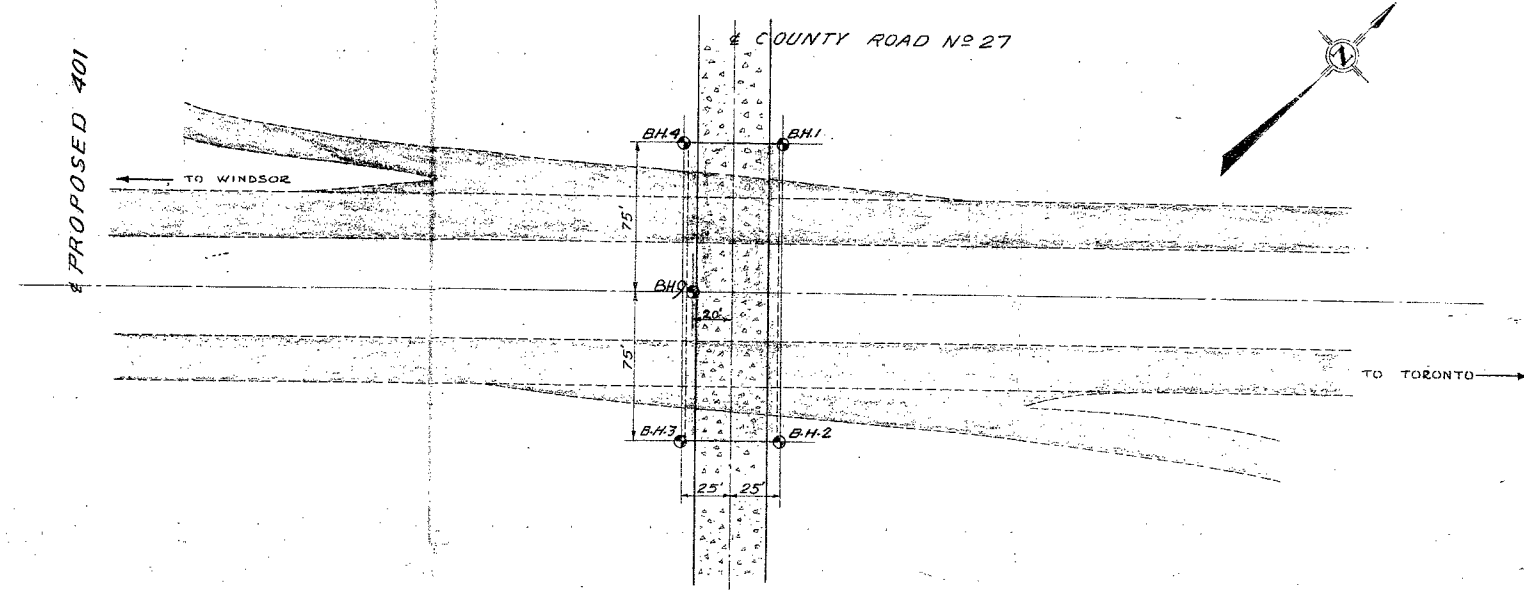
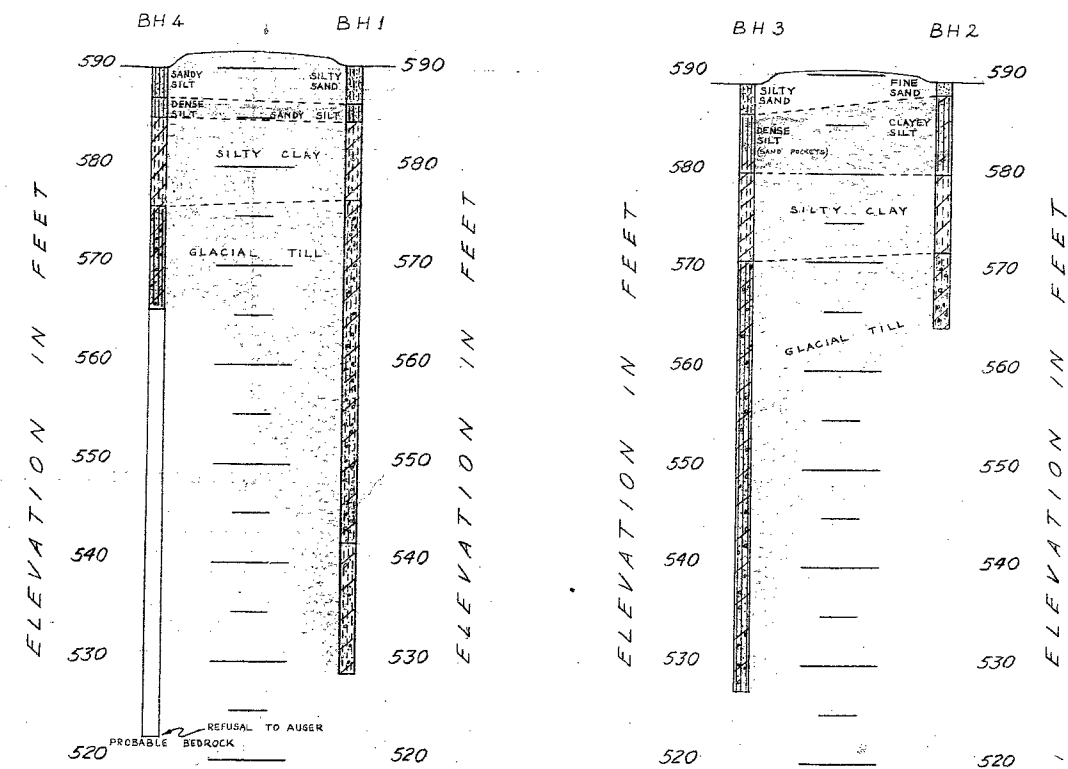


#59-F-235-C  
W.P. 297-59  
HWY. #401,  
RALEIGH TWP.  
BRIDGE #5, S.W.  
OF CHATHAM.



