#59-F-240-C
HWY. #401
OVERPASS C.P.R.
NEAR PRESCOTT

BA 880

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

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

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Project: J 344

March 27, 1959.

Mr. A. M. Toye,
Bridge Engineer,
Dept. of Highways of Ontario,
280 Davenport Rd.,
Toronto, Ont.

Attention: Mr. J. McAllister

Foundation Investigation
C.P.R. Overpass, Hwy.401,
near Prescott, Ont.

Dear Sirs:

The report contained herein describes the soil conditions existing at the railway overpass site noted above.

Although the entire area is flat and poorly drained, relatively satisfactory foundation conditions prevail at this crossing. A stiff to very stiff stratum of marine clay will be the supporting medium for the bridge and associated embankments. A permissible bearing pressure of 4500 p.s.f. has been recommended for this material. The long term settlement resulting from the weight of the overpass structure, including earth fill, has been estimated to be 4½ inches. This movement will take place at a very slow rate.

Limestone bedrock lies about 55 feet below present ground level. The support of the structure on piles bearing on this rock is not recommended.

If additional queries come to mind after your review of the contents of this report, we shall be pleased to hear from you on the matter.

It has been our pleasure, again, to serve you.

Yours very truly,

W Trow

William A. Trow (P. Eng.)

WAT/lt
Encl.

DEPARTMENT OF HIGHWAYS OF ONTARIO
280 DAVENPORT ROAD, TORONTO.

FOUNDATION INVESTIGATION
C.P.R. OVERPASS, HWY.401,
NEAR PRESCOTT, ONT.

Project: J 344 March 27, 1959.

William A. Trow & Associates

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FOUNDATION INVESTIGATION
C.P.R. OVERPASS, HWY. 401,
NEAR PRESCOTT, ONTARIO.

The site of this overpass, near Prescott, Ont., has been investigated by six borings made at representative locations in the proposed crossing area. A description of the types and physical properties of the underlying soil has been given. A discussion has been made of the probable behaviour of these natural deposits when subjected to the overpass loads, and recommendations concerning permissible bearing pressures for abutments have been given.

Site Description

At the time of the investigation about 3 to 4 feet of snow covered the ground and, as a consequence, many surface features were obscured. The site lies in flat, somewhat swampy, bush country, about two miles north-east of Prescott. The east lane of Hwy. 401 has been completed for about 1½ miles each side of the railway, which it intersects at an angle of approximately 60 degrees. A drainage ditch passes under the railway just north of the bridge site and also under the highway immediately to the west.

Field Sampling Methods

The field work of this investigation was performed by conventional methods, using a diamond drill adapted for soil sampling purposes. However, because water was in somewhat short supply, the first three borings were advanced by driving 2-7/8 inch I.D. BX casing and then cleaning out the soil between sampling intervals with AX pipe. No wash water was used in these holes.

Samples in hole No. 1 were taken at 5 foot intervals of depth down to bedrock level. Many of these samples were taken for purposes of identification and for moisture content measurement and therefore were recovered in a partially disturbed state using a 2-inch O.D. thick-walled split spoon. The sampler was driven into the ground under an energy of 350 ft.lbs.per blow, and the blows required for one foot of penetration were recorded. Portions of the sample were scraped clean of surface moisture, wrapped in tinfoil and stored in air tight plastic bags.

In addition, since the upper stratum of clay was too stiff for field vane tests, several 2 inch I.D. shelby tube samples were recovered so that measurements of shear strength could be made in the laboratory. These samples were sealed upon recovery, using low melting point wax.

Similar close sampling was carried out in hole 6 at the north end of the site. However, in the intervening holes, somewhat wider sampling intervals were used below a depth of 20 feet.

Bedrock was cored in holes 1 and 6. In the first instance, 100% recovery was obtained. However, in hole 6, the recovery was less satisfactory, although the drill was "on pressure" at all times. A slight artesian flow from bedrock was noted from each of these holes. In hole No. 1, the water rose 6½ feet overnight and froze in the casing at ground level. The flow in hole No. 6 was somewhat greater. After a period of 2½ weeks, water was still flowing from this hole at an estimated rate of 1/10 gallon per minute. The water was quite clear; no fine sand particles were noted. This absence of fines in the water should be expected, since the velocity of flow out of the borehole is approximately 0.25 feet per minute, or about 1/120 of the rate necessary for erosion of the fine sand particles in the granular strata overlying bedrock. Because there appeared to be no danger of subsoil erosion, no action was taken to stop the flow. It is proposed, however, to pack bentonite into the hole later in the spring after the water level in this swampy area has subsided.

Subsoil Conditions

The information obtained from the six borings, Dwgs. 2 to 7, indicate quite uniform conditions across the railway overpass site. The stratigraphical profile, shown in Dwg. 1, can be used to assist in the description of the soil types encountered.

In descending order from the ground surface, the soil strata are as follows:

(a) Medium dense medium grained sand:- This stratum extends from existing ground surface to elevations ranging from 280 to 275.5 feet. Except for a surface cover of topsoil and decayed forest vegetation, the sand appeared relatively clean. Some thin layers and pockets of topsoil were noted at deeper levels in this stratum, but this material represented a very small fraction of the total volume of sand. The water table lay in this free draining sand at approximate elevation 282 feet or about 4 feet below ground level. Since the area is quite flat and poorly drained and since the sand layer is not thick, marked variations in this water level should be anticipated.

(b) Stiff Clay:- This stratum is the predominate soil type at this location. It extends from the bottom of the sand layer, described above, to a depth of about 40 feet, or to approximate Elev. 243 feet. Down to a depth of 30 feet, it contained occasional thin partings or layers of fine sand and some tiny sulphide specks. Since these black specks have been noted in the marine or brackish water deposits of Cornwall, Ottawa and Moosonee, this clay is also believed to be a marine sediment.

Below a depth of 30 feet, some tiny grits or pieces of fine gravel were noted in the clay and at greater depths, a transition zone, containing intrusions of more silty clay or of pure silt, was encountered. The presence of silt is indicated by the lower moisture contents noted generally below about 35 feet.

Above 30 to 35 feet, the clay was quite plastic, with a liquid limit of the order of 50% and a plastic limit of approximately 22%. The natural moisture content of the soil was well below the liquid limit at all depths; the undrained shear strength of the clay was generally in excess of 2000 p.s.f. Because of these physical properties, this stiff to very stiff clay can be considered to be in a highly over consolidated state. The results of two consolidation tests, on samples from hole 6, shown in Dwg. 8, appear to confirm this view. The recompression stages of these test measurements have been adjusted to allow for inevitable sample disturbance. The basis for this adjustment is discussed in greater detail in the discussion of soil properties which follows.

(c) Dense grey silty fine sand:- This stratum underlies the clay described above and it has a thickness of approximately 4 feet. It exists in a very dense state.

(d) Glacial Till:- This layer rests on bedrock and it consists of a very dense mixture of gravelly sandy silt.

(e) Bedrock:- Bedrock consists of sound limestone. Ground water under slight artesian pressure was encountered in this rock.

Discussion of Soil Properties

This discussion will be limited to the stratum of marine clay; its physical properties will be examined with the view of predicting its behaviour when subjected to the weight of the railway overpass and associated earth embankments.

In the previous section it was noted that the undrained shear strength of this soil was generally in excess of 2000 p.s.f. and that the natural moisture content was well below liquid limit measurements. Because of this fact, it was felt that this stratum exists in an over consolidated state, particularly at upper levels. The shear strength results, referred to above, were based on undrained triaxial test measurements performed under confining cell pressures approximately equivalent to those exerted when the bridge loading is applied. Unconfined compression tests were performed for academic interest only and merely indicate that this type of measurement underestimates strength for stiff fissured clays of this type.

Although these results are sufficient for estimating the safe bearing capacity for abutment footings and for appraising the stability of associated embankment approaches, additional information is required for predicting the magnitude of settlement to be expected under the weight of this overpass structure. The two consolidation test results, shown in Dwg. 8, assist to some degree in this regard. However, these curves appear to be affected, somewhat, by sample disturbance and, as a consequence, the clay may appear to be more compressible than is actually the case. Therefore, it is necessary to examine all the test data available in order that a more accurate appraisal of the stress history of this clay can be made.

