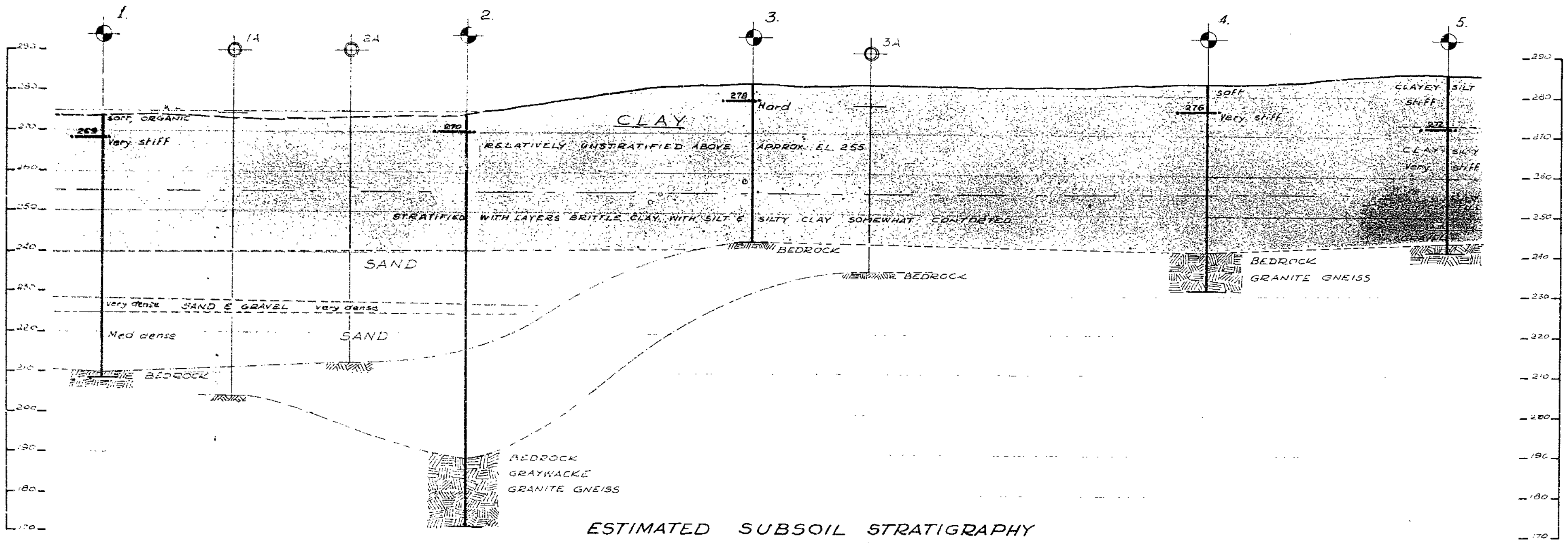
PROBE

ASSUMED OUTLINE OF BEDROCK 15 Ft RIGHT OF ℄

ASSUMED OUTLINE OF BEDROCK 15 FT LEFT OF ℄

ELEV. BEARING ELEV. WHERE TO APPLY 4 Ksf.

HOLE	ELEVATION	STATION	DIST FROM ℄
1	274 8	18 + 83	15 LT
1A.	274·8	19 + 15	15 RT
2	274·8	19 + 74	15 RT
2A	275·5	19 + 45	15 LT
3	283 0	20 + 45	15 LT
3A	276 9	20 + 74·5	15 RT
4	283·0	21 + 60	℄
5.	285·1	22 + 20	℄



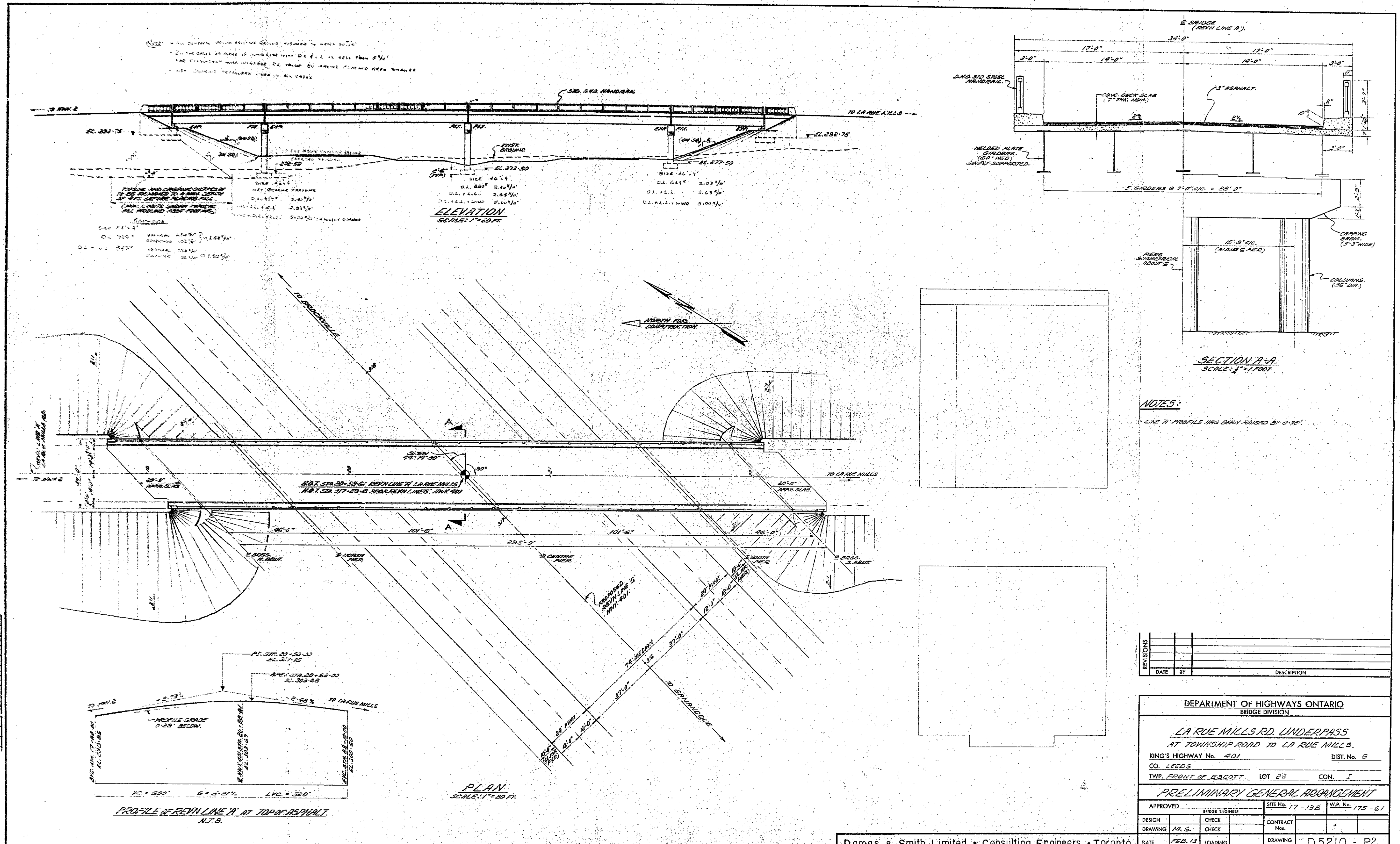
ESTIMATED SUBSOIL STRATIGRAPHY
SCALE 1" = 20'

W. A. TROW & ASSOC. LTD.

FOUNDATION INVESTIGATION

PROPOSED CROSSING AT HIGHWAY Nº 401 LINE 'G' & LA RUE MILLS ROAD LINE 'A'

PROJECT Nº T. 968	W.P. Nº 175-61.	DATE NOV. 1962	DWG. 1.



REVISIONS		
DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

LA RUE MILLS RD. UNDERPASS
AT TOWNSHIP ROAD TO LA RUE MILLS.

KING'S HIGHWAY No. 401 DIST. No. 8
CO. LEEDS
TWP. FRONT OF ESCOTT LOT 23 CON. I

PRELIMINARY GENERAL ARRANGEMENT

APPROVED	BRIDGE ENGINEER			SITE No. 17-138	W.P. No. 175-61
DESIGN		CHECK		CONTRACT Nos.	
DRAWING	M. S.	CHECK			
DATE	FEB. 13 1963	LOADING		DRAWING No.	D5210 - P2

Damas & Smith Limited • Consulting Engineers • Toronto

23-67-21

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: J968

November 21, 1962

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

Attention: Mr. A.G. Stermac, P.Eng.

Re: Foundation Conditions
La Rue Mills Road Underpass
Hwy. 401, W.P. 175-61

Dear Sirs:

In conformance with your written authorization of October 18, 1962, we have completed an investigation of foundation conditions at this county road crossing of Highway 401.

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

1) The site is underlain by deep deposits of stiff modified lacustrine clay and medium dense sand. The sand underlies the clay in a deep bedrock depression under the north half of the highway crossing. Bedrock, generally consisting of very hard granite gneiss, ranges in depth from approximate El 243 feet under the south half of the bridge site to El 188 feet under the northwest portion of the highway crossing. The subsoil stratigraphy and estimated bedrock configuration along the length of this bridge site are indicated in Dwg. 1 and in the data sheets for the borings.

2) Two foundation proposals are indicated for this underpass structure. One involves the use of simple footings exerting a net bearing pressure of 4000 psf to the soil. The other proposal is to support the structure either on H piles driven to bedrock or, - under the deep north half of the crossing, - to drive Franki type compression piles into the upper levels of medium sand existing below El 240 feet.

The permissible load for an H pile on bedrock will be equal to its safe structural capacity when considered as a short column. The maximum bearing strength of the bedrock should be well in excess of 13,000 psi, the observed load test failure stress for H piles bearing on Dundas Shale in the Port Credit area.

The safe load of a Franki pile with 3 foot diameter base should at least be equal to 100 tons. This compression pile scheme probably is less acceptable, since it requires the use of preaugered holes to avoid disturbing the clay.

3) No embankment stability problem is envisaged at this highway crossing. The shear strength of the clay is more than adequate to support the highest embankment fill load safely.

4) Approximately 5 inches of long term settlement has been computed under the weight of fill at the north abutment; the embankment height is greatest at this location. Experience indicates that the long term movement probably will be much smaller than this estimate. The settlement of the south approach will be less than this value because the height of approach fill is not so great.

5) We see no objection to the use either of the multi span open structure indicated in the preliminary bridge site location plan or to a single span structure. Smaller total and differential settlements should occur with the single span closed abutment scheme.

The factual information and soil mechanics reasoning, which form the bases for these comments and recommendations are considered under the sections that follow.

SITE

The site of this county road crossing of Highway 401 lies near the northeast end of an open section of pasture land. The installation of the overpass will result in a localized straightening of La Rue Mills Road, which follows a very winding path through the woods northward from Highway 2.

The north half of the crossing site is covered by brush growth and it is about 6 inches under water. Farther to the south and north the ground rises well above this marshy ground. The land to the north is well drained and is covered by pine and other similar trees. The soil exposed in this area is sand and gravel.

A bedrock outcrop exists about 100 feet northeast of Station 17+00, at the north end of the site. Massive bedrock also rises above the meadow on the east side of La Rue Mills Road and about 50 feet east of borehole 5.

A small creek traverses the north end of the site about 500 feet to the west of the crossing. It is understood that there is no flooding of consequence in this stream.

Typical views of the construction area are indicated in the attached photographs.

GEOLOGY AND SUBSOIL

According to Chapman*, the sediments in this area were laid down during the Champlain sea invasion in the valleys between granite knob outcroppings. However, the material at this site does not have the characteristic appearance of a marine deposit. Although unstratified, the clay above a depth of about 20 feet contains streaks, pockets and intrusions of fine sand or silt. At greater depths a definite stratification or varving was noted, although in some instances the individual layers of clay were distorted and tilted. Pockets of clay till were noted in the lower mass of clay.

In view of these observations, the deposit is believed to be of fresh water lacustrine origin, and probably modified by minor glacial action.

The sand below this clay mass contains some thin beds of clay and, at a depth of about 50 feet or El 225, approximately, under the north end of the site, a thin layer of well-graded sand and gravel was encountered. Gradings of the sand and of this dense well-graded sand and gravel are indicated in Dwg. 7.

A description of the subsoil at each test location is contained in the stratigraphical profile of Dwg. 1 and in the data sheets for the five borings. A record of laboratory tests is also presented on each of these sheets. Since this information is self explanatory, no purpose is served by a written repetition of it in this report.

Routine laboratory tests were performed on the clay, in order to assist in the appraisal of its bearing capacity and compressibility. In general, the results of undrained triaxial shear tests are in accord with the results of in situ vane tests or the strengths generally are of the order expected for the respective moisture and Atterberg limit measurements of each test sample.

However, the strengths obtained on samples from a depth of 13 feet in hole 5 and 6 feet in hole 1, are much lower than was expected from visual examination and from the resistance offered during sampling. It is concluded that these test results represent an underestimate of the in situ strength of the clay. The record of shear strength with depth is summarized on Dwg. 1.

One consolidation test was performed on a sample of less stiff clay taken from a depth of 13 feet in hole 1. This sample was chosen because it is located in the area of highest embankment fill and its result should provide a conservative basis for estimating settlement. Similar softer material was encountered below 20 feet in hole 5, but the embankment height is not so great in that area.