LOCATIONS OF BOREHOLES  
SCALE: 1IN=40FT

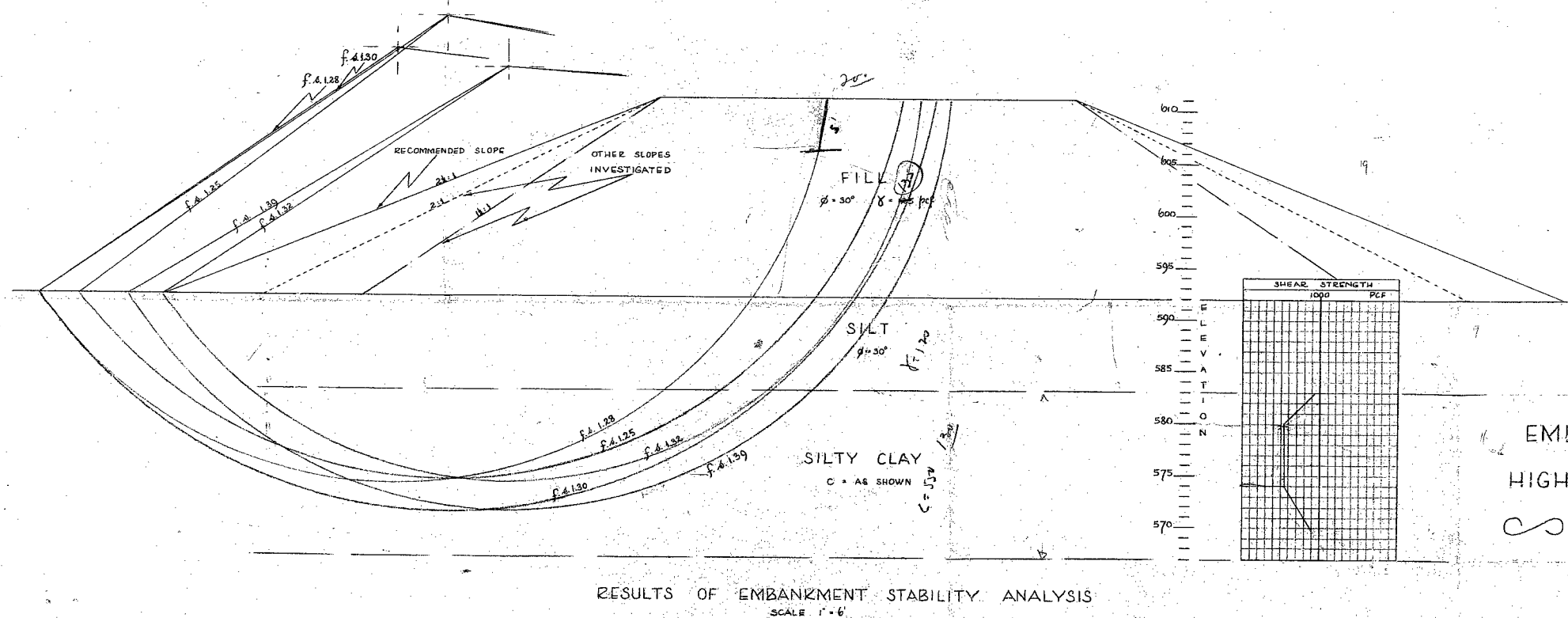
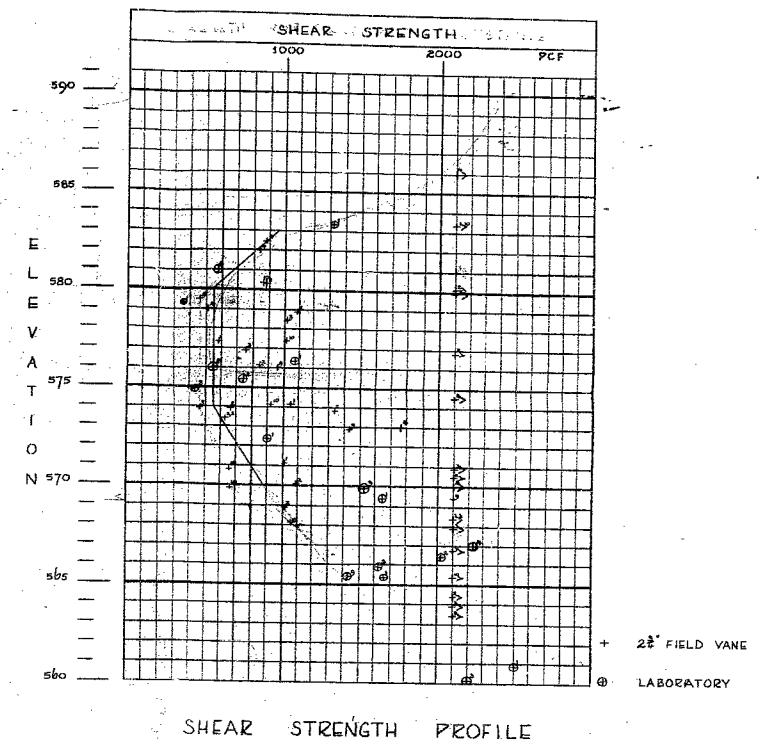
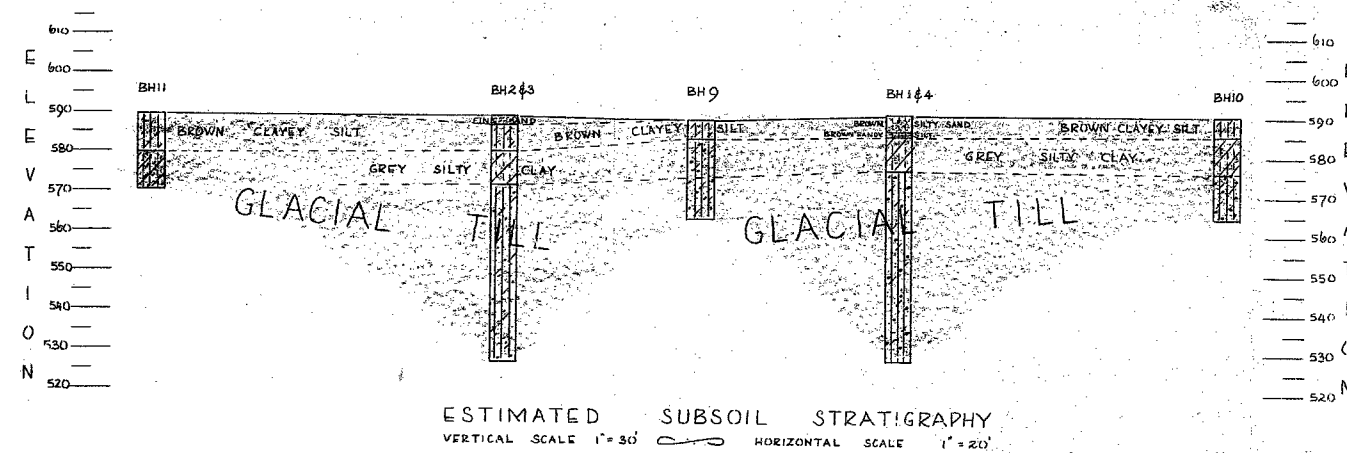
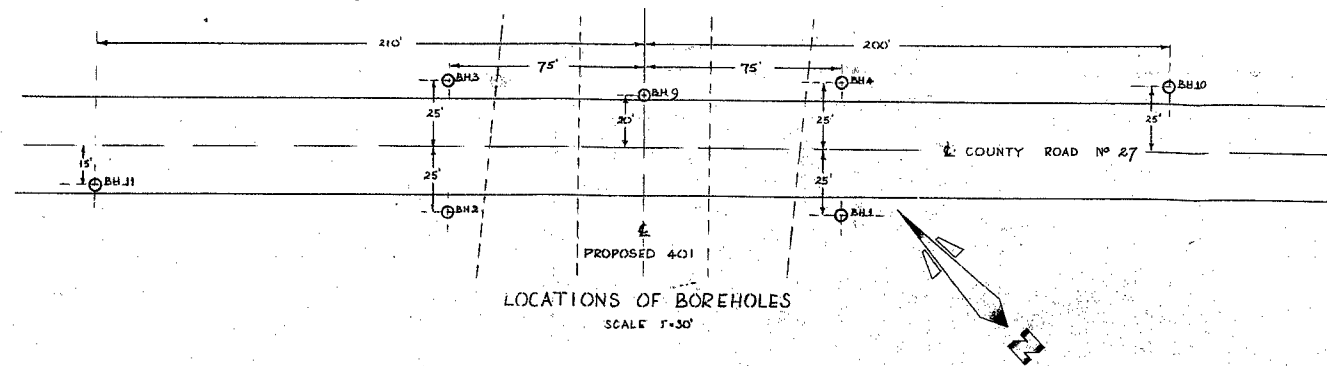