Two methods are available for estimating the magnitude of the maximum prestress or preload experienced by a clay soil.* One graphical construction, suggested by Casagrande, is based upon an examination of the shape of consolidation curves for samples subjected to alternate applications and releases of load.(1) The other procedure conforms to some of the methods suggested by Schmertmann to correct for the effects of sample disturbance.(2) The construction in each case is indicated in Dwg. 8.

Applying the method suggested by Casagrande to the laboratory consolidation curves, preconsolidating pressures of 6700 and 4100 p.s.f., respectively, were obtained for the tests at depths of 11 and 22 feet. The corresponding existing overburden pressures at these levels are 900 and 1560 p.s.f. respectively. On the basis of reasoning by Schmertmann, the minimum possible preconsolidation pressures at these depths is 4800 and 2500 p.s.f. With these minimum values, it is assumed that the final slope of the pressure-void ratio curve conforms to the relationships applying during the natural consolidation of this clay. However, present soil mechanics knowledge suggests that the compression index, or final slope, of the consolidation curve usually underestimates actual field conditions. This would appear to be the case in this instance. The measured compression index of this clay is noted to have a value of approximately 0.287. According to Skempton's liquid limit relationship** the compression index corresponding to a liquid limit of 49% is 0.351. If this higher compression index value is applied in the construction suggested by Schmertmann, preconsolidation pressures of 9100 p.s.f. and 5500 p.s.f. are obtained for the 11 and 22 foot sample depths. The undrained shear strengths at these levels in hole No. 6 were found to be 2330 p.s.f. and 1490 p.s.f. respectively. The corresponding ratios of undrained shear strength to consolidation pressure, c/P, for these values are approximately 0.253 and 0.271 respectively. These are in reasonable agreement with the relationships between c/P and plasticity index, noted by Skempton''. It would appear, therefore, that the adjusted curves provide a better indication of the compressibility of the clay than do the uncorrected laboratory test values.

The conclusion to be drawn from this reasoning is that any loading applied by the bridge or embankments at this crossing will merely cause recompression of the clay. The pressure void ratio relationships will be as shown by the heavy lines in Dwg. 8. Although this view may err slightly on the unsafe side for the clay below about 30 feet, which may not have experienced the same degree of over consolidation, this latter material becomes siltier with depth and, therefore, will be less compressible. The settlement computations of the next section will assume the condition of recompression indicated by the heavy lines of Dwg. 8.

* - (1) Soil Mechanics in Engineering Practice - Terzaghi & Peck, P. 71.
(2) Estimating the True Consoldiation Behaviour of Clay from Laboratory Test Results - ASCE Proceedings Oct. 1953.

**- Soil Mechanics in Engineering Practice - Terzaghi & Peck P.66

''- The measurement of Soil Properties in the Triaxial Test Skempton and Bishop - P. 98

Discussion of Bridge Support

Two methods of bridge support exist at this site. One is to carry the abutments on H piles, bearing directly on limestone bedrock about 55 feet below existing ground surface. With this arrangement, no bridge settlement will occur. The principal objection to this scheme however, is that the ground will settle under the weight of the adjacent approach fill and hence an abrupt change in road level will develop between the abutment and the adjacent fill surface.

The other proposal is to found the abutments directly on the surface of the stiff marine clay or in the medium sand just above clay level. On the basis of shear strength measurements, the safe capacity of this clay is slightly in excess of 4500 p.s.f. Except for some saving of concrete, however, there would appear to be no advantage in designing for a higher soil pressure since the settlement of this structure will be determined, to a considerable degree, by the weight of the adjacent fill. This fill weight will be almost equal to 4000 p.s.f.

The thickness of compressible clay at this site is of the order of 36 feet, and it lies between approximate Elev. 279 and 243 feet. The existing pressure at Elev. 261 feet, the midpoint of this stratum, is of the order of 1750 p.s.f. The increment of pressure at this depth, due to the embankment and bridge abutment weights, is approximately 3200 p.s.f., or 1.6 t.s.f. Referring to the corrected recompression curves of Fig. 8, the estimated value of Mv, the coefficient of compressibility for this range of pressure increase is .0064 sq.ft./ton. The total long term settlement corresponding to these values is:

$$S = 36 \times 12 \times 1.6 \times .0064 = 4\tfrac{1}{2} \text{ inches.}$$

Because the clay stratum is quite thick and relatively impermeable, this movement will take place very gradually over a period of several years. No differential movement should be anticipated because the underlying soil appears to be similar on each side of the railway.

Although no stability analysis has been performed, there appears to be little doubt that the shear strength of the clay at this location is quite adequate to support the embankment weights proposed. Considering the fill weight as equivalent to the stress exerted by a strip footing, the ultimate capacity of the clay is equal approximately to 5.5 times its shear strength, or about 11,000 p.s.f.; the maximum bearing stress from the fill is about 3900 p.s.f.

Although the surface deposits of medium sand contain some thin layers of organic material, it is probable that much of this soil can be used for embankment construction. Drainage of this sand will be necessary before borrow operations can be carried out. The horizontal thrust exerted by the sand fill against abutments approximately 26 feet high will be about 10,000 p.l.f.

This estimate is determined from the expression:

$$P = \frac{1}{2} \gamma h^2 N_\phi \cos \delta$$

where γ is the estimated fill weight = 120 p.c.f.
h = 26 ft.
$N_\phi = \tan^2 (45° - \frac{\phi}{2}) = 0.27$ for the angle of internal friction $\phi = 35°$.
δ = the angle of wall friction taken as 17°.

This force will be resisted, for the most part, at or just below the base of the abutments. If the abutments are founded directly on the clay, an adhesion force slightly in excess of 2000 pounds per foot of footing width will be available. If the abutments are founded in the overlying sand, the sliding resistance will be approximately equal to 0.7 times the vertical pressure at footing level. This pressure will come from the dead load of the bridge and the vertical component of fill weight transferred into the abutment.

Conclusions

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

1) The proposed railway overpass is underlain by approximately 7 feet of medium slightly organic sand and then by a thick stratum of stiff to very stiff marine clay. Bedrock, consisting of limestone, lies about 55 feet below the present ground surface.

2) Bridge foundations can be placed directly on the upper surface of the marine clay or about 2 to 3 feet above this clay in the overlying sand. The safe bearing capacity at these levels is 4500 p.s.f. Some ground water will be encountered for footing excavations to clay level. This ground water lies in the upper stratum of sand; its level will vary depending upon the amount of rainfall received. It should be possible to depress this water prior to excavation by pumping from sumps placed adjacent to each abutment location.

3) Approximately $4\frac{1}{2}$ inches of settlement will result from the application of the bridge and fill loads to the soil. This movement will occur over several years and will involve recompression of the stratum of marine clay. No differential movement of consequence is anticipated.

(4) The strength of the marine clay is quite adequate for the support of the embankment approaches to this bridge. Much of the upper stratum of sand should be suitable for embankment construction.