* "The Physiography of Southern Ontario" - Chapman and Putman

Bedrock at this site was proven or reached in five borings and three probed locations. It consists generally of extremely hard granite gneiss, although some ancient greywacke was found overlying this Pre-cambrian deposit in hole 2. The additional wash boring probes were in the north half of the site, since marked variations in bedrock level were noted in this area. If end-bearing piles are to be used, some indication of this variation was needed. Many more probings would be required to outline the bedrock configuration exactly. Such a program was considered to be an uneconomic undertaking for this investigation, because, even with many probings under each pier and abutment position, there would be no positive guarantee that some of the piles did not bear on the edge of a steeply inclined bedrock slope. An opinion on the satisfactory seating arrangement of the piles can best be obtained by careful examination of the refusal pattern obtained after the piles are driven.

DISCUSSION OF FOUNDATION REQUIREMENTS

a) Bearing Capacity - As indicated in the opening statements of this report, at least two foundation alternatives are available for this bridge crossing of Highway 401. One of these incorporates the use of simple footings bearing near the top of the desiccated crust of the thick clay deposit. The recommended bearing level at each test location is indicated on Dwg. 1 and the safe net bearing value to apply at this level is 4000 psf. This limiting pressure is determined from the expression:

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

where: $C = 2000$ is the estimated average shear strength within the zone of influence below the recommended bearing levels. In some locations C is greater than 2000 psf.

N is a bearing capacity factor equal approximately to 6 for the footing shape and embedment conditions applying

$F = 3$ is the recommended factor of safety for stability.

The settlement resulting from this stress application should be less than 1 inch for those footings located well clear of embankment fill. The settlement of footings buried in the embankment approaches, however, will be determined by the fill.

No particular excavation difficulty is envisaged when digging to footing level at the various pier and abutment locations. The swampy area should, of course, be drained prior to construction and even after this is done some seepage will pass through thin seams and partings of the upper layer of sand and clay material. This will occur, not only in the swampy area, but also in the vicinity of holes 4 and 5 where a high ground water table was noted. This small seepage of water can be directed to sumps in the excavation for disposal and therefore it will not hinder construction operations.

It is quite conceivable that soil of satisfactory bearing will be revealed at slightly higher levels than are indicated on Dwg. 1. In order to take advantage of this possibility, a careful examination of each excavation should be made by a soils engineer as digging operations proceed.

The other alternative for bridge support is to use piles, either end-bearing on bedrock or on a densified base in the stratum of sand below El 240 feet. In the former instance, H piles are recommended and the permissible load that may be applied to them will be determined by their safe structural capacity when considered as short columns. As indicated in the opening paragraphs, the ultimate capacity, in bearing, of the hard granite gneiss bedrock is believed to be well in excess of 13,000 psi. This was the measured load test stress on six 12BP53 piles when they began to penetrate into the Dundas Shale underlying the Port Credit waterfront.

A careful examination of the refusal pattern of H piles should be made after the driving program is complete, particularly in the deep valley section around hole 2. This refusal pattern should be the best indicator of the proximity of a pile to the edge of a buried bedrock cliff.

If desired, the use of long piles under the north half of the structure can be avoided by installing Franki-type compression piles in the sand stratum just below El 240 feet. In order to minimize disturbance to the overlying clay, the shaft should be installed in a pre-augered hole. The medium sand below the clay will be compacted to a dense condition during the formation of the Franki bulb. A very light driving energy should be used in the development of the shaft in order to avoid disturbance to the clay. The last 5 feet of pile above the sand should be driven through the clay, since otherwise, water will tend to rise in the augered hole and make concrete placement difficult.

This same method, involving caissons to rock, could be used for the southerly portions of the bridge. In all instances, it may be necessary to provide casing in the upper levels of the borings to support seams and layers of running sand.

The safe **capacity** of a compaction pile, with a 3 foot diameter bearing surface on the consolidated sand, is estimated to be at least equal to 100 tons. This estimate is confirmed in an approximate manner by the Terzaghi expression*

$$Q = A\gamma N(0.3B + D)\frac{1}{F}$$

* "Soil Mechanics in Engineering Practice", Pg. 172 - Terzaghi and Peck

where:

A is the end-bearing area in square feet of a base of diameter B = 3 feet

γ is the effective unit weight of the soil taken here to be equal to 70 pcf

N is a bearing capacity factor assumed equal at least to 100 after the sand has been compacted.

D is the depth of the bearing surface = 40 feet

F = 3 is the recommended safety factor.

2) Embankment Stability - Since the undrained shear strength of the soil is equal to or in excess of 1000 psf, no embankment stability problem exists at this location for the fill heights proposed. It is recommended, however, that the softened veneer of organic materials be removed from the low swamp area of the site before fill is installed. The thickness of this organic soil is in the order of 4 feet.

3) Embankment Settlement - In view of the variations in the strength of the clay at this site and the prominence of silt below a depth of 20 feet, estimates of settlement of this structure must necessarily take an approximate form. A detailed computation of settlements at various locations under the bridge and adjacent embankments may be misleading, since it would imply an accuracy which is not necessarily present. It is proposed, therefore, to examine the settlement problem in a general manner initially in order to determine, in fact, if a problem does exist.

Since the thickness of the clay below footing level is more or less uniform across the bridge site, the amount of settlement to be experienced will be determined by the height of the approach fill only. At the north approach, the fill is in the order of 28 feet high; at the south side the embankment height is about 18 feet. The worst abutment loading condition, as regards settlement, occurs when the fill spills through, in an uninterrupted manner, toward the adjacent pier. Recent computations, made for another project, indicate that the stress on centre line at 20 feet, - the approximate mid depth of the clay stratum, - is about $\frac{6}{7}$ of the full embankment weight. For an embankment 28 feet high, and a fill unit weight of 125 pcf, this average stress is approximately 3000 psf. The stress at a corresponding depth under the centre of a solid abutment, where the fill is entirely retained, is reduced to approximately 50 percent of the full embankment weight, or in this instance to about 1800 psf.

A small increment consolidation test on a sample of weaker clay from a depth of 13 feet in hole 1, indicates a modulus of compressibility, Mv, equal to .0036 sq.ft./kip. The average modulus of the entire clay mass,

* "Settlement Study - Dingman Creek Road Overpass - W.P. 22-59-1, July 20, 1962" W.A. Trow and Associates Ltd.

having regard for the generally stiffer soil condition and the silty nature of the ground at greater depths, is estimated conservatively to be .003 sq.ft./kip. The anticipated consolidation settlement under the centre of the abutment, with fill spilling through it on a 2 to 1 slope is determined from the expression:

$$S_c = Mv \Delta pH$$

$$= .003 \times 3 \times 30 \times 12$$

$$= 3.24 \text{ inches}$$

where: $H = 30$ ft. is the thickness of the compressible clay below abutment footing level

The elastic settlement under this load is computed from the relationship:

$$S_i = 0.5 \Delta p H/E$$

where: E, the modulus of elasticity, is estimated from the stress strain curves of Dwg. 9 to have a value of 250 ksf

$$= 2.1 \text{ inches.}$$

The experience of field installations indicates that actual settlements will be considerably less than this estimate. This being the case, it is believed that the maximum settlement occurring under the weight of the north embankment fill will be within tolerable limits.

Lesser amounts of settlement will occur at the ends of the abutments, theoretically, although, because of the skew nature of the crossing, relatively more movement will be experienced at the obtuse northeast corner than at the acute northwest corner of the fill. Lesser amounts of settlement will take place under the adjacent pier and also under the south embankment fill which, being lower, exerts a much lower surcharge stress.

It also follows that less settlement will occur under a single span closed abutment-type structure, since the stress exerted by the fill under the abutment will have a lower magnitude. Because of its more rigid nature, there will be less tendency for differential settlement with the single span scheme.

In all instances, the deleterious effects of the settlement can be reduced considerably by placing the fill well in advance of bridge construction. Since the soil contains many permeable, horizontal drainage channels, in the form of silt and fine sand seams, consolidation should occur at a relatively rapid rate. No attempt has been made in this report, to estimate the rate of settlement.

We believe that the contents of this report contain sufficient information for the design of this highway crossing. We shall be pleased to discuss any matters that may occur to you after you have had an opportunity to review its contents.

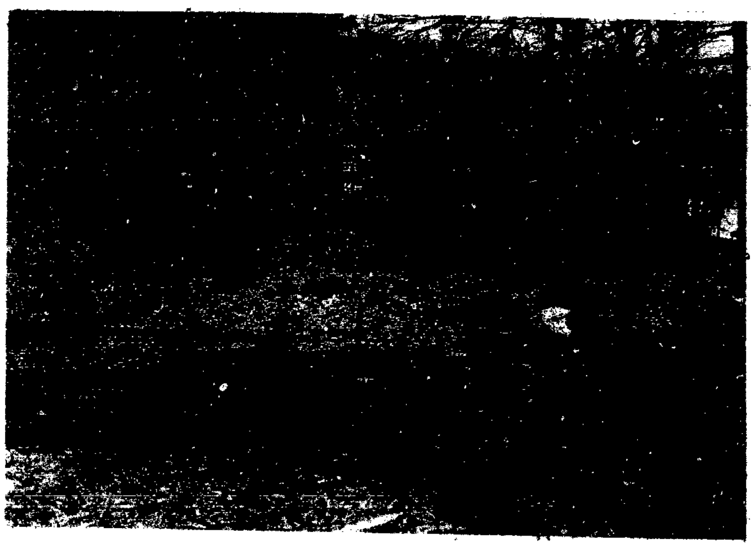
Yours very truly,

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

W. Trow
William A. Trow, P.Eng.

WAT/gc
Encls.

View From the South
Drill on Hole 5



View Looking South
Across Swamp Along CL

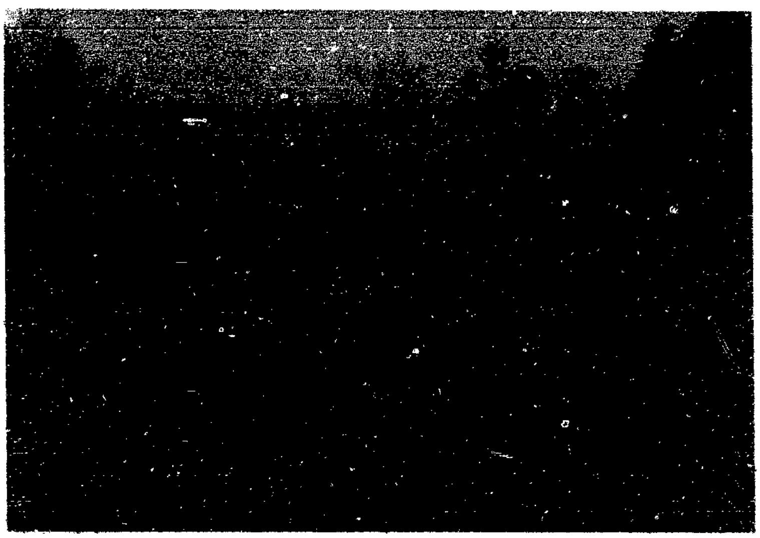
[illegible]

[illegible] Hole 5

[illegible] South

[illegible]

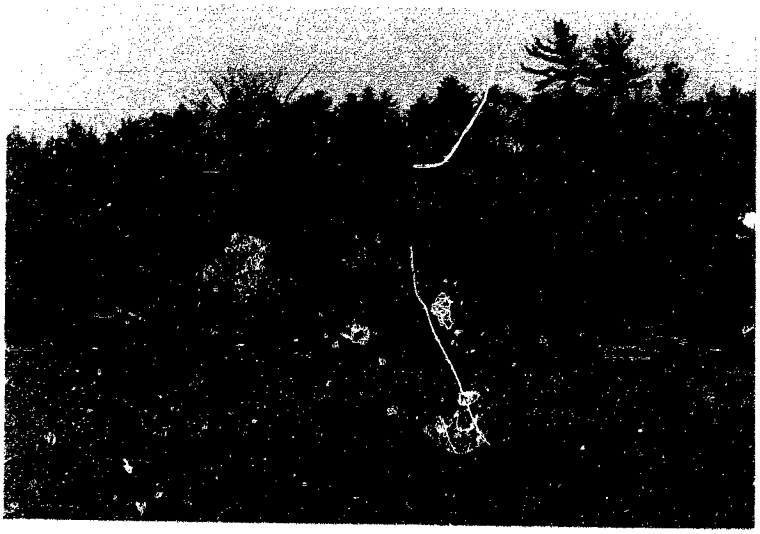
View From the East
Drill on Hole 1



Looking Along 401 CL
From the Southwest

[illegible] East
[illegible] on Hole [illegible]

[illegible]
[illegible]