PROFILE BETWEEN BH4 & BH1

PROFILE BETWEEN BH3 & BH2

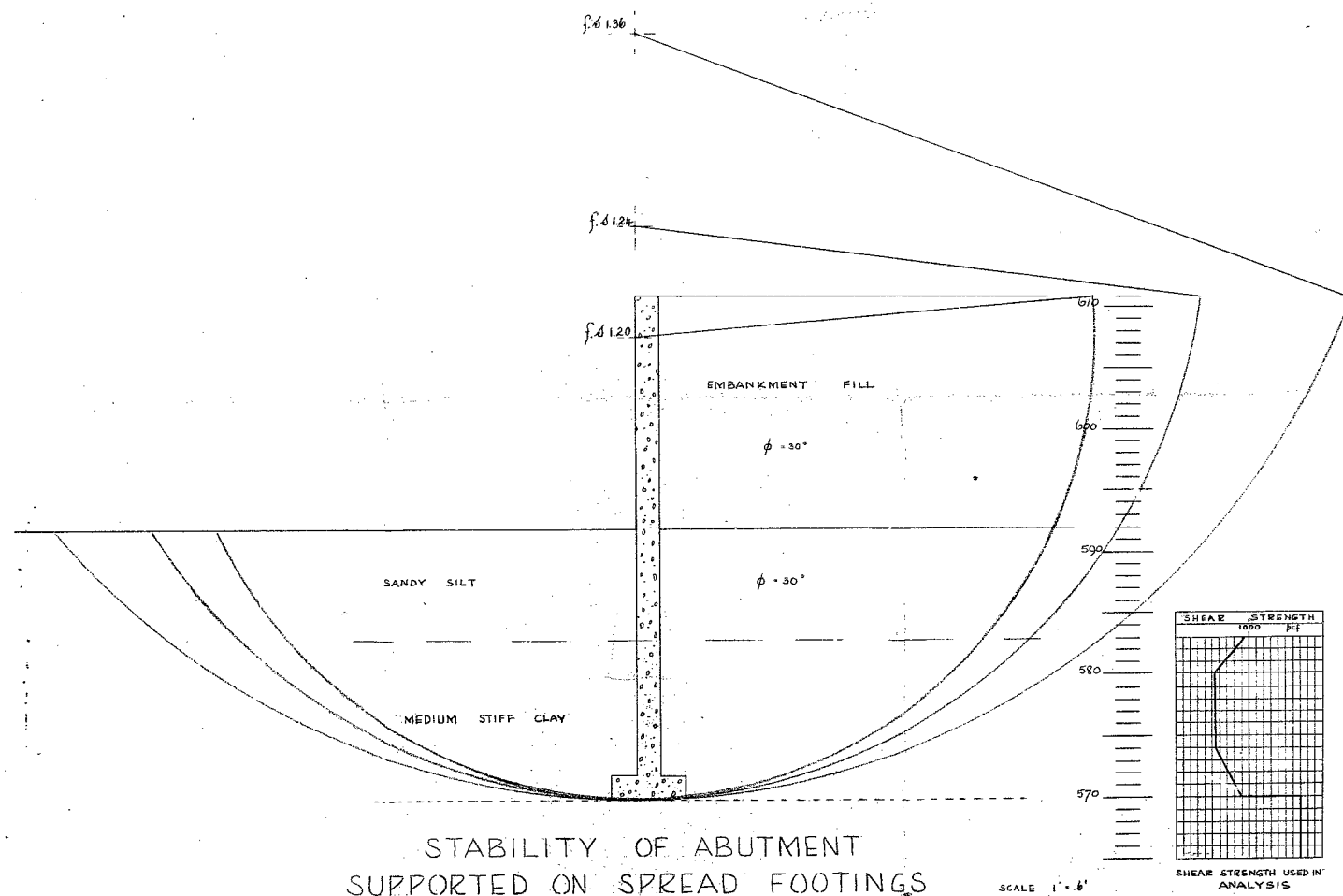
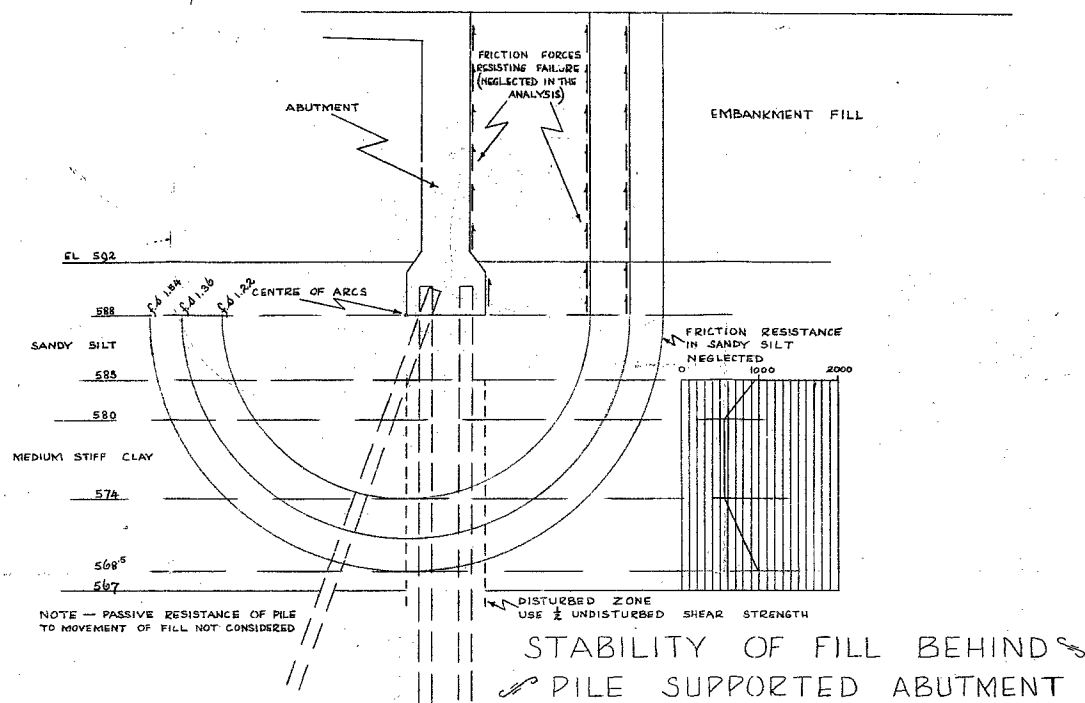
SCALES: HORIZONTAL 1IN=20FT  
VERTICAL 1IN=8FT