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

WAT/lt
March 27, 1959.
J344

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

WILLIAM A. TROW AND ASSOCIATES

TABLE NO. 1

SUMMARY OF LABORATORY TEST MEASUREMENTS

Depth Feet	Shear Strength, s, ksf**, Nat. Unit Wt., γ, pcf. Hole No. 1	2	3	4	5	6	Natural Moisture Content % Dry Weight Hole No. 1	2	3	4	5	6
5 – $6\frac{1}{2}$							18			15.9		
7 – $8\frac{1}{2}$							32					32.9
10 – $11\frac{1}{2}$	s=2.28 γ=121.0			s=2.5 γ=122.5 s=1.9* γ=120U	s=2.5 γ=122.5 s=1.76 γ=119.2U	s=2.33 γ=115.1	30.2 LL=45.6 PL=20.2	30.4	37.2	28.3 31.8U	30.6 33.4U	35.1 LL=47.7 PL=22.0
15 – $16\frac{1}{2}$	s=1.97 γ=122.5		s=1.4U γ=116			s=2.0 γ=121.1	28.2 LL=47.7 PL=22.9		35.9 LL=52.5 PL=25.5	33.9	26.3	30.7
20 – $21\frac{1}{2}$					s=2.2 γ=122.5 s=1.9 γ=120U	s=1.49 γ=120.3	30.6	32.6	31.6		26.1 30.7U	31.3 LL=49.1 PL=21.3
25 – $26\frac{1}{2}$	s=1.96 γ=121.2					s=2.5 γ=116	32.1			29.2		29.9
30 – $31\frac{1}{2}$							30.8	31.8	29.7		29.4	26.4
35 – $36\frac{1}{2}$							22.4					20.5
40 – $41\frac{1}{2}$	s=2.95 γ=131.2						20.0					13.6
45 – $46\frac{1}{2}$							14.0					7.9

** – Undrained triaxial test except where noted.
* – Failure occurred in sand layer.

LEGEND: U – Unconfined Compression Test
LL – Liquid Limit
PL – Plastic Limit

PROJECT NO. J 344

DRAWING NO. 2

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

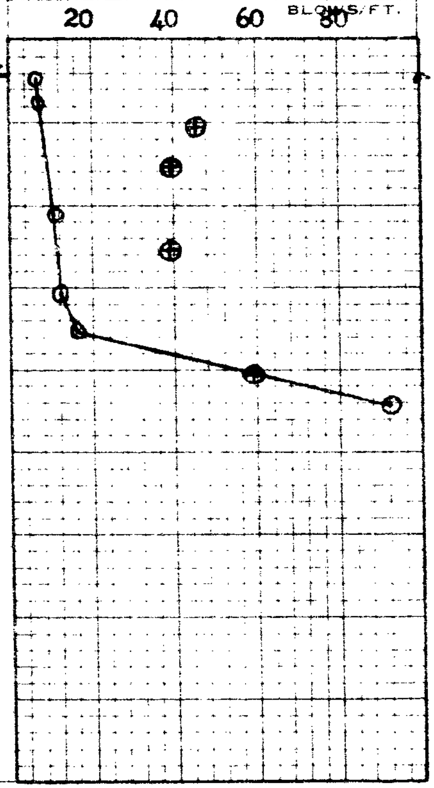
LEGEND
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (Qu) ⊕
- VANE TEST (C) AND SENSITIVITY (S) +
- NATURAL MOISTURE AND LIQUIDITY INDEX X
- LIQUID LIMIT
- PLASTIC LIMIT

PROJECT Hwy.401 Overpass C.P.R.
LOCATION Prescott, Ont.
HOLE LOCATION See Dwg.1
HOLE ELEVATION AND DATUM 285.9 Top of box culvert north of hole 6 - 284.7

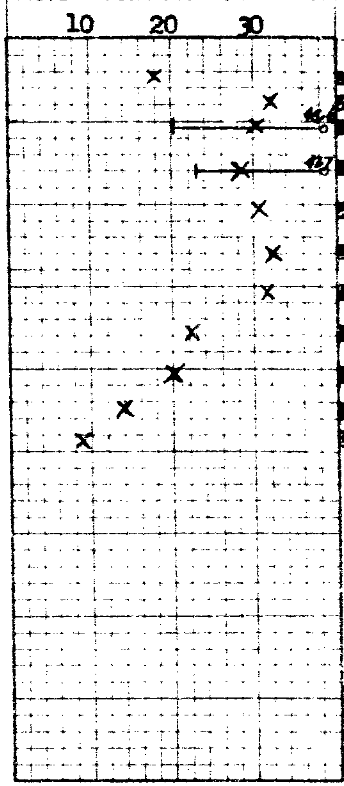
BOREHOLE NO. 1
FIELD SUPERVISOR MBS
DRILLER CB
PREP. MBS

DESCRIPTION	ELEV. FEET	DEPTH FEET
	285.9	0
Brown and grey med. sand. Some fine gravel & organic material	279.9	W.L.
Stiff grey marine clay: some black sulphide specks, shells & thin partings of fine sand to 30 ft.; tiny grits below 30 ft. becoming siltier below 40 ft.	243.4	10, 20, 30, 40
Dense grey silty fine sand	239.9	
Dense glacial till (stoney sandy silt)	232.4	50
Bedrock: limestone (100% recovery)	224.3	60
End of hole		70

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY



SAMPLE	NATURAL UNIT WT. P.C.F.
S1	
S2	
T3	121.0
T4	122.5
S5	
T6	121.2
S7	
S8	
T9	131.2
S10	
S11	

NOTES: 1) Boring advanced by driving BX casing and cleaning out with AX case. Hole dry to depth of 50 ft.

2) All shelby tube samples levered into ground.

3) With casing at 7 ft. water rose to 4'5" overnight: with casing at 44'5" water rose to 21 ft. overnight. Water rose to ground level overnight when drilling bedrock.

4) Organic cont. of upper sand of minor nature.

PROJECT NO. J344

DRAWING NO. 3

WILLIAM A. TROW AND ASSOCIATES

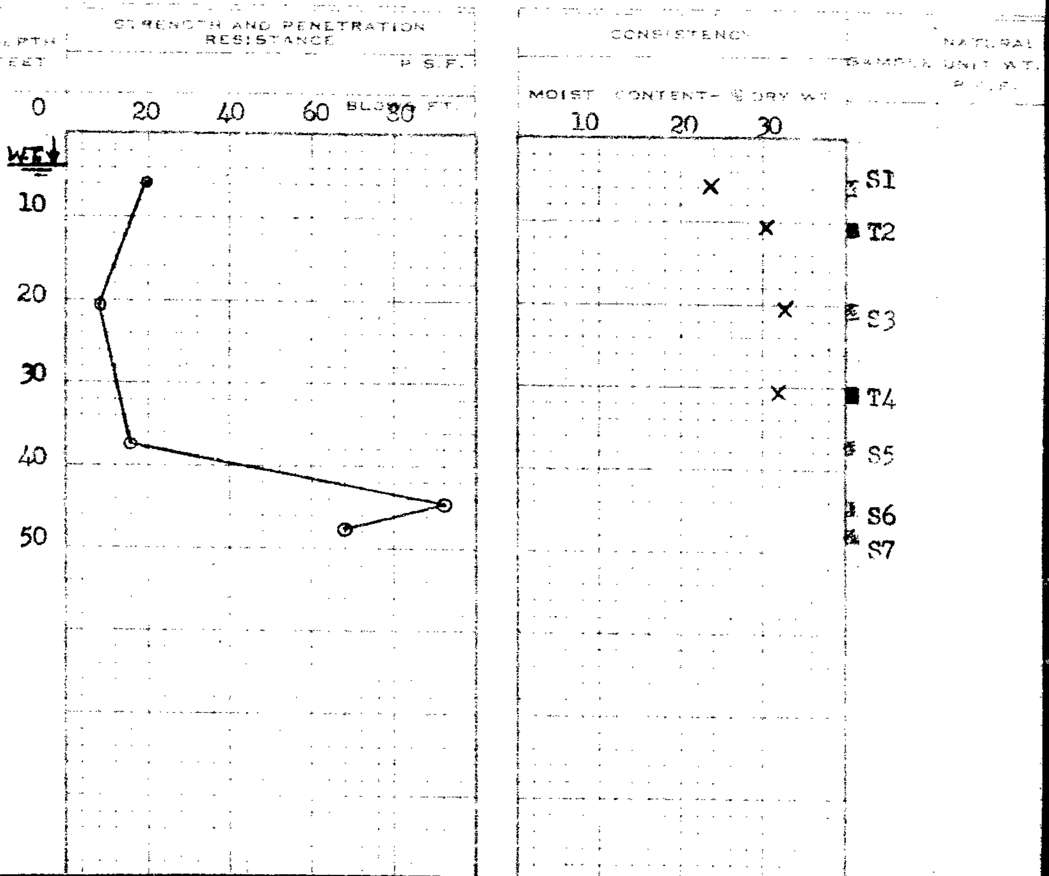
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

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

PROJECT: Hwy.401 Overpass CPR
LOCATION: Prescott, Ont.
BOREHOLE LOCATION: See plan
DATUM ELEVATION: 286.5 Top of culvert north of H-6 - 284.7

BOREHOLE NO. 2
FIELD SUPERVISOR: MS
DRILLER: CB
PREP: MS

SAMPLE	DESCRIPTION	ELEV. FEET	DEPTH FEET
		286.5	0
	Brown & grey med. sand: some fine to coarse gravel and some layered organic matter	278.5	
	Stiff grey marine clay: Some black sulphide specks and thin partings of sand above 30'. Grits in clay below 30 ': clay becomes siltier below 40 ft.	241.5	
	Dense grey silty fine sand, some fine angular stones		
	End of hole	238.0	



NOTES: Borehole cleaned out using AX casing: no wash water used.
2) Water table 48 hrs. after bore completed - 4 ft. depth
3) Organic content of upper sand of relatively minor nature; sand just slightly more compressible than clean sand.