View From the West
Drill on Hole 5

Looking North From North of Swamp
Looking along Proposed La Rue Mills CL

[illegible] From the [illegible]est

[illegible] Hole 5

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No 2
PROJECT No J968

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
Vane Test > 2100 psf

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

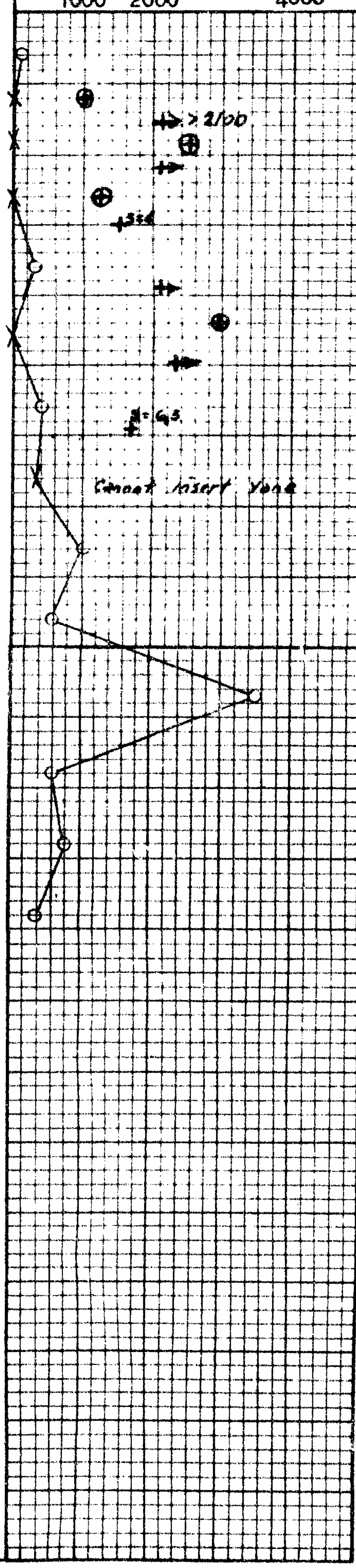
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT

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

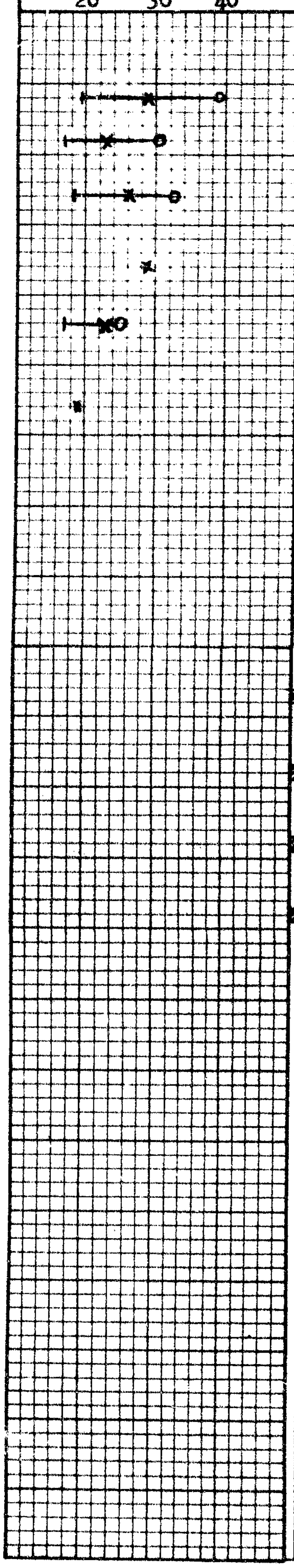
BOREHOLE No 1
PROJECT La Rue Mills Road, Underpass, Hwy. 401, WP 175-61
LOCATION North of Rockport, Ontario
HOLE LOCATION Sta 18+83 - 15 ft. west
HOLE ELEVATION 274.8 ft.
DATUM See Dwg. 1.

SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
6 ins. water	274.8	0		
CLAY-soft, brown, numerous fine sandy silt pockets or layers, organic pockets to 4 ft., very stiff below 4 ft.			SS1	
			TW2	121
		10	TW3	129
			TW4	125
clay faintly varved or layered at 17 ft.; rythmic bands of dark brittle clay, light clay and silt.			SS5	
		20	TW6	131
Quite silty below 22 ft.; brittle chunks of hard dark grey clay appear inclined or contorted; silt predominates.			SS7	
		30	TW8	
	240			
SAND-uniform med., silty with some clay seams.			SS9	
		40	SS10	
	228			
SAND & GRAVEL-very dense, some clay seams.	225	50	SS11	
SAND-med. dense, grey and brown, some gravel sizes, some thin clay seams.			SS12	
		60	SS13	
			SS14	
BEDROCK-hard granite gneiss.	209.8			
Proved bedrock for 13 ins., 13 ins. recovery; diamonds worn off in extremely hard granite.	208.7			
		70		
		80		
		90		
		100		
		110		

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT: 20, 40, 60, 80
SHEAR STRENGTH P.S.F.: 1000, 2000, 4000



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



Notes:
1) Boring cased to full depth, conventional wet sampling methods used.
2) No long term water level measurement possible since hole under water.
3) Jetted to rock at Sta 19+11.5 15 ft. west. Refusal at 70.9 ft. or El 203.9 ft.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

DRAWING No. 3
PROJECT No. J968

BOREHOLE No. 2
PROJECT La Rue Mills Road, Underpass, Hwy. 401, WP 175-61
LOCATION North of Rockport, Ontario
HOLE LOCATION Sta. 19 + 74
HOLE ELEVATION 274.8 ft.
DATUM See Dwg. 1.

LEGEND

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

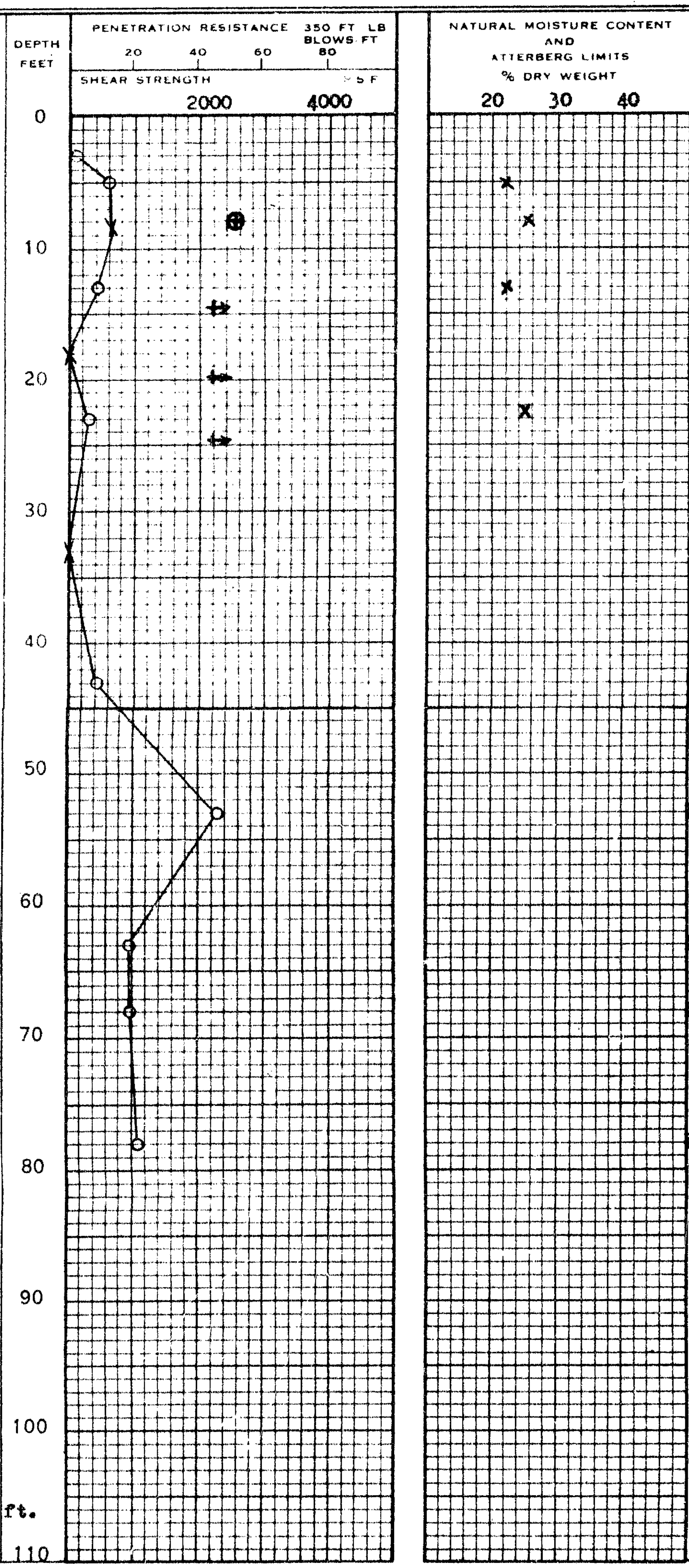
SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
- UNCONFINED COMPRESSION
- VANE TEST AND SENSITIVITY (S)
- Vane test > 2100 psf

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

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

SYMBOL	SOIL DESCRIPTION	ELEV. FEET
	6 "water, rotted vegetation to 1.2 ft.	274.8
	CLAY-soft grey with considerable sandy silt, organic pockets to approx. 4½ ft.; very stiff, silty clay below about 5 ft.; some sand intrusions; predominantly silt below about 20 ft. with some layers of chunky, dark grey clay.	~240
	SAND-silty, fine, grey, with some thin clay seams.	~225
	SAND & GRAVEL-very dense, some clay intrusions.	~213
	SAND-with gravel and some thin clay seams, medium dense.	188.6
	BEDROCK-hard greywacke to 93 ft.; hard granite gneiss below 100% recovery to 88.9 ft. 45% " to 93 ft., remainder ground 100% " to 103.1 ft. (Drill on pressure full water return all times)	
	End of Bore	



SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
SS1	
SS2	
TW3	125
SS4	
TW5	
SS6	
TW7	
SS8	
SS9	
SS10	
SS11	
SS12	

Notes:
1) As in hole 1.
2) Jetted to refusal at 63 ft. or El 212.5 ft. Sta 19+45 - 15 ft. east.
3) Water level at end of bore = 0.4 ft. After 1 day = 2.3 ft., hole open to 23 ft.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO 4

PROJECT NO J968

LEGEND

BOREHOLE NO. 3

PROJECT La Rue Mills Road, Underpass, Hwy. 401, WP 175-61

LOCATION North of Rockport, Ontario

HOLE LOCATION Sta. 20 +45

HOLE ELEVATION 283.0 ft.

DATUM See Dwg. 1.

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

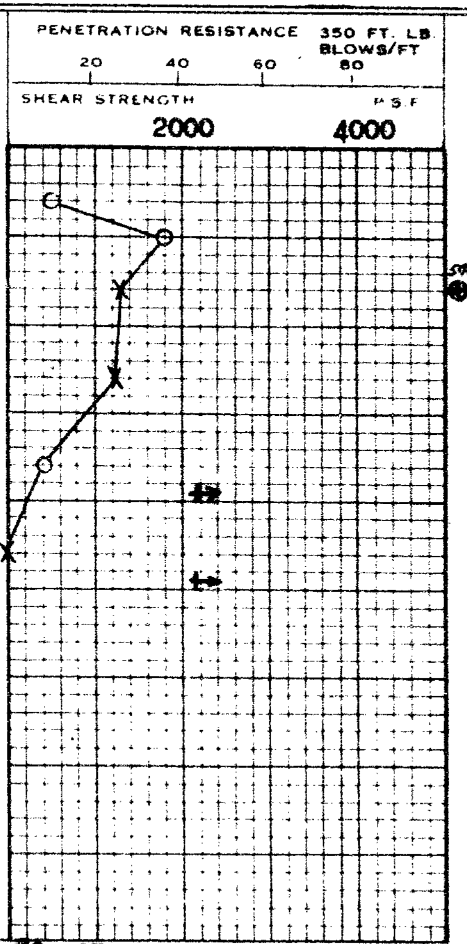
SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
- UNCONFINED COMPRESSION
- VANE TEST AND SENSITIVITY (S)
- Vane Test > 2100 psf

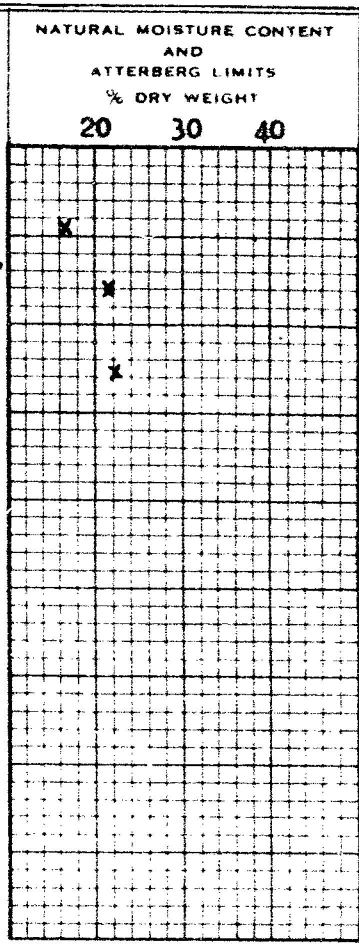
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS
- LIQUID LIMIT
- 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
	5" topsoil	283.0	0		
	SILTY CLAY-firm, brown, some sand partings & silty med. sand seams; becomes hard & dry below about 4 ft., some root hairs at 5 ft.			SS1, SS2	
	CLAY-very stiff, brown, silty; grey below about 17 ft.; quite silty below about 22 ft., with gravel sizes and layers of dark grey chunky clay.	276	10, 20	TW3, TW4, SS5, TW6	129
	No sampling below 23 ft., washed ahead to refusal at 39.3 ft. Very silty and sand wash.		30		
	Refusal to split spoon at 39.3 ft.				
	Refusal and assumed bedrock.	243.7	40		





Notes: 1) Ws in hole 1.
2) W.L. = 0.4 ft. at end of bore.
3) Refusal in hole 3A, Sta 20+74.5, 15 ft. west = 41.7 ft. or El 235.2.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS – SOIL MECHANICS CONSULTATION

DRAWING No. 5
PROJECT No. J968

LEGEND

BOREHOLE No.	4
PROJECT	La Rue Mills Road, Underpass, Hwy. 401, WP 175-61
LOCATION	North of Rockport, Ontario
HOLE LOCATION	Sta. 21 + 60
HOLE ELEVATION	283.0 ft.
DATUM	See Dwg. 1.