UNDERPASS OVER HWY NO 401 - COUNTY ROAD NO 27 NEAR CHATHAM  
JOB NO 419 DWG NO 1  
William A Trow & Associates  
SEPT 9 1959



EMBANKMENT STABILITY  
HIGHWAY 401 UNDERPASS  
S.W. OF CHATHAM

WILLIAM A. TROW & ASSOCIATES  
DWG # 2



WILLIAM A. TROW & ASSOCIATES LTD

DWG #3

BA 946 A

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS  
AND  
SOIL MECHANICS CONSULTATION

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

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

Project: J419

October 2, 1959

Mr. A. Rutka,  
Department of Highways of Ontario,  
Materials and Research Branch,  
Parliament Buildings,  
Toronto 5, Ontario.

59-F-235

Attention: Mr. L.G. Soderman, P. Eng.,  
Principal Soils and Foundation Engineer.

Re: Foundation Investigation, Proposed Underpass Structure  
Highway 401 at County Road 27, Southwest of Chatham.

Dear Sirs:

Enclosed herewith is our report on the soil conditions encountered under the above underpass site.

As stated in a recent interim letter to you the soil at this location exists generally in a competent condition with the exception of a medium stiff lacustrine clay deposit which lies between depths of about 8 and 20 feet below ground surface. This material is not strong enough to carry the weight of the bridge and, as a consequence, support must be found at greater depths.

Three alternative types of foundation have been investigated. One involves the installation of deep footings at elevation 568 feet or about 22 feet below ground surface. With this arrangement an abutment settlement of  $4\frac{1}{2}$  inches and a differential settlement, between the abutments and the centre pier, of 3 inches have been computed. Some differential settlement between the abutments and the adjacent fill, also, should be anticipated. The permissible gross bearing value has been given as 6100 psf. The stability of the adjoining embankment fill is maintained.

Another proposal is to support the abutments and, if desired, the centre pier on cylindrical displacement piles driven about 40 feet below ground level. A capacity of about 35 tons per pile has been estimated. A slight reduction in settlement should be anticipated with this arrangement. Batter piles to support the horizontal thrust of the embankment fill would appear to be necessary.

The other alternative is to support the abutments on H piles and-bearing on assumed bedrock about 70 feet below ground surface. No differential settlement of consequence should take place in the structure with this scheme but severe local settlement between the fill and the abutment should be expected.

Because of the weak nature of the clay above 20 feet, embankment side slopes of  $2\frac{1}{2} : 1$  are required. The upper levels of sandy clayey silt on the site should be suitable for use as embankment fill.

We hope that the contents of this report assist you in deciding upon a design for this structure. If we can be of assistance in amplifying any statement please do not hesitate to contact us.

Yours very truly,

*W. A. Trow*

William A. Trow, P. Eng.

WAT/kb  
ENC.

WILLIAM A. TROW AND ASSOCIATES LTD.

DEPARTMENT OF HIGHWAYS OF ONTARIO  
MATERIALS AND RESEARCH BRANCH  
PARLIAMENT BUILDINGS, TORONTO, ONTARIO

FOUNDATION INVESTIGATION  
PROPOSED UNDERPASS STRUCTURE  
HIGHWAY 401 AT COUNTY ROAD 27  
SOUTHWEST OF CHATHAM

Project: J419

Oct. 2, 1959

William A. Trow & Associates Ltd.

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ENCLOSURES

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FOUNDATION INVESTIGATION  
PROPOSED UNDERPASS STRUCTURE  
HIGHWAY 401 AT COUNTY ROAD 27  
SOUTHWEST OF CHATHAM

This report describes the soils investigation carried out at the above site. The types of soils that exist at this location and their physical properties are outlined. Types of underpass structure foundations most compatible with these soil conditions are discussed in detail. The stability of the approach embankments to the structure is analysed.

Description of Site

The route of proposed Highway 401 intersects Kent County Road No. 27 in an area of relatively level farmland. Shallow ditches parallel the existing county road and drain runoff water into nearby Jeannette Creek. No surface water was evident during this investigation and Jeannette Creek was nearly dry.

The proposed underpass structure will carry the county road over Highway 401 which will be a four lane divided roadway. The underpass structure will consist of a retaining wall type abutment at either side of Highway 401 and a centre pier between the east and westbound lanes. Approach embankments of the order of 19 feet maximum height will be required.