PROJECT NO. J344

DRAWING NO. 4

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

LEGEND

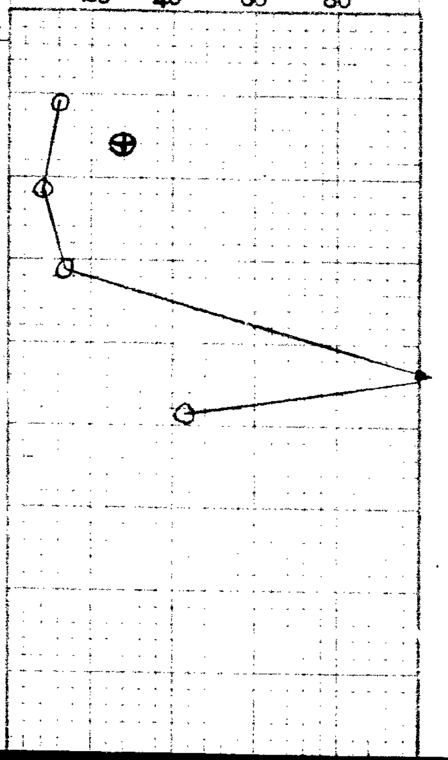
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (Qu)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

PROJECT Hwy. 401 Overpass C.P.R.
LOCATION Prescott Ont.
HOLE LOCATION See Plan
HOLE ELEVATION AND DATUM 286.2 Top of box culvert north of H6 - 284.7

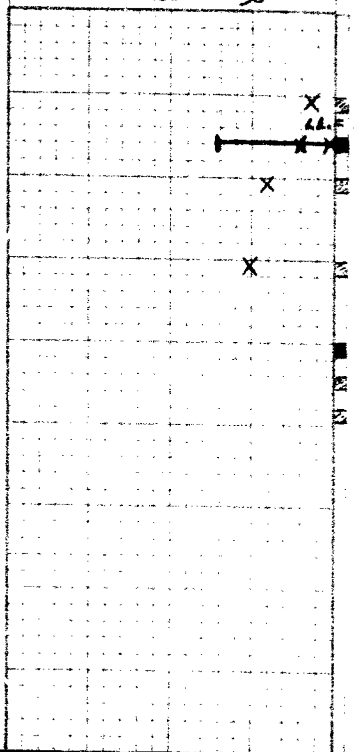
BOREHOLE NO. 3
FIELD SUPERVISOR
DRILLER
PREP.

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
		286.2	0
	Brown and grey medium sands, little silt, some organic matter		W.T.
		275.4	10
	Stiff grey marine clay: some sulphide specks and partings of fine sand above 30 ft; some tiny grits below 30 ft.		20
			30
		244.2	40
	Dense grey silty fine sand	240.4	
	Dense glacial till (stony sandy silt)	236.7	50
	End of hole		

STRENGTH AND PENETRATION RESISTANCE — Shear Str. 2000, 4000 P.S.F.; BLOWS/FT. 20, 40, 60, 80



CONSISTENCY — MOIST. CONTENT - % DRY WT. 10, 20, 30



SAMPLE	NATURAL UNIT WT. P.C.F.
S1	
T2	116.0
S3	
S4	
T5	
S6	
S7	

NOTES: 1) Samples T2-levered into ground
2) Hole advanced by cleaning out with AX casing; no water used.
3) Water level - 3'9" after hole complete

PROJECT NO. J 344

DRAWING NO. 5

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

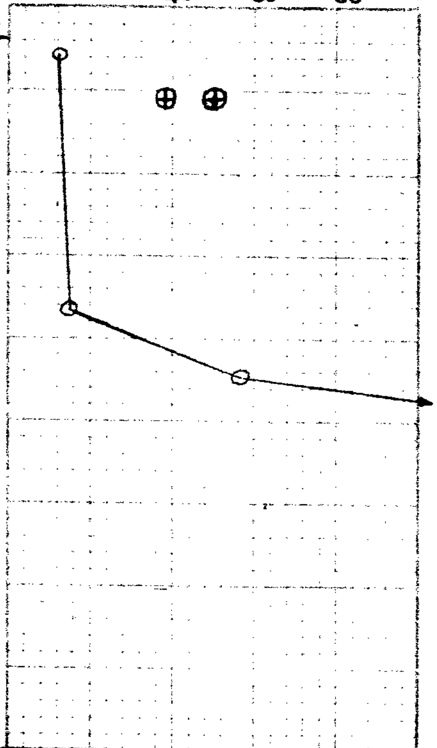
PROJECT Hwy.401 Overpass C.P.R.
LOCATION Prescott, Ont.
HOLE LOCATION See plan
HOLE ELEVATION 286.7 Top of box culvert north of H 6 - 284.7

BOREHOLE NO. 4
FIELD SUPERVISOR MS
DRILLER A
PREP. MS

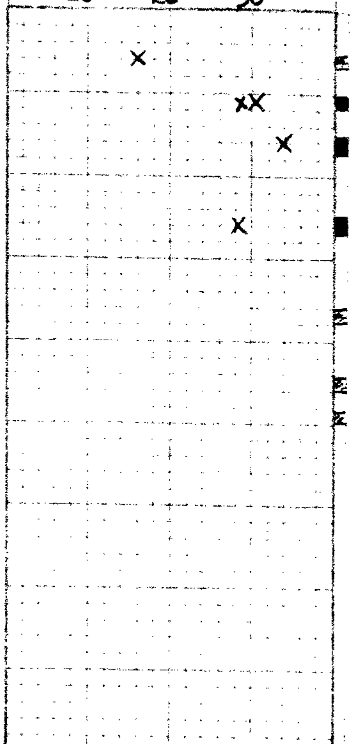
LEGEND
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (Qu) ⊕
- VANE TEST (C) AND SENSITIVITY (S) +S
- NATURAL MOISTURE AND LIQUIDITY INDEX X
- LIQUID LIMIT
- PLASTIC LIMIT

DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	286.7	0		
Brown and grey sands, little silt, with some fine & med. gravel (W.T.)	~ 278.7		S1	
Stiff grey marine clay, some sulphide specks and fine sand partings to 30 ft.; tiny grits below 30 ft.		10	T2	122.5
		20	T3	
		30	T4	
	243.7	40	S5	
Dense grey silty fine sand some med.	239.5			
Dense grey Glacial Till ((stones (Stoney sandy silt)	237.2	50	S6, S7	
End of bore.		60		

STRENGTH AND PENETRATION RESISTANCE — Shear Str. 2000 4000 P.S.F.; BLOWS/FT 20 40 60 80



CONSISTENCY — MOIST. CONTENT - % DRY WT. 10 20 30



NOTES: 1) Samples 2 to 4 levered into ground.
2) Boring cased to full depth; wash water used.
3) Water level 24 hrs. after end of bore - 3'11"

PROJECT NO. J 344

DRAWING NO. 6

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

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) +S
- NATURAL MOISTURE AND LIQUIDITY INDEX X
- LIQUID LIMIT
- PLASTIC LIMIT

PROJECT Hwy. 401 Overpass C.P.R.

LOCATION Prescott, Ont.