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)

Vane test 2100 psf

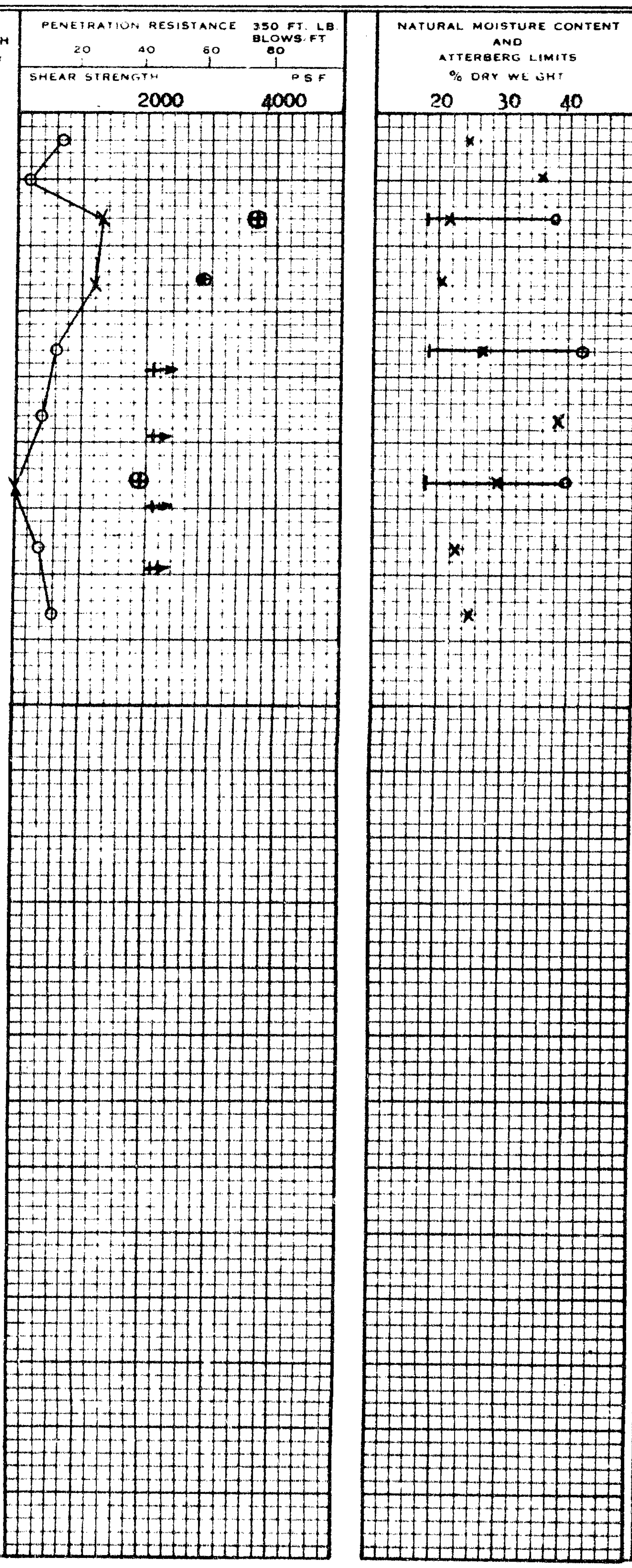
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT

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

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET
	8 ins. topsoil	283.0	0
	CLAY-soft silty some wet fine sand seams, organic pockets to 2½ ft., very stiff, grey below about 6 ft. with dry sand partings and pockets; quite silty below about 25 ft.; some layers of hard dark grey, brittle clay, almost varved; many medium sand partings and seams below about 35 ft.		
	BEDROCK hard granite gneiss 80% recovery to 44.9 ft. 100% " to 48.9 ft. Drill under pressure at all times.	241.0	40
	End of Bore	231.9	

Note: 1) As in hole 1.
2) Water level at end of bore = 8.3 ft.
After 8 hrs. (casing out) = 1.4 ft.
" 12 days " " = 1 ft.
Open to 22 ft.



SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
SS1	
SS2	
TW3	130
TW4	130
SS5	
SS6	
TW7	124
SS8	
SS9	

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

DRAWING No. 6

PROJECT No. J968

BOREHOLE No. 5

PROJECT La Rue Mills Road, Underpass, Hwy.401, WP 175-61

LOCATION North of Rockport, Ontario

HOLE LOCATION Sta 22+20

HOLE ELEVATION 285.1 ft.

DATUM See Dwg. 1.

LEGEND

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

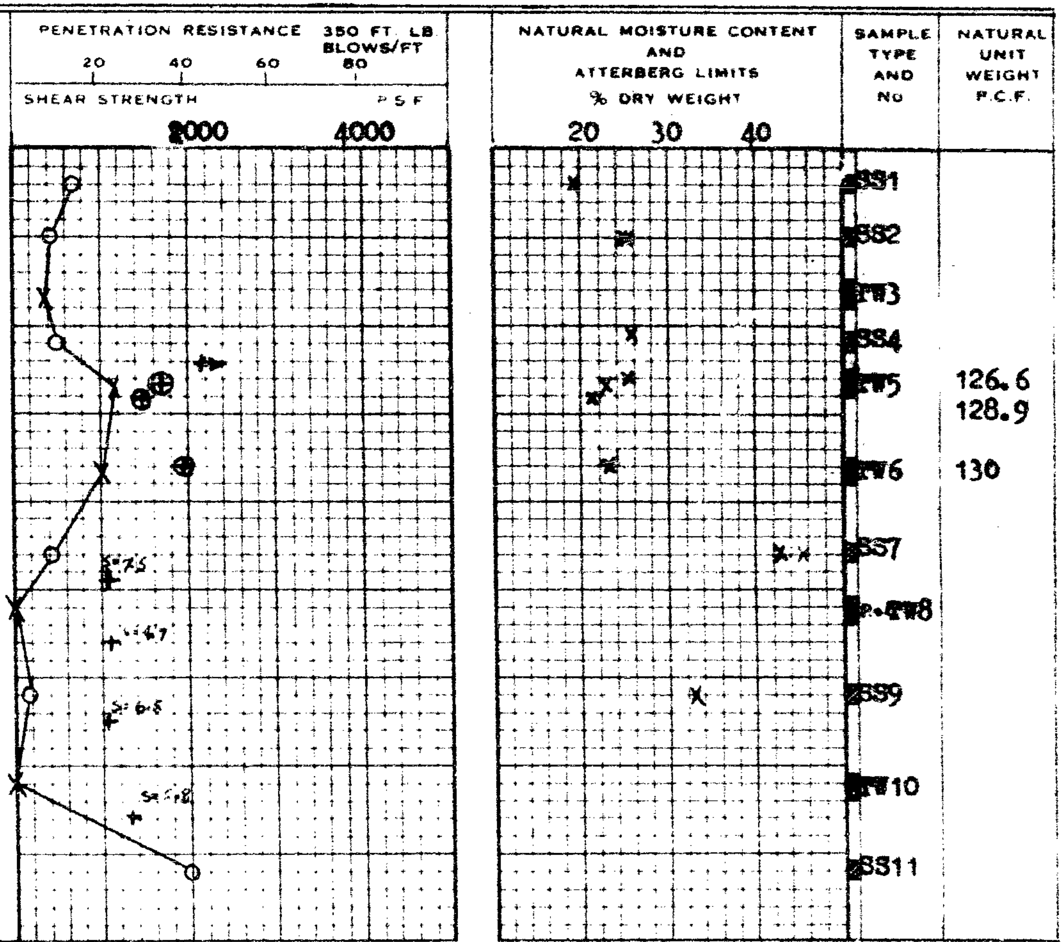
SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) +S

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

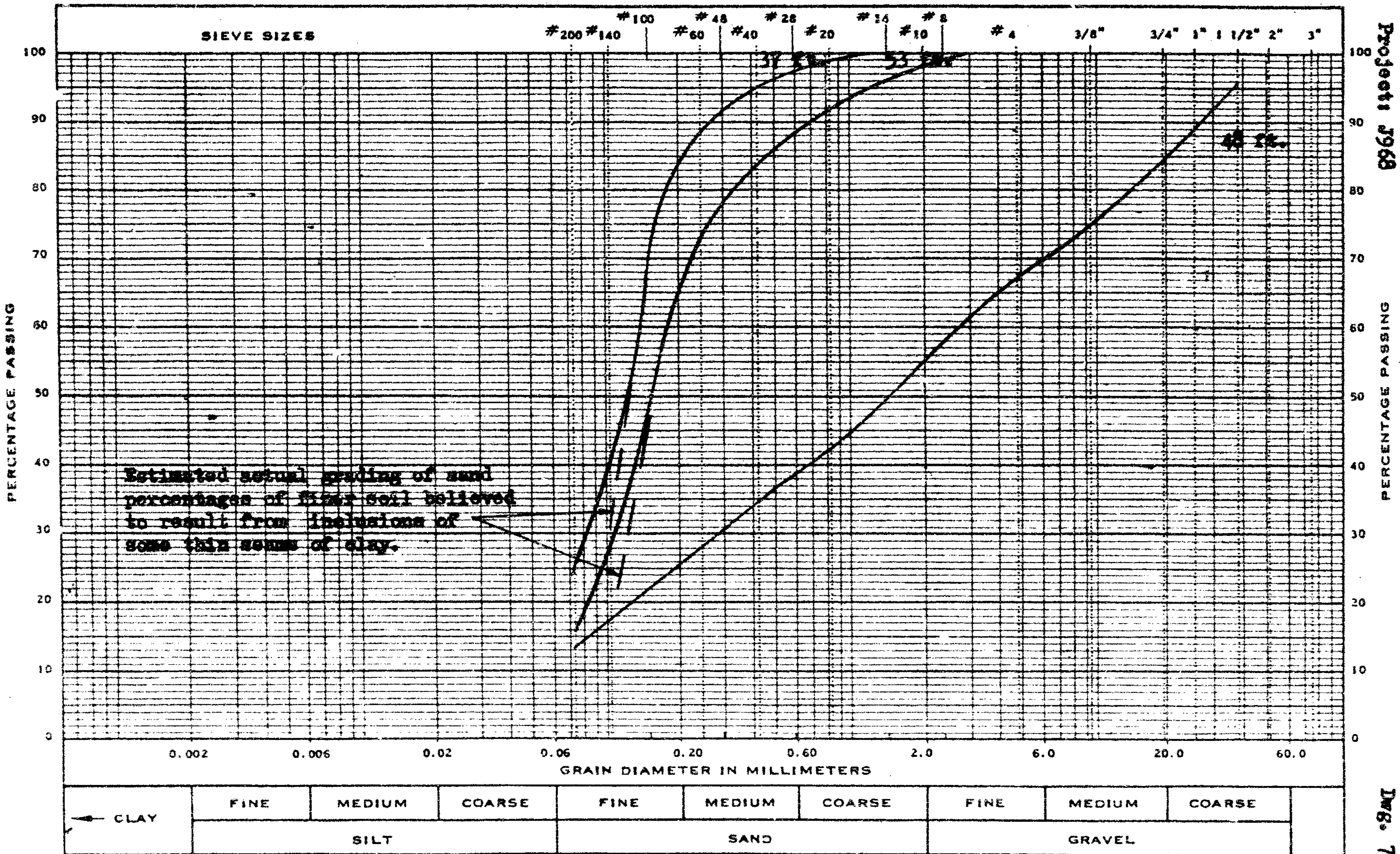
SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET
	8 ins. topsoil	285.1	0
	CLAYEY SILT-stiff, brown, with wet sand layers of varying thickness, some organic pockets above 2 ft. Sand predominates 7 to 10 ft.	~273.6	10
	CLAY-very stiff, grey, silty, unstratified to approx. 20 ft.; then layers of stiff silty clay, silt, hard dark grey chunky clay, and some sand partings. In sample, 25 -27 ft. silty clay occurred in thin layers about 1/8 inch thick, the dark clay was slightly inclined; above 26 ft. the sample consisted of a silty clay till with numerous granite grits and fine gravel.	245.4	20, 30, 40
	GRAVEL-dense with fine to coarse sand & silt.	243.5	
	BEDROCK-hard granite gneiss 100% recovery	241.4	



SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
SS1	
SS2	
TW3	
SS4	
TW5	126.6 128.9
TW6	130
SS7	
TW8	
SS9	
TW10	
SS11	

Notes, 1) As in hole 1. 2) Water level after 54 hrs. = 3.5 ft.