Soil Types Encountered

Drawing 1 contains profiles showing the soil conditions that exist at each abutment location. Drawing 2 presents a soils profile along the C.L. of the county road.

Inspection of these drawings shows that, in general, a shallow stratum of brown sandy clayey silt covers the area and it overlies a deep deposit of grey silty clay. The brown silt is either dry or moist depending on location and its water content probably will vary depending upon weather conditions. Because of this it either appeared soft or dense. Sand was found overlying the silt at some locations. The thickness of this silt stratum is of the order of 5 feet north of the proposed highway and approximately 9 feet to the south.

The silty clay deposit, underlying the brown silt can be considered to comprise two distinct layers or soil types at most locations. The upper zone contains very little or no sand and gravel sizes and it was found to be stratified or varved. The lower zone is of glacial origin and is a heterogeneous mixture of silty clay to clayey silt with numerous sand and gravel sizes. The limits of the zones can be seen on the profiles.

Immediately below the brown silt, the grey silty clay is stiff to very stiff in consistency. It quickly softens with depth to a medium stiff state and this condition extends to a depth of about 15 to 20 feet. The till then becomes stiff to very stiff and remains so for a considerable depth.

Refusal to augering and sampling was reached at elevation 522.3 in hole 4. This is equivalent to approximately 68 feet below ground level and it probably represents the bedrock contact. This agrees with a depth to rock of about 70 feet, reported by the Department of Highways, for a nearby site and with the experience of well drillers in the area.

Although a number of borings were left open over a period of days, water infiltration was so slow that recorded levels have little significance. The upper boundary of the medium stiff silty clay zone probably represents the lowest level of the water table and is the level assumed to exist during the investigation. This change to medium stiff clay occurs at depths of 8 to 10 ft. below ground surface.

The ground water level in the bedrock lies about at elevation 570 feet as determined by observations in hole 4. The water level stabilized quickly at this depth after the boring was made. Natural gas was given off.

#### Foundation Considerations

1) Spread Footings. Holes 1, 2, 3, 4 and 9 were put down at footing locations. The borehole logs, drawings 4 to 10, illustrate the types of soil encountered and their structural properties. Spread footings are feasible only if placed on the stiff silty clay till found below elevation 570.

The safe gross bearing value to apply at this depth of approximately 20 feet can be determined from the expression

$$q = \frac{7.5 C}{F} + \gamma d$$

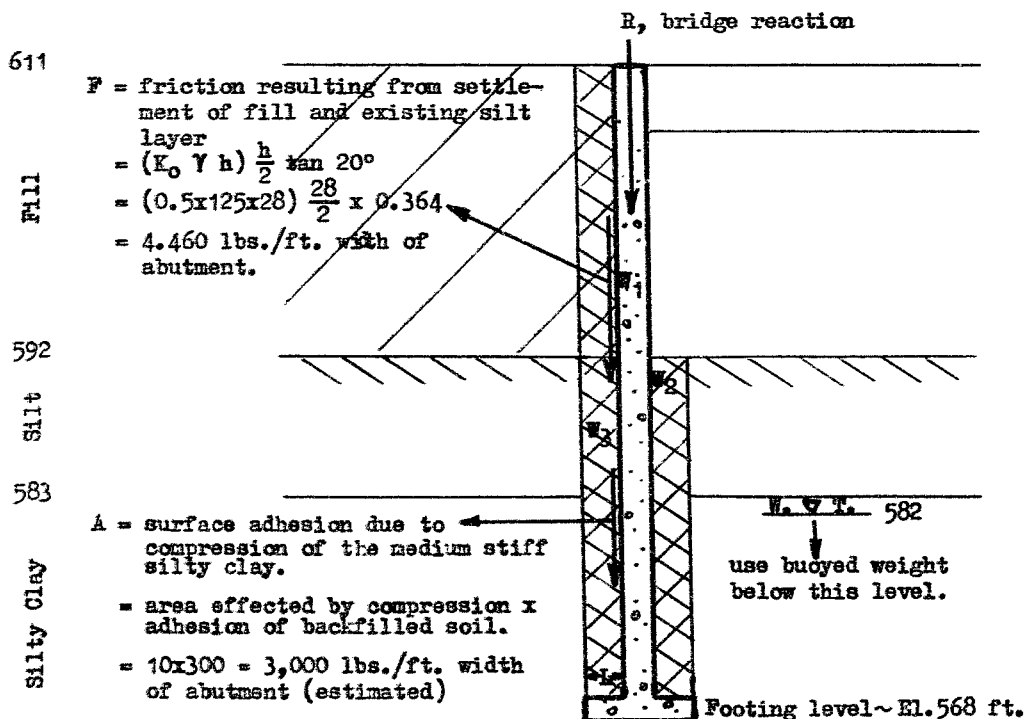
where  $C = 1600$  psf is the shearing resistance of the soil at this depth.  $7.5 C$  is the net increase in bearing pressure that can be applied.

$F = 3$  is the required factor of safety.

$\gamma d$  is the weight of soil above footing level = 2100 psf approx.

Therefore the gross safe bearing value  $q \approx 6100$  psf.

Computations of footing load must account for the bridge reaction and the dead load of the entire abutment or pier including those parts below ground level. In addition the friction forces transmitted to the abutments as the fill and original ground settles must be considered also. The weight of soil backfill that rests on the footing should be added to arrive at the total footing load except in those instances where this amount is less than the friction forces. The accompanying sketch outlines the computations involved.



$$\text{Total footing load} = R + W_1 + W_2 + W_3 \quad \left. \begin{array}{l} \text{or} \\ R + W_1 + W_2 + F + A \end{array} \right\} \text{whichever is the greater}$$

$$F + A = 4,460 + 3000 = 7,460 \text{ lbs./ft. width of abutment}$$

The critical length of heel,  $L$ , occurs with  $F + A = W_3$

$$\text{or } L = \frac{7,460}{[(19 \times 125) + (10 \times 125) + (14 \times 70)]} \sim 1 \frac{2}{3} \text{ feet}$$

where heel of retaining wall is less than  $1 \frac{2}{3}$  feet wide use

$$\text{total footing load} = R + W_1 + W_2 + (7,460 \times \text{width of abutment})$$

where heel of retaining wall is greater than  $1 \frac{2}{3}$  feet wide use

$$\text{total footing load} = R + W_1 + W_2 + W_3$$

Stability analyses of a retaining abutment supported on spread footings placed at elevation 570 ft. showed a minimum factor of safety of 1.20 against overturning (see dwg. No. 3). To ensure a factor of safety of at least 1.3 the abutment foundation will have to be keyed into the stiff soil below elevation 570. A depth of penetration of 2 feet appears warranted. Therefore footings will have to be placed at elevation 568. This is subject to visual inspection of the footing excavations to ensure that the 2 feet of penetration into stiff soil is maintained in areas where the stiff contact may be depressed slightly.