HOLE LOCATION See plan

HOLE ELEVATION AND DATUM 284.3 Top of box culvert north of H 6 - 284.7

BOREHOLE NO. 5

FIELD SUPERVISOR

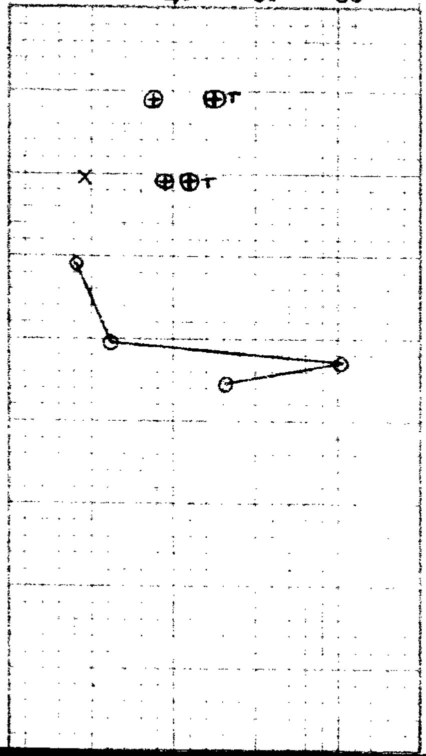
DRILLER

PREP.

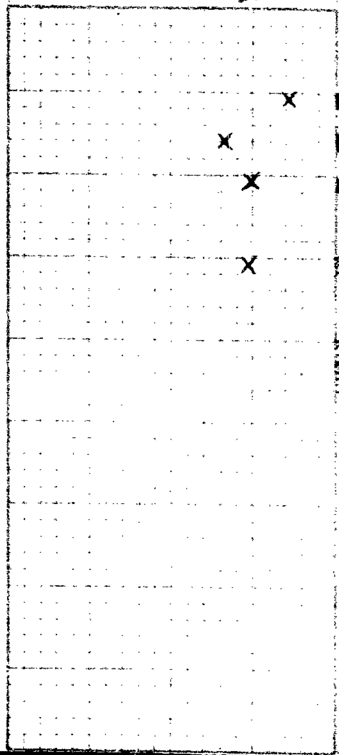
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
		284.3	0
	Brown and grey sands some fine and medium gravel	~279.3	
	Stiff grey marine clay some sulphide specks and sand partings to 30 ft.; some fine grits below 30 ft.; siltier below 40 ft.		10 / 20 / 30 / 40
	Dense grey silty fine sand with few fine stones	242.3	
	Dense grey glacial till-stoney sandy silt	240.3	
	End of bore.	238.2	50

NOTES: 1) Samples 1 & 2 levered into ground.
2) Boring cased; wash water used.

STRENGTH AND PENETRATION RESISTANCE — Shear strength 2000 4000 P.S.F.; BLOWS/FT 20 40 60 80



CONSISTENCY — MOIST. CONTENT % DRY WT. 10 20 30



SAMPLE	NATURAL UNIT WT. P.C.F.
T1	122.5
T2	
T3	122.5
S4	
S5	
S6	
S7	

PROJECT NO. J 344

DRAWING NO. 7

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

LEGEND

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

PROJECT Hwy. 401 Overpass C.P.R.

LOCATION Prescott, Ont.

HOLE LOCATION See plan

HOLE ELEVATION AND DATUM 283.2 Top of box culvert north of H 6 - 284.7

BOREHOLE NO. 6

FIELD SUPERVISOR

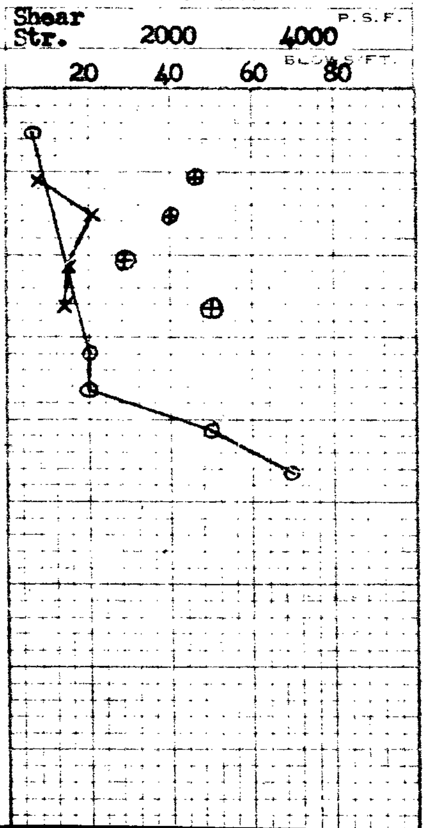
DRILLER

PREP.

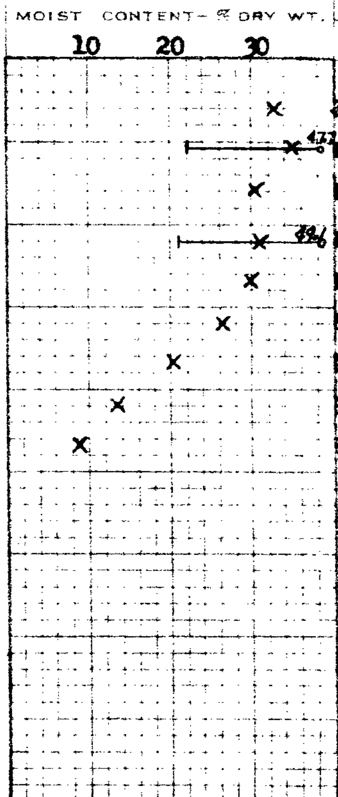
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
			0
	Brown & grey sand and gravel		
	Stiff grey marine clay: some sulphide specks to 24'0" some thin seams or layers of fine sand. some grits or tiny gravel below 30 ft.; becoming siltier below 35 ft.		10 / 20 / 30
	Dense grey silty fine sand-some fine (stones)		40
	Dense grey glacial till-stoney sandy silt		
	Bedrock: Limestone: 13% recov.-51 ft. 52% recov.to 56': 91%recov.to 60 ft.		50 / 60
	End of hole		70

NOTES: 1) Boring cased, wash water used
2) Water flowing from ground level after pipe withdrawn at end of bore. Flow after 2 wks. approx.1/10 gal.per minute.