MECHANICAL ANALYSIS



MODIFIED M.I.T. CLASSIFICATION

RESULTS OF GRADING ANALYSES

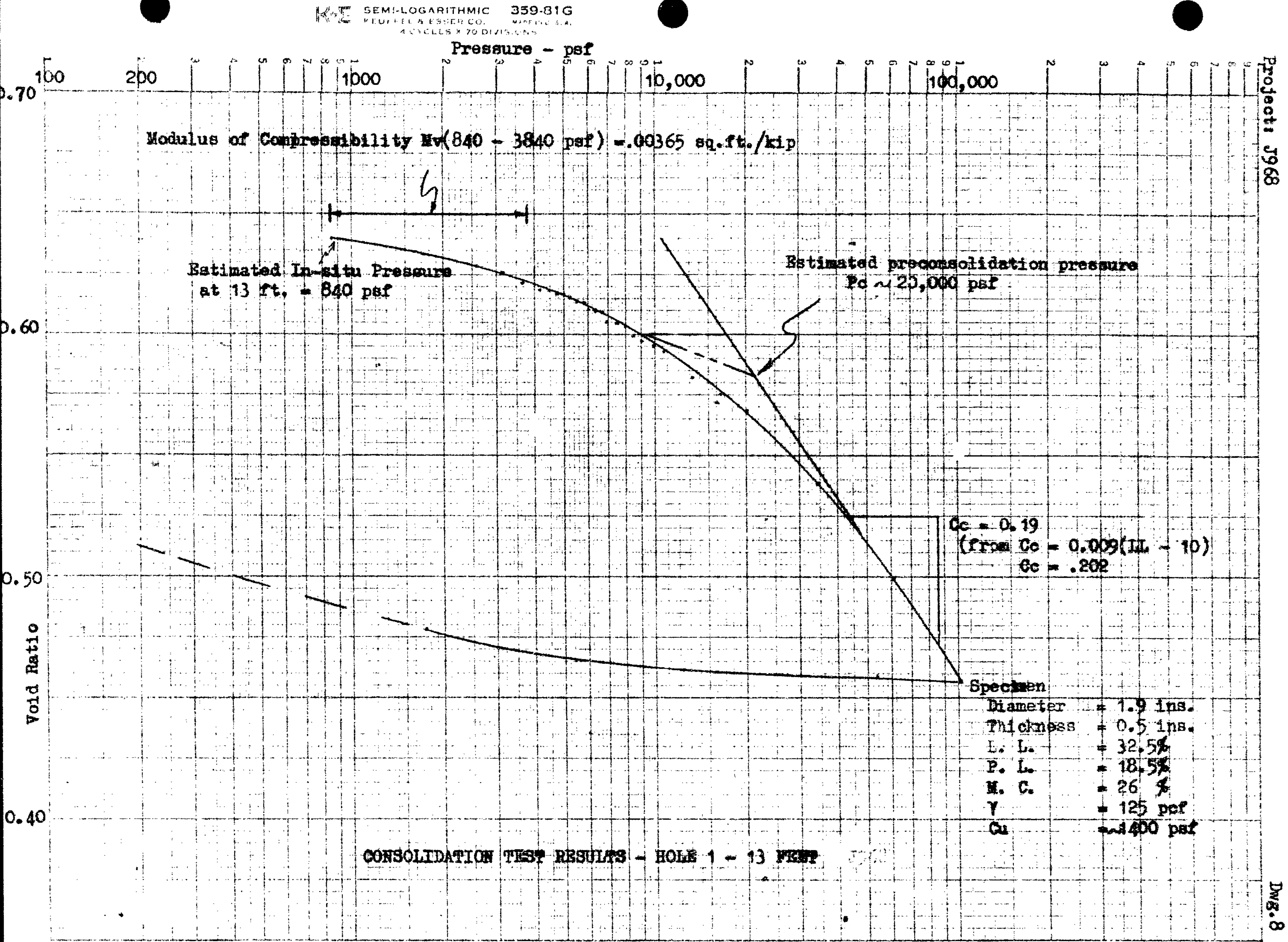
SAMPLES OF SAND FROM HOLE 1

LA RUE ROAD UNDERPASS W.P. 175-61

WILLIAM A. TROW AND ASSOCIATES LTD.

Project: J968

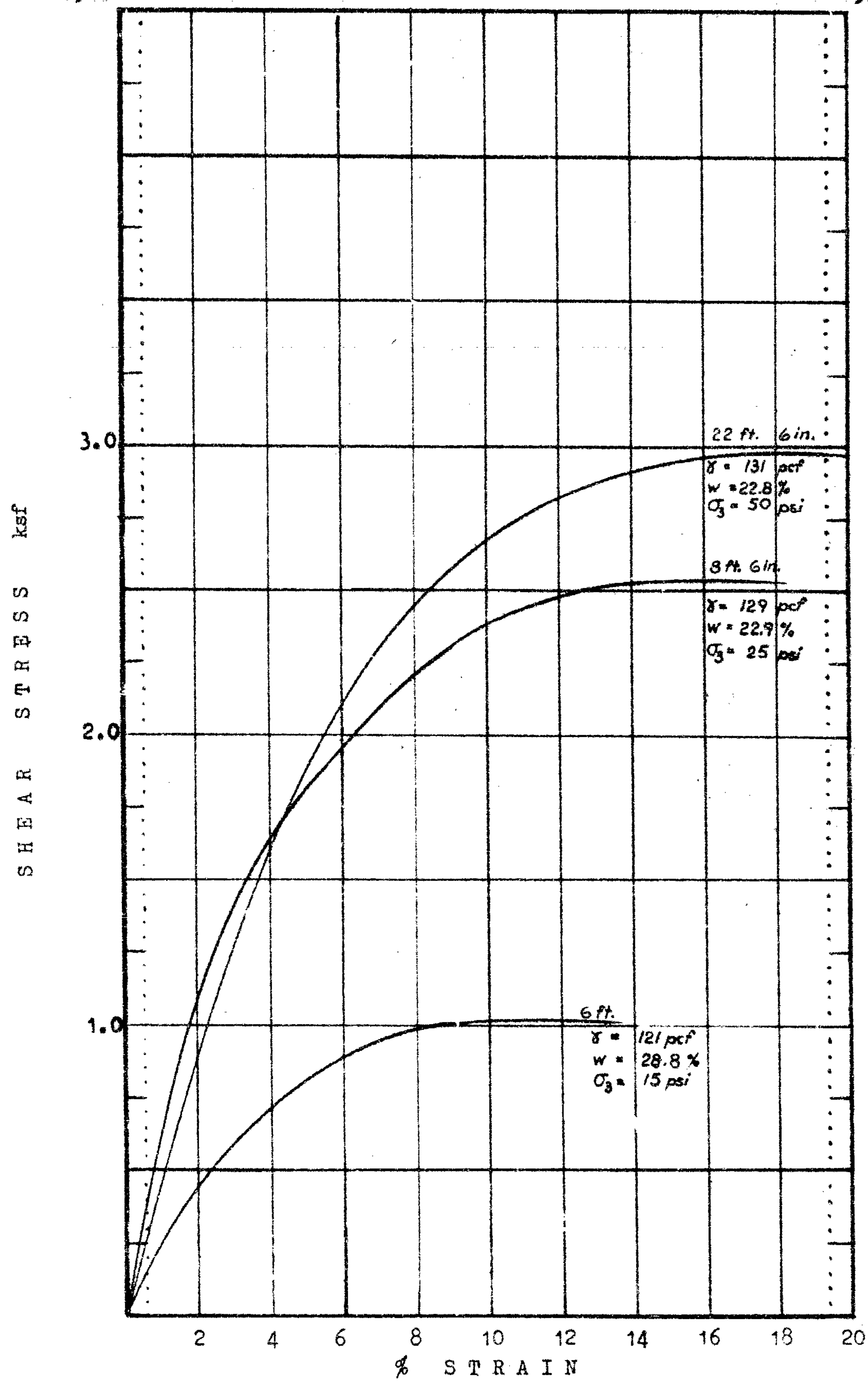
Dwg. 7



K+E SEMI-LOGARITHMIC 359-81G
KEUFFEL & ESSER CO.
4 CYCLES X 70 DIVISIONS
Pressure - psf
100
200
1000
10,000
100,000
0.70
0.60
0.50
0.40
Void Ratio
Project: J968
Modulus of Compressibility Mv(840 - 3840 psf) =.00365 sq.ft./kip
Estimated In-situ Pressure
at 13 ft. = 840 psf
Estimated preconsolidation pressure
Pc ~ 20,000 psf
Cc = 0.19
(from Cc = 0.009(LL - 10)
Cc = .202
Specimen
Diameter = 1.9 ins.
Thickness = 0.5 ins.
L. L. = 32.5%
P. L. = 18.5%
M. C. = 26 %
γ = 125 pcf
Cu =~1400 psf
CONSOLIDATION TEST RESULTS - HOLE 1 - 13 FEET
Dwg.8