Spread footings for the centre pier can be placed at the surface of the stiff silty clay. The results of borehole 9 indicate this to be at elevation 570.

Settlements of the underpass structure and approach embankments have been estimated in Appendix 2. Although the accuracy of these estimates is influenced by the variations in in-situ conditions from those assumed or indicated by test results, they are useful for indicating the order of movements, and particularly of differential movements, to be anticipated. It has been shown that the estimated long term settlement of the abutments will be about  $4\frac{1}{2}$  inches and that this movement will occur at a decreasing rate for about 30 years; one half of it should be complete in 7 years. Most of this movement can be attributed to the weight of the adjacent embankment and, as a consequence, very little relief can be gained by reducing the permissible bearing pressure. A very slight reduction in settlement will result if the abutments are supported on displacement piles floating in the glacial till at 40 feet.

Immediately adjacent to the abutments the fill should settle about  $6\frac{1}{4}$  inches or about  $1\frac{1}{4}$  inches more than the ends of the bridge. This additional movement results from the consolidation of the medium stiff clay above 20 feet. It should be complete after about 2 years. Presumably the resulting slight bump at the bridge entrance can be corrected permanently at this time.

The estimated settlement of the centre pier has been computed to be  $7\frac{1}{4}$  inches and therefore the maximum differential settlement between the pier and the adjacent abutments will be about  $3\frac{1}{4}$  inches. It will be attained after about 30 years. This differential movement can be reduced only by supporting the abutments on end-bearing piles driven to bedrock.

If wing walls were to be supported directly upon the ground surface, the ends adjacent to the abutments would settle  $6\frac{1}{4}$  inches and the ends near the toe of the slope would settle 1 inch. As a consequence, the top inside end of the wall would tend to bear against the abutment. Because of this tendency for differential movement and of the relatively weak condition of the soil between about 8 and 20 feet, it would appear desirable to incorporate the wing walls as an integral part of the abutment.

About 50 feet back from the abutments the fill should be expected to settle a total of 10 inches. Between this 50 foot distance and the abutment, the settlement will diminish gradually to  $6\frac{1}{4}$  inches. This movement will be

complete in 30 years. Since most of this movement will take place in the compressible soil above 20 feet and below 50 feet, where the drainage paths are relatively short, initial consolidation will be rapid. It is estimated that about  $6\frac{1}{2}$  inches of this settlement will be complete in about 2 years which corresponds to a state of 90% consolidation in the upper clay and about 50% consolidation in the soil below 50 feet.

The foregoing computations have been made assuming embankment side slopes of  $1\frac{3}{4} : 1$ . Unfortunately, as the analysis progressed, it was found that slopes of  $2\frac{1}{2} : 1$  were required. The effect of this change on the settlement estimates will be negligible.

2) Displacement Pile Foundation. Large diameter displacement piles could be driven into the stiff silty clay existing below elevation 570. The optimum pile length appears to be 40 feet. The capacity of piles driven to elevation 550 is dependent on the tip bearing capacity and the adhesion of the silty clay to the sides.

Because the medium stiff silty clay above elevation 570 will compress under the weight of embankment fill, the adhesive resistance of this material to movement of the piles will be converted into a negative friction tending to pull the piles down. The capacity of the piles therefore will be generated only in the stiff silty clay below elevation 570. The minimum adhesion over the 20 foot depth from 570 to 550 should be equal at least to the remoulded strength which is considered to be about 1000 psf. Accordingly, the pile capacity =

$$q = \frac{C N_c A_{tip}}{F.S.} + 1000 A_{pile}$$

where  $C$  = soil cohesion at pile tip = 2000 psf

$N_c$  = bearing capacity factor = 9

$A_{pile}$  = surface area of piles between elevations 550 and 570.

$A_{tip}$  = area of pile tip

F.S. = factor of safety, normally = 2

for a pile 1 foot in diameter

$$\text{pile capacity} = \frac{2000 \times 9 \times 0.785}{2} + 1000 \times 3.14 \times 20 = 69,800 \text{ lbs.}$$

Therefore the safe capacity of piles 40 feet long and one foot in diameter is 35 tons.

In computing the loads to be placed on the piles, consideration must be given to the forces of friction and adhesion transferred to the abutments as fill and overlying ground settles. It is assumed conservatively that the

friction and adhesive forces act on the area of the backwall and on the rear surface area outlined by the pile group above elevation 570 ft. The forces computed on this basis will be the same as those outlined in the previous section on spread footings. Accordingly, a force of 7,460 pounds per foot width of abutment will have to be added to the bridge reaction and abutment dead weight.

The stability of the embankment, retained by the pile-supported abutments, has been analysed in drawing 3. In this computation no allowance has been made for the arching effect taking place in the embankment fill or for the resistance to soil movement provided by the piles. It is assumed also that the clay adjacent to the piles has been remoulded to a lower strength. The lowest factor of safety under these circumstances was found to be  $F = 1.22$ . If allowance had been given to the permanent areas of resistance a much higher safety factor would have been obtained.

It is assumed that batter piles will be installed to resist the embankment fill pressures and the traffic impact. The horizontal thrust from the fill should be of the order of 4.2 tons per lineal foot of wall.

As stated in the foregoing section, the settlement of the abutments, if founded on displacement piles, should be slightly less than was computed for the condition of footing support directly at elevation 570 feet.

3) End Bearing H Piles. Steel H piles can be driven to refusal in the bedrock which underlies the site. The surface of this assumed rock is at elevation 522 approximately. This is some 70 feet below the general ground level. The capacity of this type of pile will depend on its structural properties acting as a short column.

If the abutments are supported on steel H piles then settlement will be negligible. Since the estimated settlement of the centre pier is very small with either a spread footing or a displacement pile foundation, the advantage of using H piles to support the pier is limited.

The approach embankment will settle the amounts indicated in a previous section. Since the abutments will not settle when supported on H piles, an appreciable change in level, of the order of 6 inches, will develop ultimately at the junction of the embankments and the bridge. About  $2\frac{1}{2}$  inches of this movement will be complete in two years time, about  $4\frac{1}{2}$  inches in 7 years time and the remainder will continue for an overall period of about 30 years.