STRENGTH AND PENETRATION RESISTANCE

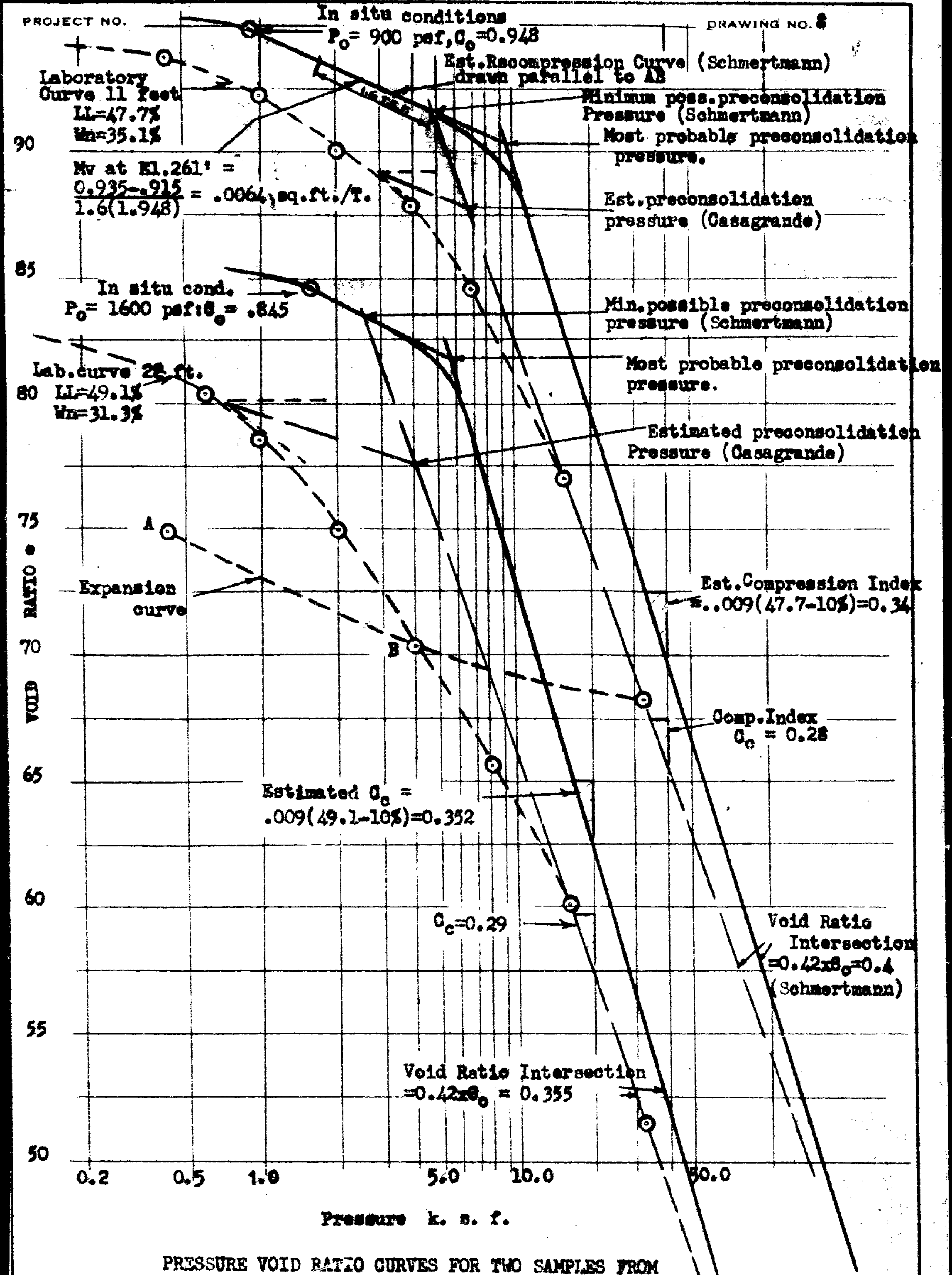


CONSISTENCY



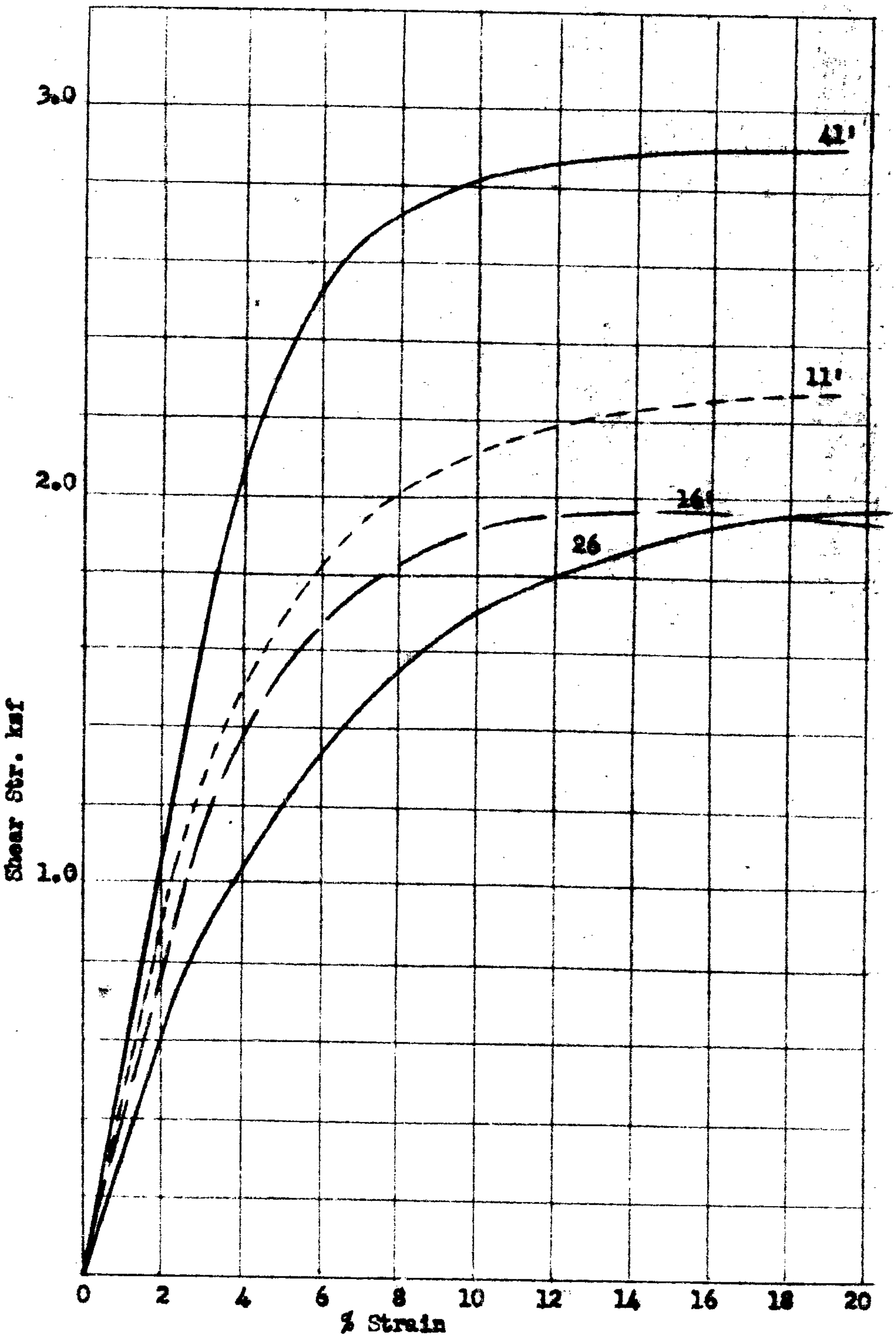
SAMPLE	NATURAL UNIT WT. P.C.F.
S1	
T2	115.1
T3	121.1
T4	120.3
T5	116
S6	
S7	
S8	
S9	