PROJECT NO. J968

DRAWING NO. 9a

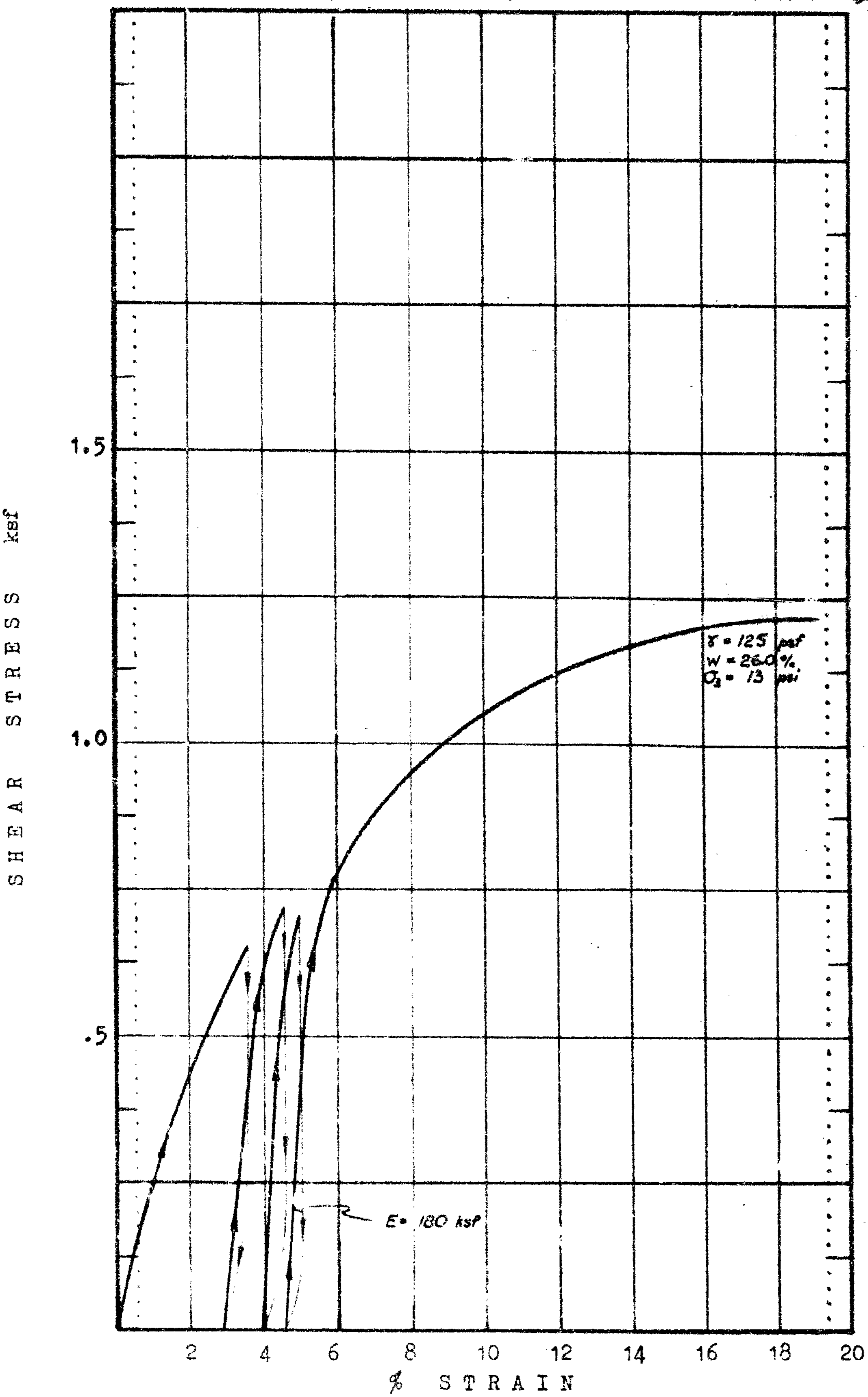


UNDRAINED TRIAXIAL TEST RESULTS

HOLE 1

WILLIAM A. TROW AND ASSOCIATES

PROJECT NO. J968 DRAWING NO. 9b

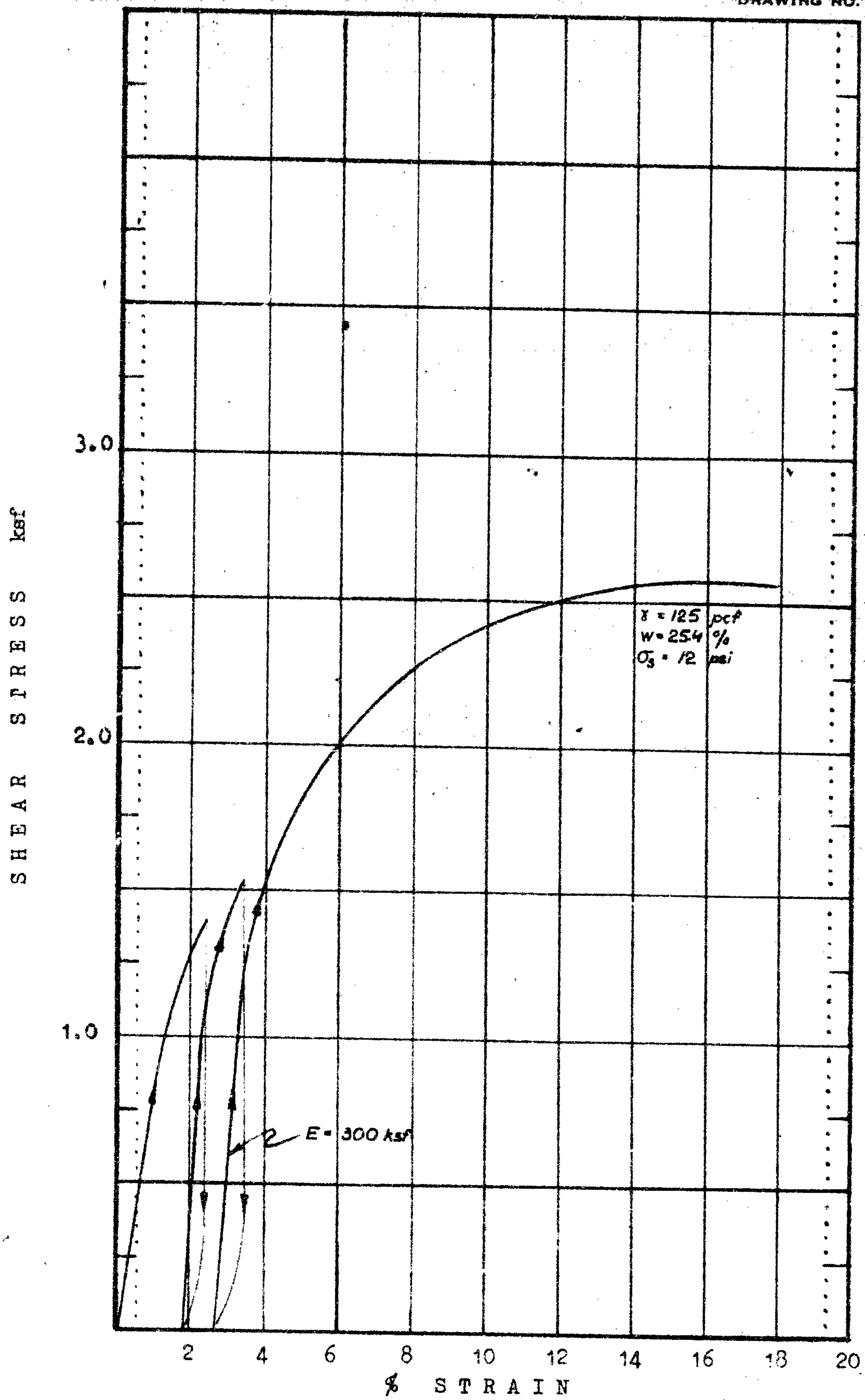


UNDRAINED TRIAXIAL TEST RESULT
WITH ESTIMATES OF ELASTIC MODULUS E
HOLE 1 - 13 FT.

WILLIAM A. TROW AND ASSOCIATES

PROJECT NO. J968

DRAWING NO. 9c

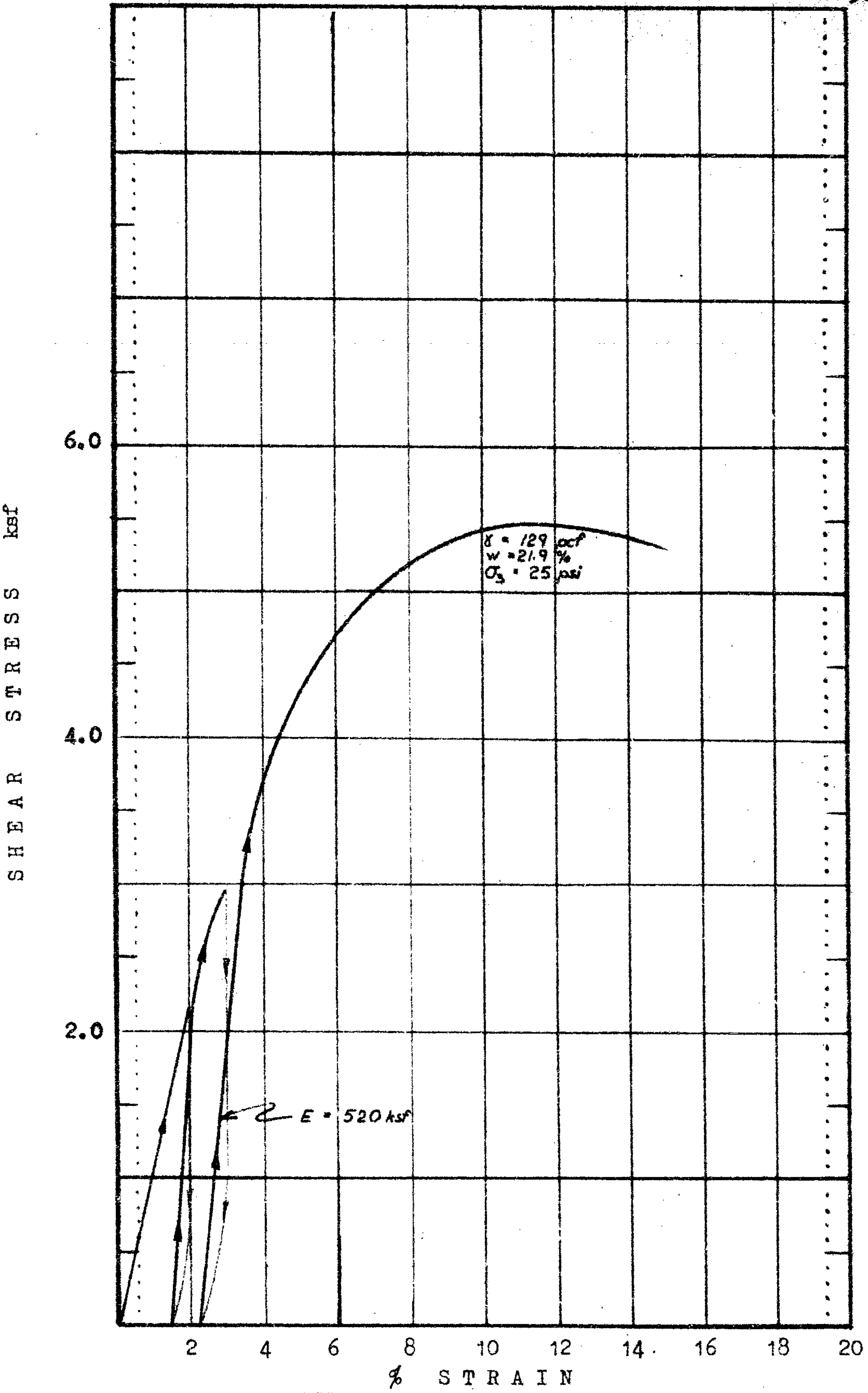


UNDRAINED TRIAXIAL TEST RESULT
WITH ESTIMATES OF ELASTIC MODULUS E
HOLE 2 – 8 FT.

WILLIAM A. TROW AND ASSOCIATES

PROJECT NO. J968

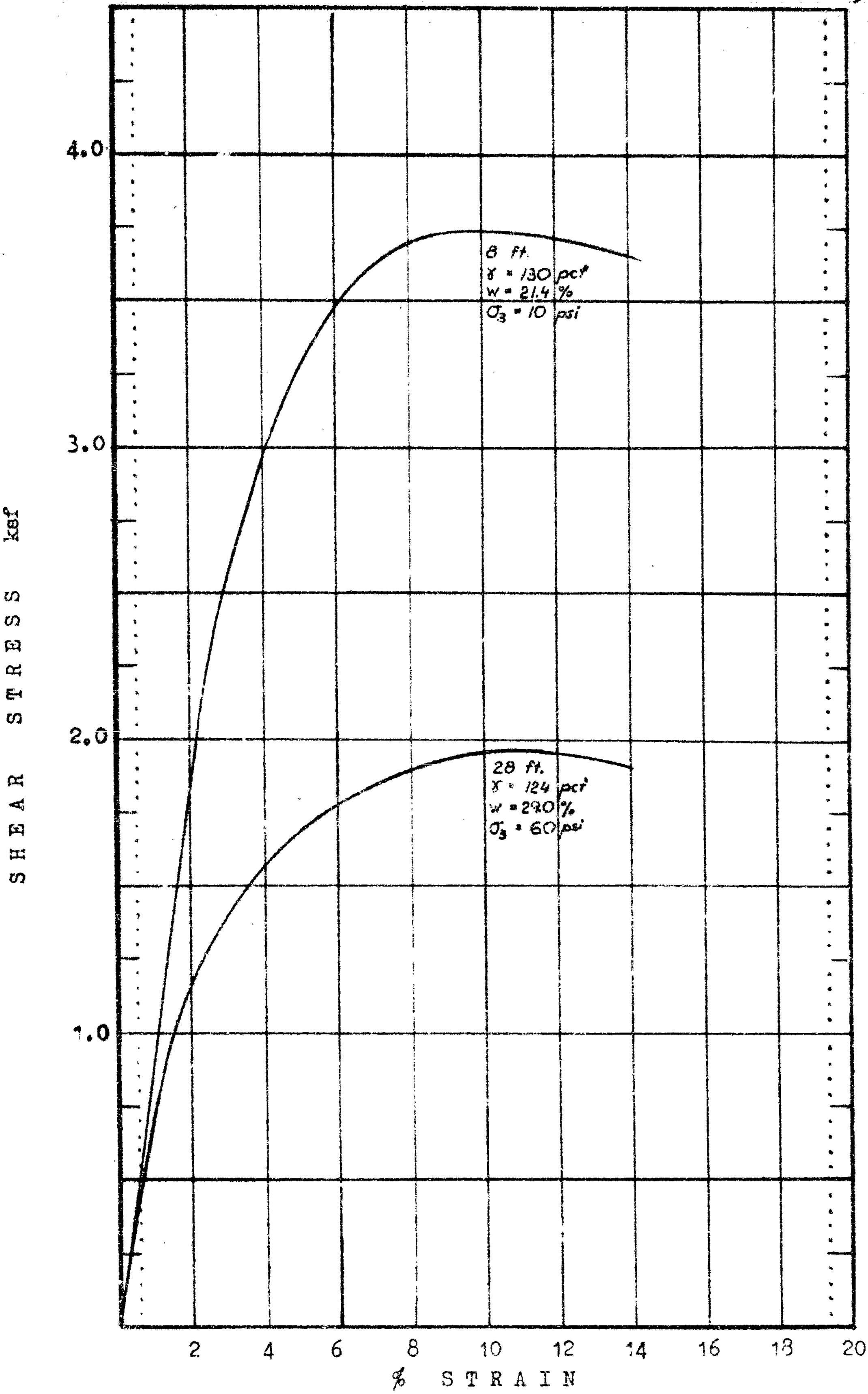
DRAWING NO. 9d



UNDRAINED TRIAXIAL TEST RESULT
WITH ESTIMATES OF ELASTIC MODULUS E
HOLE 3, - 8 FT.