#### Embankment Stability

Quite conservative assumptions had to be made concerning the fill material that will be used to form the embankments. Since the type of soil has not been decided on, material, with an angle of internal friction of  $30^\circ$  and possessing no cohesion, was considered. The surface sands and silts found at this site would be suitable for fill and would possess a certain amount of apparent cohesion especially when in a compacted state.

Drawing 2 summarizes the soil conditions and soil properties assumed in the stability analysis. An embankment 19 feet high with a top width of 40 feet was considered.

The analysis showed that embankments with side slopes of  $2\frac{1}{2} : 1$  will have a least factor of safety of 1.25 against a circular arc type of failure for the conditions assumed. It is felt that the fill material will possess properties more favourable than those considered and also that some cohesion does exist in the surface silt deposit. None was considered in this analysis. Only 50 psf of cohesion is required throughout the fill and silt layer in order to obtain a factor of safety of at least 1.3.

A slope of  $2\frac{1}{2} : 1$  is recommended for the approach embankments.

#### Summary of Observations and Conclusions

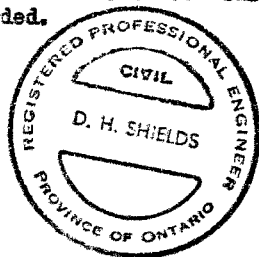
Soil conditions at the underpass site consist of from 5 to 9 feet of sandy clayey silt overlying a stratum of grey silty clay. Localized and shallow surficial sand deposits were encountered. The silty clay appears to be a lacustrine deposit to a depth of from 14 to 17 feet below ground level. Beneath this level the soil contains much sand and gravel and is of glacial origin. These clay deposits are stiff immediately under the silt but become medium stiff and remain in this softer condition to a depth of some 20 feet. Below elevation 570 the soils are stiff to very stiff over a considerable depth. Assumed bedrock lies about 70 feet below the present ground surface.

Three types of foundation for the underpass structure are discussed in the body of the report. Support on deep spread footings, founded at elevation 568 ft., or on displacement piles, will result in long term differential settlements of approximately 3 inches between the abutments and the centre pier. The abutments and adjacent fill will settle fairly uniformly with these types of foundation and a differential settlement of only  $1\frac{3}{4}$  inches can be expected between them. Steel H piles driven to refusal at a depth of about 70 feet will essentially eliminate differential movements between the pier and abutments but will result in the greatest differential movement of the fill relative to the structure.

The deep excavations necessary to place spread footings on competent soil and to guard against instability of the abutments appears uneconomical. Should these excavations be attempted, no ground water difficulties should be anticipated.

Embankments with side slopes no steeper than  $2\frac{1}{2} : 1$  are recommended. The upper sandy clayey silt should be suitable for embankment fill provided that wet areas are avoided.

DHS/kb  
Oct. 2, 1959  
J419



*D. H. Shields*

Donald H. Shields (P. Eng.)

Field Work

A total of 7 borings were put down at this site; their locations are shown on drawings 1 and 2. Two boreholes were put down at each abutment site, one at the centre pier and one under each approach embankment. Continuous flight auger equipment was used to form the boreholes. The holes were 5 inches in diameter and were uncased to full depth.

Samples were recovered at various intervals as the holes progressed. Both disturbed and undisturbed samples were taken depending upon the soil encountered.

A standard 2 inch O.D. split spoon was used to recover disturbed soil samples. This sampler was driven into the soil using a hammer transmitting 350 ft.lbs. of energy. The number of hammer blows of this magnitude required to drive the sampler from 6 to 18 inches penetration into the undisturbed soil ahead of the boring was recorded. This numerical value is the penetration resistance of the soil at the sampling depth. Occasionally the soil was soft enough to permit pushing the split spoon into the soil. This fact was also recorded. On withdrawal the sampler was dismantled and the soil classified and retained in moisture-proof containers.

Relatively undisturbed samples of the soil ahead of the boring were taken with thin-walled Shelby tubes. The 2-inch I.D. size was used almost exclusively. Three 3-inch I.D. Shelby tube samples were taken above a depth of 22 feet in hole 1 to enable a study of the consolidation characteristics of the soils to be made. Whenever possible the Shelby tubes were pushed into the soil. If this proved impossible, the tubes were driven in accordance with the procedure outlined for the split spoon. On withdrawal, the samples were sealed in the steel tubes and brought into the laboratory.

When the augers were withdrawn prior to each sampling operation, the soil retained in the flights was identified. In this way a continuous record of the subsoil types was made.

Careful note was also taken of the ground water conditions in each boring both during the advance of the hole and for a period of time after its completion.

All elevations were referenced to the centre line elevation of the county road apposite each boring. The centre line elevations were taken from the profile of DHO drawing BW-313. The elevation of the road was checked at several locations with the DHO bench mark located on a tree on the west side of the road about 200 feet north of hole 4.

In addition to sampling, field vane measurements were made of the shear strength of the cohesive soil in each boring. A  $2\frac{3}{4}$  inch diameter four-bladed vane was pushed into the undisturbed soil. The torque required to rotate the vane was recorded. When this value is related to the vane dimensions the shear strength of the soil can be computed. The soil was then completely remoulded by rotating the vane several times. The ratio of the torque required to rotate the vane in undisturbed and remoulded soil is recorded as the sensitivity of the soil to disturbance.



A log showing sampling interval, field vane measurements, soil types encountered and water level observations is presented for each boring. Drawings 4 to 10 are the logs for boring 1 to 4 and 9 to 11 respectively.

### Laboratory Testing

Measurement was made of the natural moisture content of each sample taken in the field. Atterberg limit determinations were carried out on selected representative samples. Natural unit weight of the soils was computed from the volume-weight measurements of the Shelby tube samples.

An undrained triaxial test was performed on each 2 inch diameter Shelby tube sample. A cylindrical specimen of soil was surrounded by a confining pressure equal at least to the total pressure existing in the soil at the depth from which the sample was taken. The sample was then failed at a constant rate of strain in axial compression. No drainage of the sample was allowed at any time. The shear strength of the soil was considered to be  $\frac{1}{2}$  of its compression strength.

An unconfined compression test was carried out on each 3 inch diameter Shelby tube sample. The shear strength of these specimens was taken equal to  $\frac{1}{2}$  the maximum axial compression load.