f 344



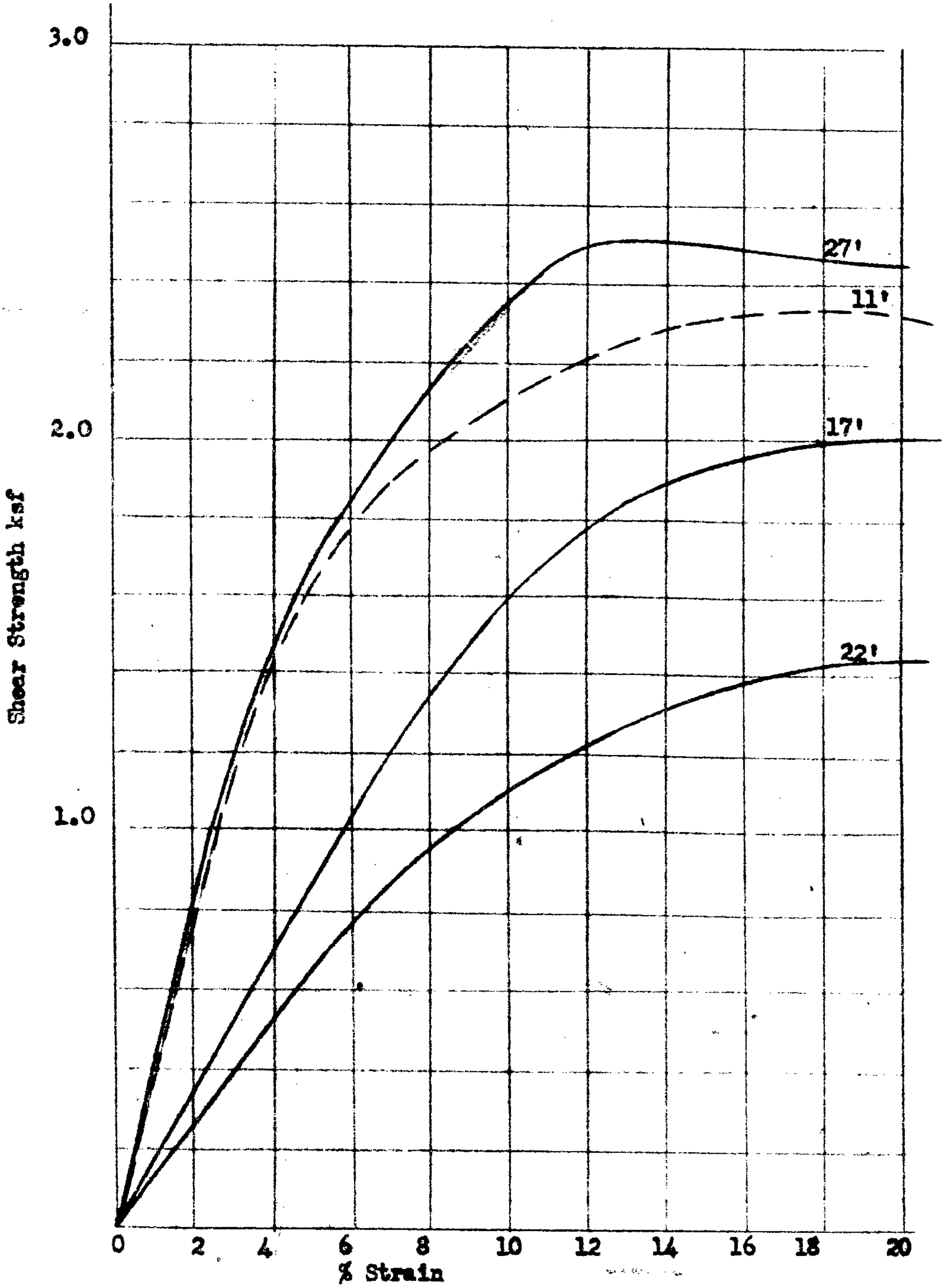
PRESSURE VOID RATIO CURVES FOR TWO SAMPLES FROM
HOLE 6

WILLIAM A. TROW AND ASSOCIATES



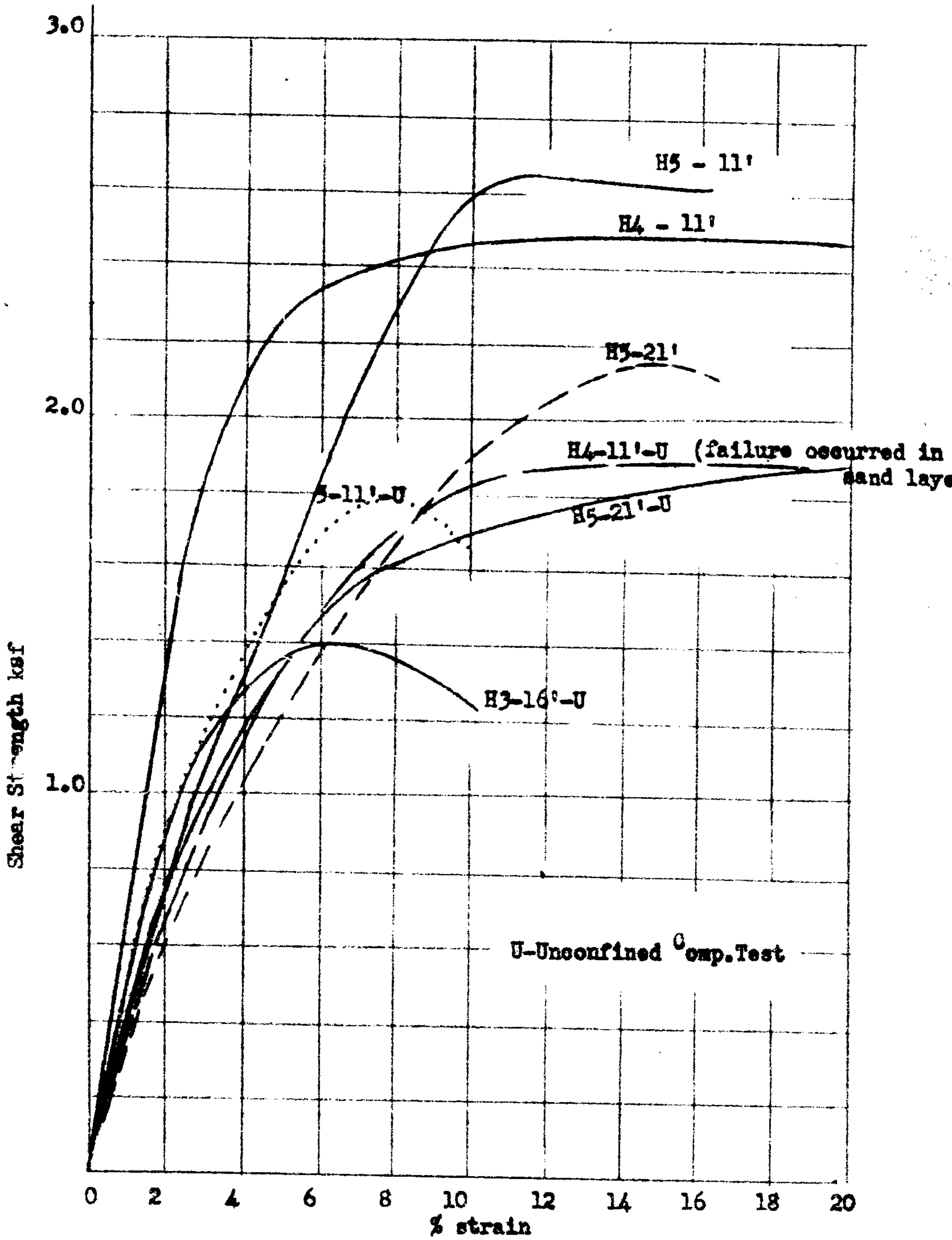
STRESS STRAIN CURVES FOR UNDRAINED TRIAXIAL TESTS ON SAMPLES OF MARINE CLAY
HOLE NO.1

WILLIAM A. TROW AND ASSOCIATES



STRESS STRAIN CURVES FOR UNDRAINED TRIAXIAL TEST ON MARINE CLAY
HOLE 6 (Cell press. = 20 psi)