WILLIAM A. TROW AND ASSOCIATES

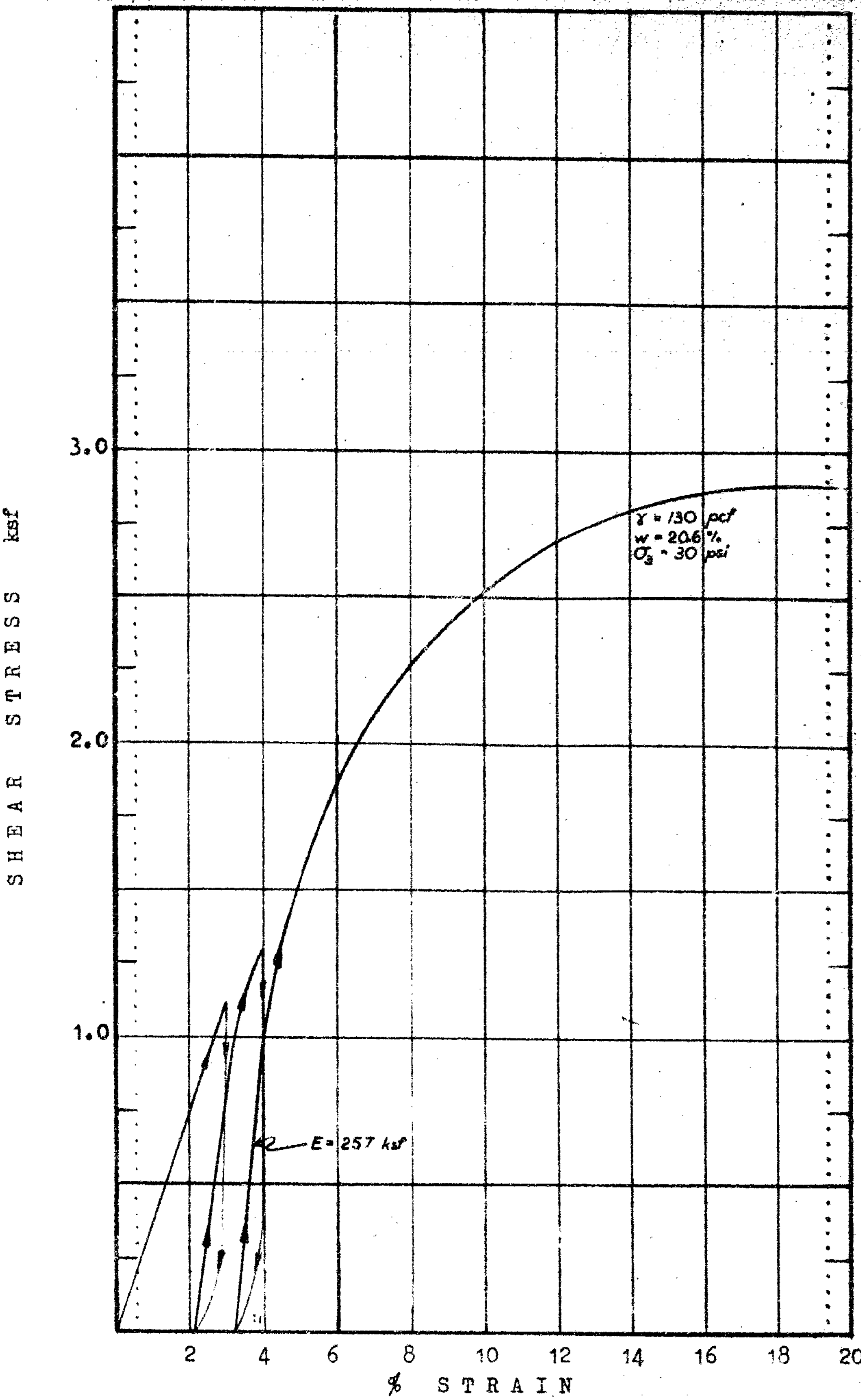
PROJECT NO. J968 DRAWING NO. 9e



UNDRAINED TRIAXIAL TEST RESULTS
HOLE 4

WILLIAM A. TROW AND ASSOCIATES

PROJECT NO. J968 DRAWING NO. 9f



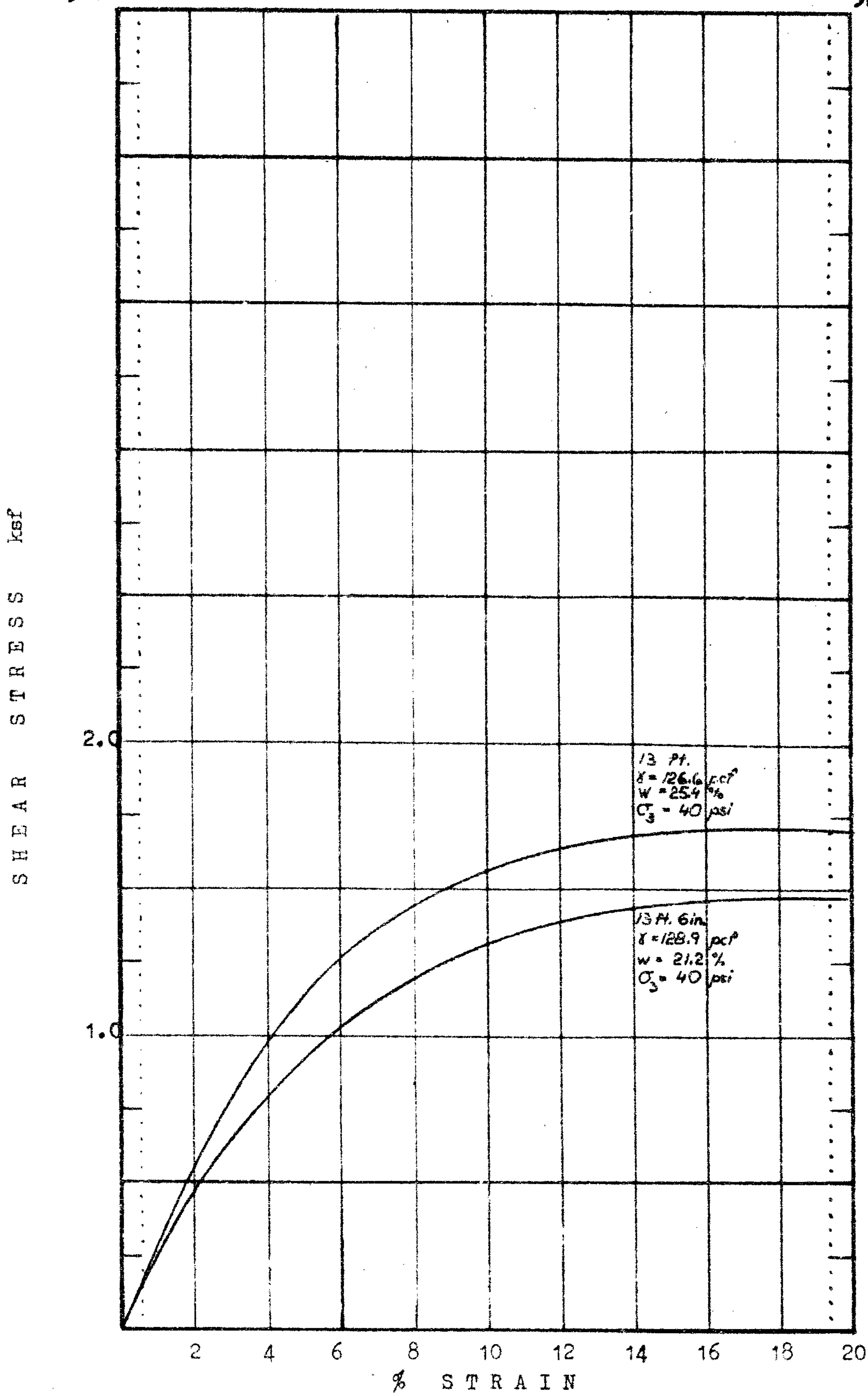
UNDRAINED TRIAXIAL TEST RESULTS

WITH ESTIMATES OF ELASTIC MODULUS E

WILLIAM A. TROW AND ASSOCIATES HOLE 4 - 13 FT.

PROJECT NO. J968

DRAWING NO. 9g

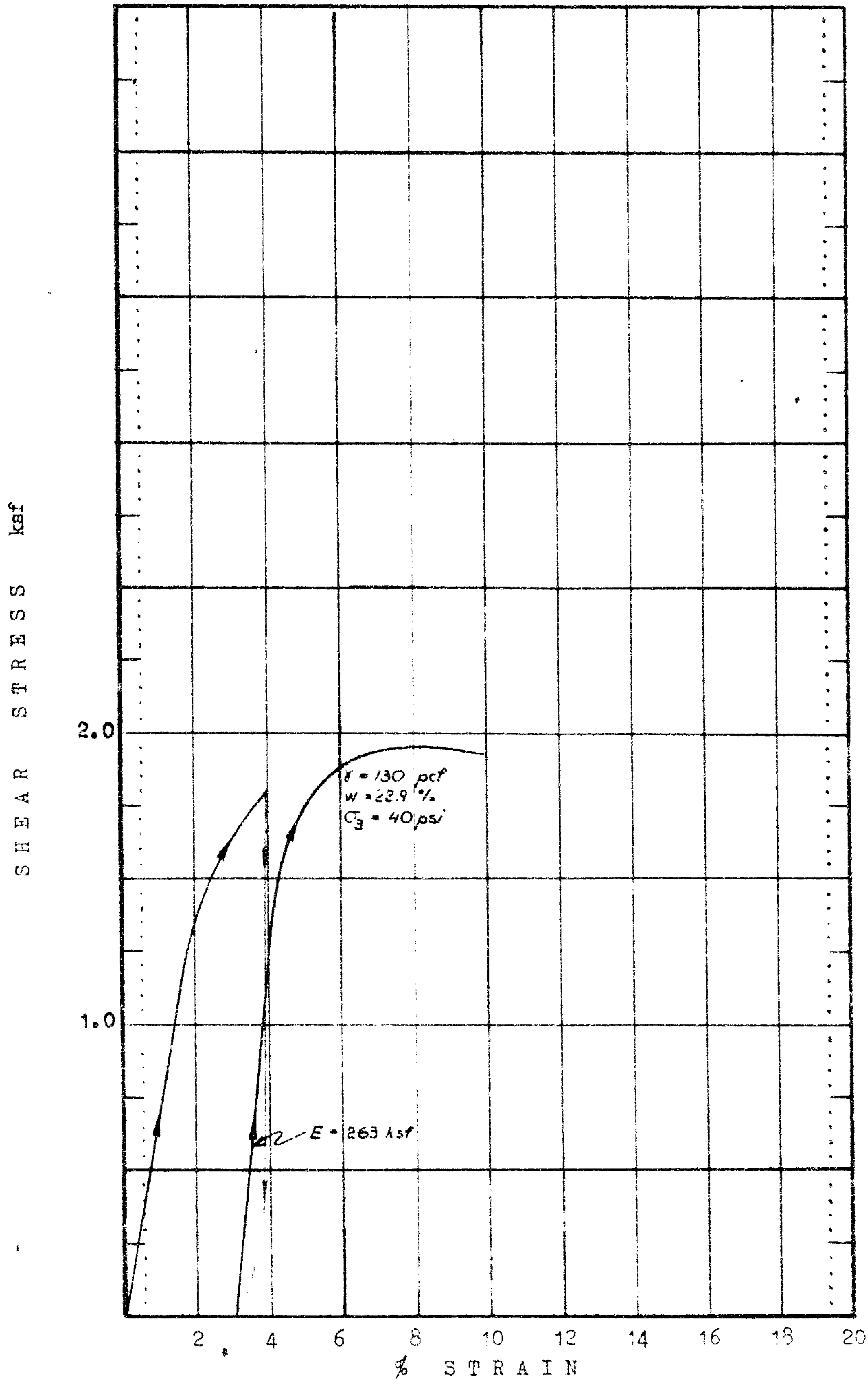


UNDRAINED TRIAXIAL TEST RESULTS

HOLE 5

WILLIAM A. TROW AND ASSOCIATES

PROJECT NO. J968 DRAWING NO. 9h



UNDRAINED TRIAXIAL TEST RESULT
WITH ESTIMATES OF ELASTIC MODULUS E
HOLE 5 - 7 FT.

WILLIAM A. TROW AND ASSOCIATES

W.P. 175-61

Note on telephone conversation with G. Scott on August 3. 1966.

It appears that some difficulties could be encountered with the granular material specified for the foundations of the abutment footings. This is the information comming from Road design. Therefore piles may have to be considered as an alternative. However, up till now nobody has yet officially requested such a change. Gavin Scott is still awaiting the Road Design or the District to raise the question again. As soon as anything transpires that will concern the Foundation Section, we will be informed.

Aug. 3. 1966. A.G. Stermac,

cc: Foundations Office (Rm. 110)

Mr. S. McCombie,
Bridge Planning Engr.,
Bridge Division.

Attention: Mr. A.P. Watt.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.
March 25, 1963.

W.P. 175-61, Bridge Site No. 17-138,
LaRue Mills Road Underpass at Twp. Rd.
to LaRue Mills, Hwy. 401, District 8.

We are in receipt of Drawing No. 5210-P2, Preliminary General Arrangement, for the above-mentioned structure and, herewith, submit our comments for your consideration:

Excavation and removal of the topsoil and soft organic silty clay under the north approach embankment, as shown on the mentioned drawing, is only limited to an area bounded by the intersection of lines drawn at a 45° angle from the edges of the abutment footing. Although this would probably satisfy the basic needs as far as the abutment footing is concerned, we are of the opinion that the presence of soft material under the toe of the embankment - i.e., between the abutment and the first pier, and also at a certain distance behind the abutment, could cause some undesirable consequences. It is therefore recommended that the removal of the soft upper material be carried out to beyond the first pier location, and also to a minimum of 60 ft. back from the centre of the abutment footing.

Although not indicated in the foundation report, some soft topsoil may be present also at the south abutment location. The same procedure as recommended for the north side, should apply here as well.

As is evident from the drawing, the south abutment footing will be placed on some 5 ft., and the north abutment, on approx. 18 ft. of compacted fill. We would recommend that a special note be put on the drawing, or in the Contract, providing for special control and verification of adequate compaction.

We would appreciate being advised of the incorporation of recommended changes.

AGStermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF

cc: Foundations Office
Gen. Files.

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. G. Stermac,
Prin. Foundation Eng.,
Room 107, Lab. Bldg.

From: A.P. Watt
Br. Location Eng.

Date: March 19, 1963

Our File Ref.

In reply to

Subject: W.P. 175-61 Bridge Site #17-138
LaRue Mills Road Underpass
at Twp. Rd. to La Rue Mills,
Hwy. #401, District #8

Enclosed please find ~~one~~ Two copys of the preliminary plan D 5210-P2 for the above noted structure.

The designer appears to have complied with the requirements of the foundation report but we would appreciate any comments you wish to make.

The footing pressures the designer is using are noted on the preliminary plan.

APWatt

APW/m
c.c. S.McCombie

A. P. Watt,
Bridge Location Engineer

Discussion with George Tilly from Duncan Smith & Assoc.

It is believed that the reported settlements (3.24") are on the high side. This is based on available information on settlements of overconsolidated clays (Homer, Welland.)

If maximum settlements are in the order of 2 to 2½ inches, differential settlements will definitely be not more than 1 to 1½ inches. Design will account for settlements of 1½" (George Tilly)

Stage construction would be desirable but is not absolutely necessary.

If the ends of the approach fills are well compacted and granular material used footings of abutments can be placed on the fill. Final decision will depend on economic factors. If piles are used they should be friction piles ending in the clay.

January 24. 1963 K.Y. Lo Ag Stermac

Ted Henson called worrying about settlement of fill during winter placing and thought of timber piles. However, timber piles (friction piles) will not help the settlement problem. Steel H-pile suggested if control of fill cannot be insured.

KYLo 19/2/63

~~cc: Gen. Files~~ Foundation Files

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

Attention: Mr. A. McCooble.

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

November 27, 1962.

FOUNDATION INVESTIGATION REPORT BY -
Wm. A. Trow & Associates, Limited,
La Rue Mills Road Underpass, Hwy. #401,
District #8 — W.P. 175-61.

The report for the foundation investigation at the above site, by the Consultant, has been reviewed and we submit the following comments:-

Due to the extreme irregularity of the depth of bedrock, and the presence of a very dense sand and gravel stratum of variable thickness, it is difficult to predetermine the lengths of piles required. We therefore, recommend the use of spread footings for foundations of all piers and abutments in order to avoid construction problems. In any case, if foundations employing piles driven to bedrock are contemplated, another investigation has to be carried out in order to determine the surface bedrock in greater detail.

KYLo

K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

KYL/McoF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
J. Ford
W. A. Cash
J. E. Gruspier
T. J. Kovich
J. Fey
R. T. Saint
F. Dureman
A. Watt
Foundations Office
Gen. Files.

Materials and Research Division

October 13, 1962.

William A. Trow & Associates, Ltd.,
1850 Jane Street,
Weston, Ontario.

Attention: Mr. Wm. A. Trow.

Re: W.P. 175-61, Hwy. #401,
La Rue Mills Rd. Underpass,
District #9, Kingston.

Dear Sirs:-

Please consider this your authority to carry out a foundation investigation at the above site. Plans and profiles were provided to your representative on October 12, 1962.

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

Fourteen copies of the completed foundation report, plus an additional copy of the subsoil profile, should be submitted to the Foundation Section as soon as possible. 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 May 24, 1957, and invoice 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.

Yours very truly,

A. Rutka,
MATERIALS & RESEARCH ENGINEER.

AR/MMcF

cc: Messrs. J. McCombie
J. Ford
W. A. Cash
J. G. Gruspier
N. D. Smith (2)
Mrs. T. Tate
Foundations Office
Gen. Files.

Nick DAMAS   W.P 175-61

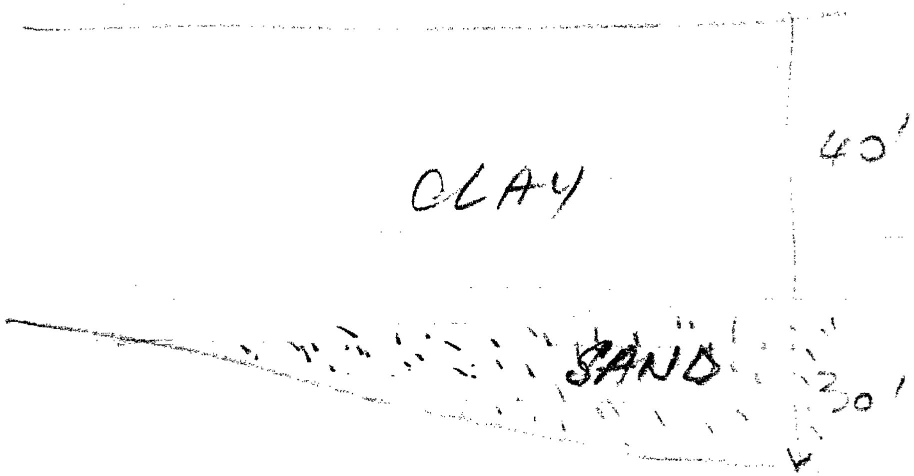
401 District 8 Gananoque - Brockville

La Rue Mills Rd Underpass

W.A. Trow

50' - 100' below grade (bedrock)

Soft clay 40'



Spread footings - q_a = 3000 lbs/sq ft

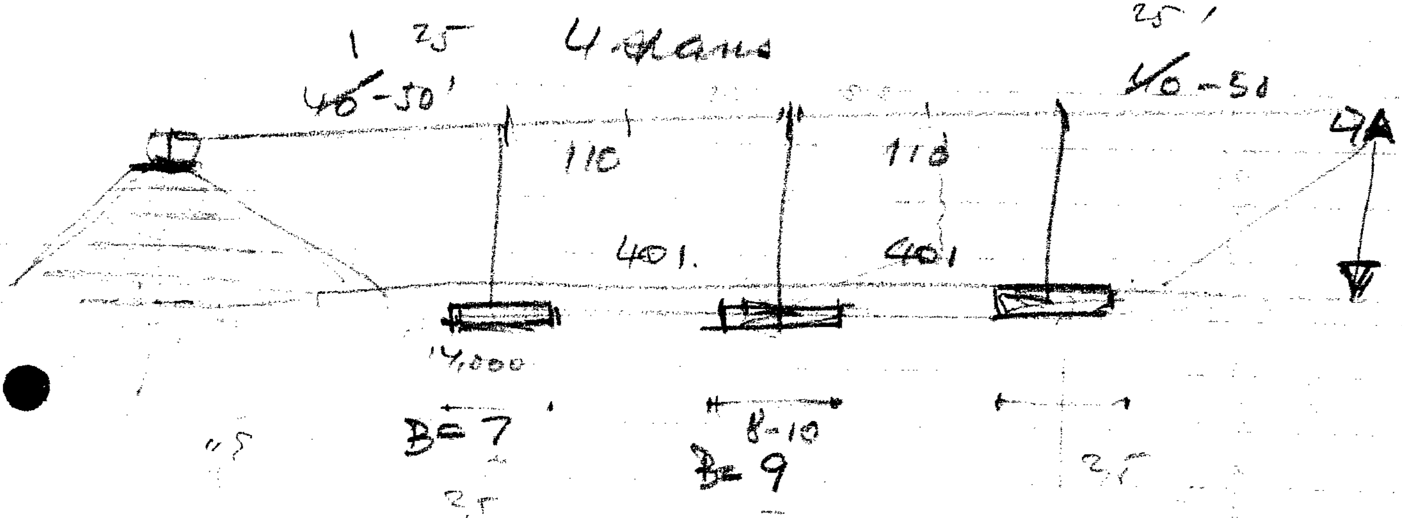
Piles to bedrock

Span - differential settlements ?

End spans open abutments

OVER

Abutments on columns
or spread footings
on compacted fill



34'

305
275
30

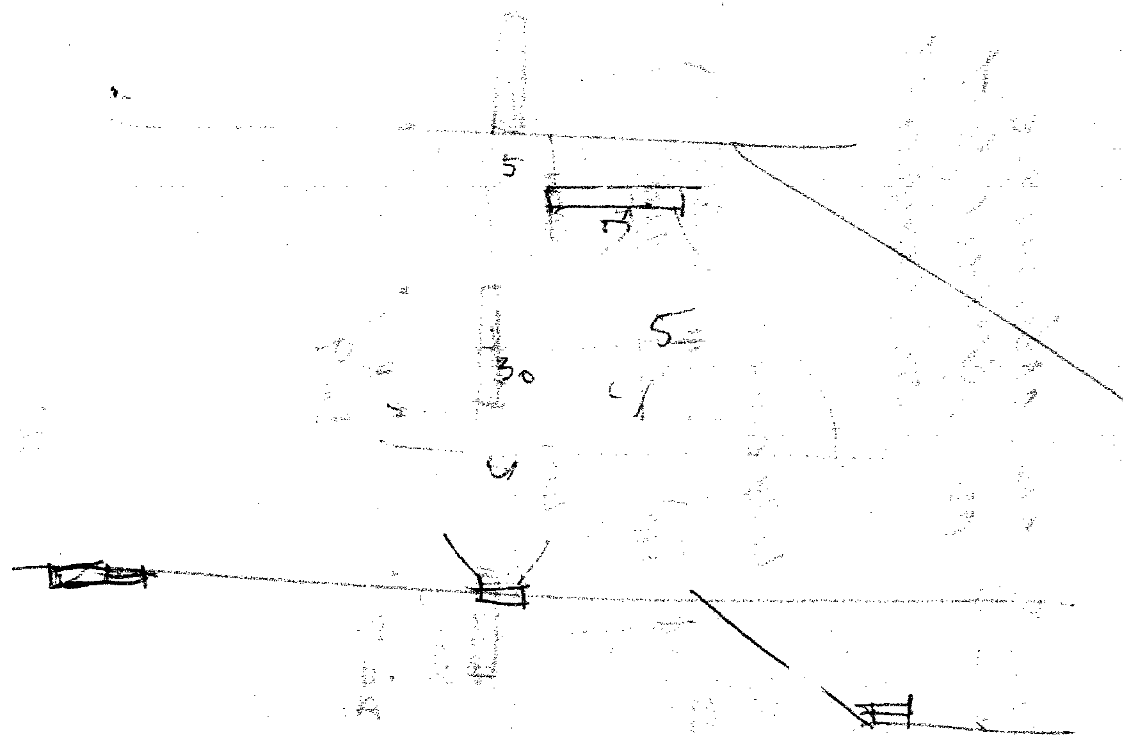
24' + 24 = 48'

WIDTH OF ROAD (EMBANKMENT) = 48'
HEIGHT OF EMBANKMENT = 30'

OVER

lb/cu ft

$\frac{130 \times 30\ ft}{3900\ lb/sq\ ft} \sim \underline{4000} \sim \underline{2.0\ T/sq\ ft}$?



Department of Highways Ontario

Copy for the information of
Mr. A. Stermac, Principal Foundation Engineer,
Room 107, Lab. Building

Mr. E.A. Cash,
District Engineer,
Kingston District

Bridge Division,
Downsview, Ontario

Attn: Mr. K. Westerby,
Construction Engineer

September 30, 1966

La Rue Mills Rd. Underpass
W.P. 175-61
Hwy. 401, District No. 8

This will confirm our telephone discussion in which we explained that Mr. A. Stermac, Foundation Engineer, has requested that a 10 foot high surcharge be placed over the earth fill bench at each abutment location. This fill should be 10 feet wide and the width of the roadway. This extra fill will be removed when the structure is built. Presently construction is scheduled for early next year.

The bridge plans call for granular fill under the abutments, which we understand is most difficult to obtain in this area. The earth fill presently being placed may be adequate after it has been surcharged. This will eliminate the necessity of granular material.

The Foundation Section wish to be notified before this surcharge is placed so they can instrument the fill and record settlements.

If you require further information regarding this surcharge kindly contact Mr. Stermac.

C.S. Grebski,
Bridge Design Engineer

CSG:rd

c.c. A. Stermac
A.E. McKim
W. Wigle

cc: Foundations Office (Rm. 110)

Mr. E. A. Cash,
District Engineer,
Kingston (Dist. #8).

Attn: Mr. K. Westerby,
Construction Engr.

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

October 7, 1966

La Rue Mills Road Underpass
Hwy. 401 - District 8 (Kingston)
-- W.P. 175-61 --

66-F-100

Further to our telephone discussion pertaining to the 10-ft. surcharge to be placed over earth fill at each abutment location, we have prepared a detailed drawing which will enable you to proceed with the construction work. The Regional Materials and Testing Section will carry out the installation of settlement plates at the abutment locations prior to the construction of the 10-ft. high surcharge. Our drawing shows the limits of surcharge, including the sequence of construction of the approach fills for the above mentioned project.

This Section wishes to be notified at the completion of the surcharge, so that the settlement observations can be carried out. If you require further information pertaining to this project, please feel free to contact our Office.

AGStermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

MD/MdeF
Attach.

cc: Messrs. J. E. Gruspier
C. S. Grebski

Foundations Office
Gen. Files

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. G. Stermac
Principal Foundations Engineer
Downsview.

From: M. & T. Division
Kingston

Date: October 11, 1966

Our File Ref.

In reply to:

Subject:

Re: Hwy. 401, Cont. 65-217
La Rue Mills Rd. Underpass

W.P. 175-61
66-F-100

This memo is to report on the placement of settlement plates at the approach embankments for this proposed structure on October 6th, 1966.

The plates were placed on the north approach at the following locations with these elevations recorded:

Plate No. 1	-	19+01 (8.5' Lt.)	-	293.05
Plate No. 2	-	19+11 (1.5' Rt.)	-	292.81
Plate No. 3	-	19+20 (11.0' Rt.)	-	292.55

The plate for the south embankment could not be placed due to a pile of loose material which had been placed in the way. The District personnel will place this plate at Sta. 22+07 (1.5' Lt.) when this loose material is spread and compacted.

H. A. Meyer
H. A. Meyer

for: J. E. Gruspier
Regional Materials Engineer

HAM:mgm

cc: Foundations Files (Rm. 110)

Mr. E. A. Cash,
District Engineer,
Kingston (District #8),

Attention: Mr. K. Westerby,
Construction Engr.

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

May 29, 1967

La Rue Mills Rd. Underpass --
Hwy. #401, District #8 (Kingston),
W.P. 175-61 -- Contract 65-217.

Further to your request, the Foundation Section installed a total of three (3) settlement plates to observe the settlements of the approach fills of the above mentioned structure. The details were discussed in our memo to you dated October 7, 1966.

We have reviewed the settlement readings observed from November 7, 1966 to May 24, 1967. It appears that no further settlements have taken place since February 1967 and, for practical purposes, we can assume that the settlements have been completed due to the imposed 10-ft. surcharge at the approach fill locations. Therefore, we recommend that the 10-ft. surcharge be removed, and the construction of abutments on spread footing type of foundations, be carried out.

If you have any further queries, regarding this project, please contact our Office.

MD/MdeF

M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

cc: Messrs. C. S. Grebski
J. E. Gruspier

Foundations Files
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