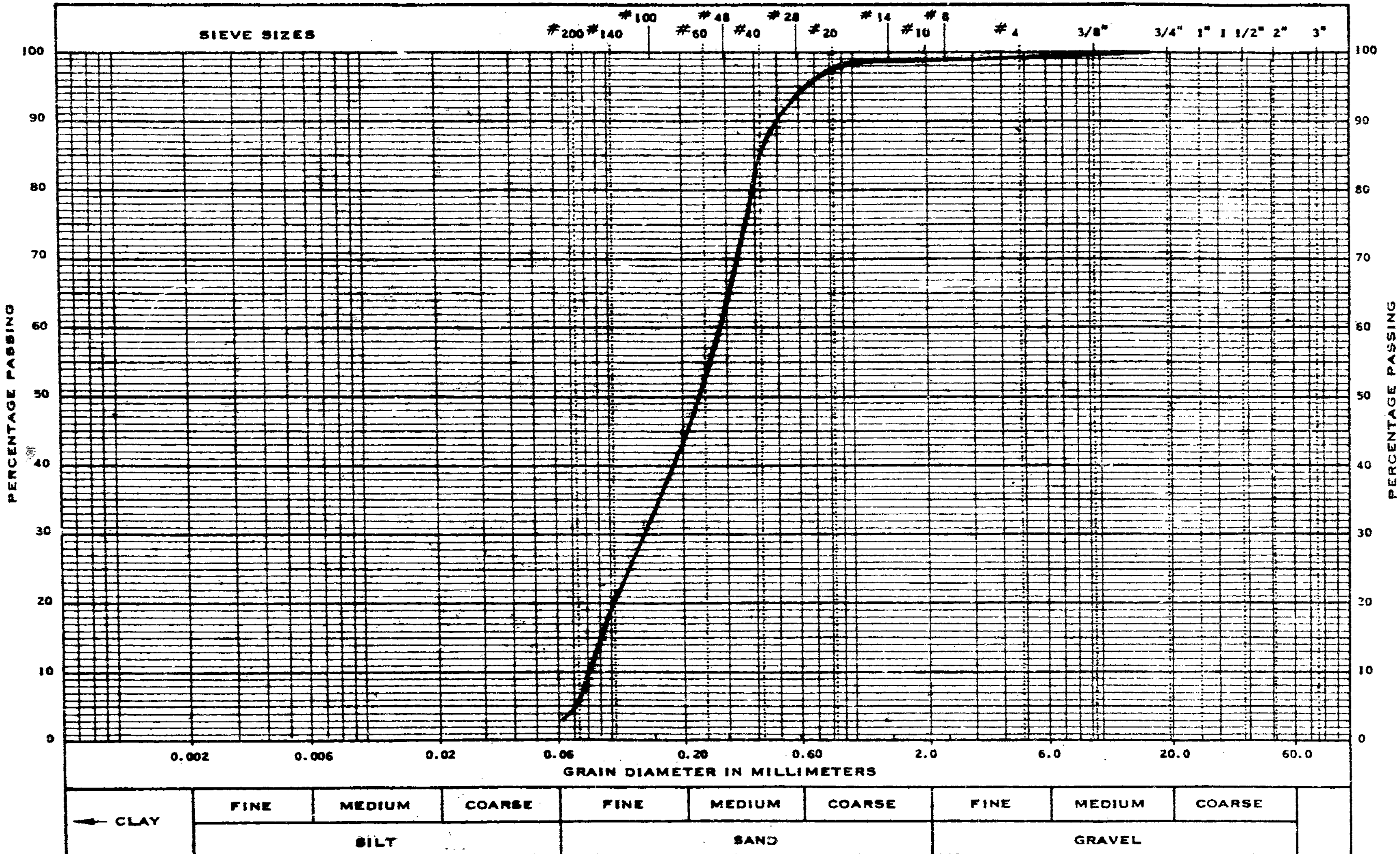
WILLIAM A. TROW AND ASSOCIATES



STRESS STRAIN CURVES FOR UNDRAINED TRIAXIAL AND UNCONFINED TESTS ON SAMPLES

WILLIAM A. TROW AND ASSOCIATES MARINE CLAY H 3-4-5 (Cell press.triax.Tests-20 psi)

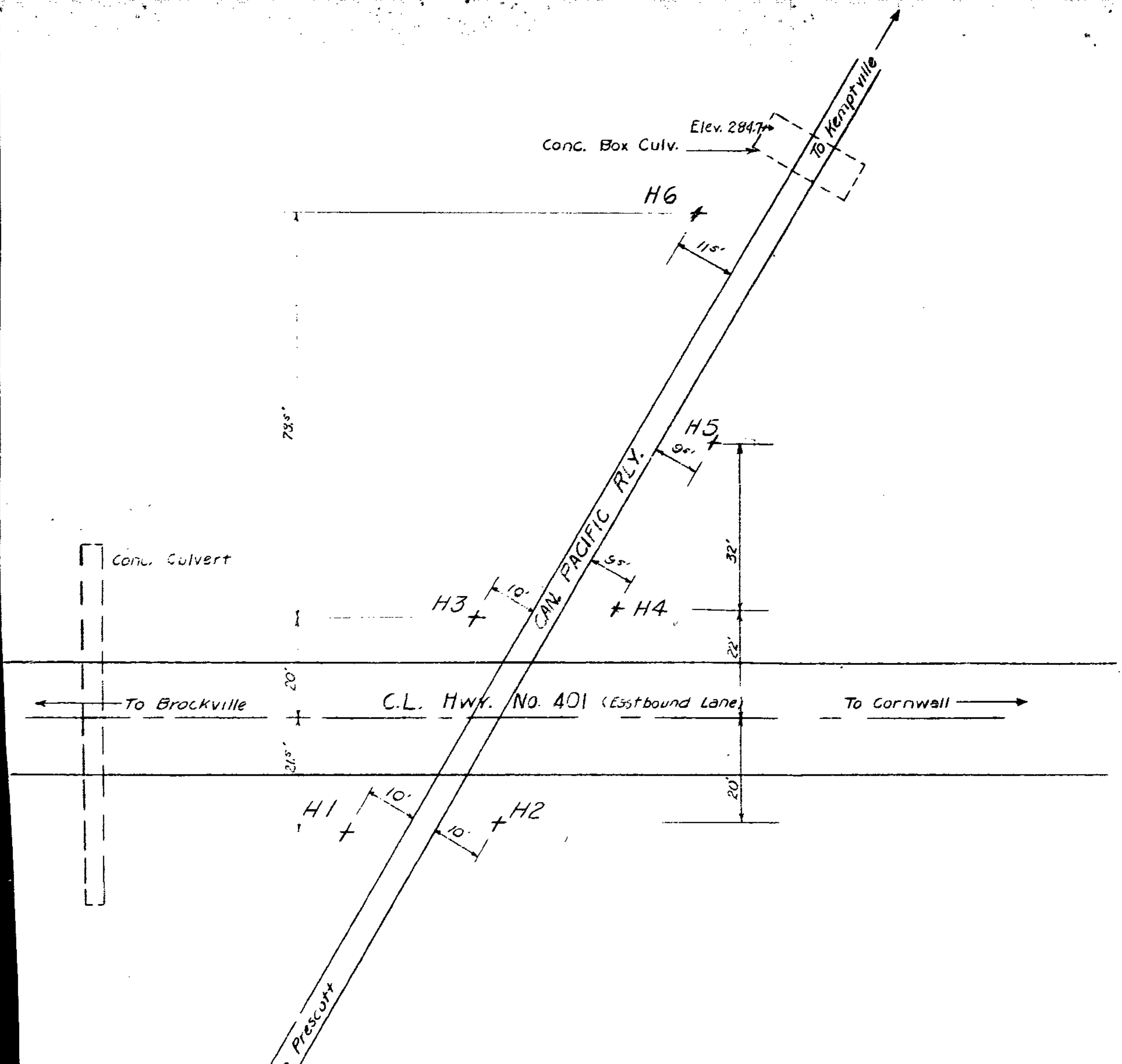
MECHANICAL ANALYSIS



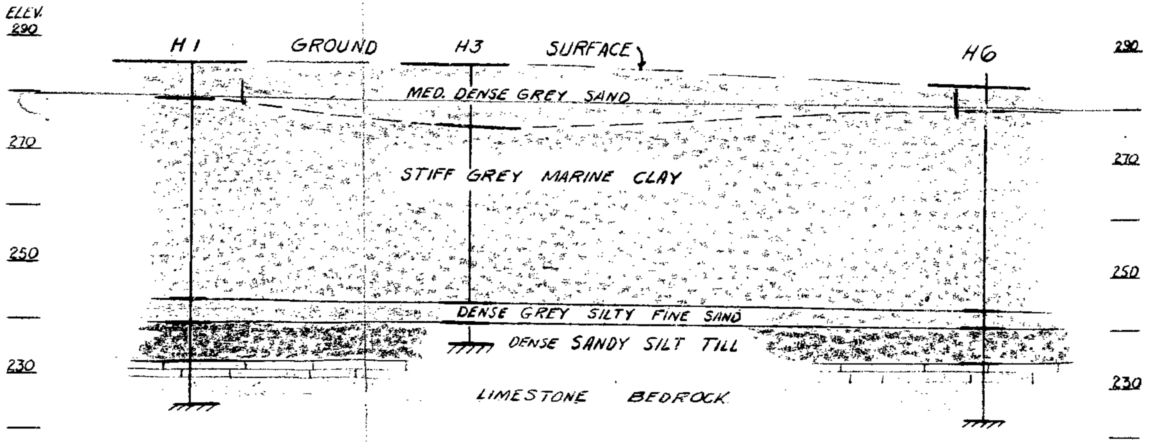
MODIFIED M.I.T. CLASSIFICATION

Mechanical Analysis for sand H 2 - 6 - 6½ ft.

WILLIAM A. TROW AND ASSOCIATES



PLAN OF OVERPASS SITE
scale 1" = 20'



ESTIMATED STRATIGRAPHY
scale 1" = 20'

280.0 bottom of footing [illegible]
placed [illegible] Apr. 23/69
[illegible]