The results of these laboratory determinations are presented in table 1. The field vane measurements are also recorded here.

Actual stress-strain curves recorded during the triaxial and unconfined tests are presented in drawings 11 and 12.

All of the laboratory and field measurements are recorded on the borehole logs.

Consolidation tests were made on each of the 3 inch Shelby tube samples. The results of these tests are presented as drawing 13. It is noted that the specimens were trimmed to 2.5 inches in diameter by .75 inches thick. Patching of the sample from hole 1, 21 feet, was required because of the numerous small grits and gravel particles present.

APPENDIX 2SETTLEMENT COMPUTATIONSI Settlement of Abutments

Assume abutments founded at depth of 20 feet.

Recommended net safe bearing value = 4000 psf, which must include effect of negative friction from embankment fill and from consolidating clay above 20 feet (see computations on page 3).

(a) Computations of Pressures

Assumed abutment footing 6 feet wide and 40 feet long; total capacity = 480 tons.

Assume spread of load below footing at 30 degrees to vertical.

Assume water table at 10 feet or elevation 582, unit weight of natural soil and embankment fill above 10 feet = 125 pcf, unit weight below water table = 70 pcf.

Assume embankment 20 feet high, with 40 feet top width and  $1\frac{3}{4} : 1$  slopes (Note: Final safe slopes found to be  $2\frac{1}{2} : 1$ ; this will produce slightly higher soil pressures at greater depths below surface; see computation for 40 feet; effect on settlement computation negligible). Computation of fill pressures using Newmark diagram.

Depth Ft.	Initial Overburden Pressure $P_0$ pcf (1)	Pressure from Adjacent Embankment psf				Pressure from Footing psf (3)*	Final Pressure Under Abutment (1)+(2)+(3)
		Top Centre	Top Slope	Average (2)	Bottom Slope		
10	1250	1535	1240	1420	90		
15	1600			1240	160		
20	1950	1300	1000	1130	230	4000	7080
25	2300			1060	280	1780	5140
30	2650	1070	950	1000	320	1060	4710
35	3000			990	365	716	4706
40	3350	1025	930	980	380	520	4850
45	3700			975	395	397	5072
50	4050	1010	920	970	410	324	5344
55	4400			970	420	261	5630

Embankment pressure  
increment at 40 ft. 1095 1040 1070 300  
for  $2\frac{1}{2} : 1$  slopes.

\* Neglecting soil weight excavated for footing.

APPENDIX 2 cont'd.(b) Settlement Computation

Assume compressibility of soil from 20 to 50 feet defined by corrected consolidation test result for hole 1, 21 feet. Assume coefficient of volume compressibility,  $M_v$ , to conform to pressure and compression conditions at depth of 30 feet.

$$\text{Here } \Delta p = 2.06 \text{ ksf} \quad e_o = 0.505 \quad \text{and } \Delta e = .009$$

$$\text{Therefore } M_v = \frac{.009}{2.06 (1.505)} = 0.0029 \text{ sq.ft./kip.}$$

$$\text{Settlement 20 to 30 feet } (\Delta p @ 25 \text{ feet} = 2.84 \text{ ksf})$$

$$= 10 \times 12 \times 2.84 \times .0029 = 0.99$$

$$\text{Settlement 30 to 40 feet } (\Delta p @ 35 \text{ feet} = 1.7 \text{ ksf})$$

$$= 10 \times 12 \times 1.7 \times .0029 = 0.59$$

$$\text{Settlement 40 to 50 feet } (\Delta p @ 45 \text{ feet} = 1.37 \text{ ksf})$$

$$= 10 \times 12 \times 1.37 \times .0029 = 0.47$$

$$= 2.05"$$

Assume soil from 50 to 60 feet "normally loaded". Since no consolidation test done compute compression index from liquid limit.

$$C_c = 0.09 (45\% - 10) = 0.315$$

$$P_o \text{ at 55 feet} = 4400; \text{ final pressure } P_1 = 5630$$

$$\text{Settlement } S = H \frac{C_c}{1 + e_o} \log_{10} \frac{P_1}{P_o}$$

$$= 10 \times 12 \times \frac{0.315}{1.7} \log_{10} \frac{5630}{4400} = 2.42"$$

$$\text{Therefore total estimated settlement of abutments} = \underline{\underline{4.5 \text{ ins.}}}$$

II Settlement of Embankment Fill Immediately Adjacent to Abutment

Settlement below 20 feet will be essentially the same as computed for the abutments. However additional compression will occur in clay above 20 feet. For depth increment 0 - 5 feet settlement of silty sand will be negligible. For depth increment 5 - 10 feet use corrected consolidation curve hole 1,  $7\frac{1}{2}$  ft.

$$S = H \Delta p M_v = 5 \times 12 \times 1.42 \times .00242 = 0.206 \text{ ins.}$$

For depth increment 10 - 20 feet use corrected consolidation curve hole 1,  $11\frac{1}{2}$  ft.

$$S = 10 \times 12 \times 1.23 \times .0107 = 1.6 \text{ ins.}$$

Therefore estimated total settlement of embankment immediately adjacent to abutment

$$= 4.5 + 0.2 + 1.6 = \underline{\underline{6.3 \text{ ins.}}}$$

APPENDIX 2 cont'd.III Settlement of Embankment Fill 50 Feet Back from Abutments

Fill remains approximately 20 feet in height. Fill pressures almost doubled under central parts of embankment.

Depth Ft.	Initial Overburden $P_0$ psf	Approx. Average Pressure from Embankment $\Delta P$ psf	Final Pressure under Fill $P_1$ psf
8	1000	2500	3500
15	1600	2350	3950
25	2300	2100	4400
30	2650	2000	4650
35	3000	1950	4950
45	3700	1900	5600
55	4400	1900	6300

Settlement: Depth

0 - 5 ft. = nil

5 - 10 ft.  $S = H \frac{\Delta e}{1 + e_0} = 5 \times 12 \times \frac{.015}{1.74} = 0.517$  ins.  
(corrected curve hole 1,  $7\frac{1}{2}$  ft.)

10 - 20 ft.  $S = H \frac{\Delta e}{1 + e_0} = 10 \times 12 \times \frac{.06}{1.81} = 4.0$  ins.  
(corrected curve hole 1,  $11\frac{1}{2}$  ft.)

20 - 50 ft.  $M_v$  30 ft. =  $\frac{(0.504 - .496)}{2.0 \times 1.505} = 0.00266$  sq.ft./kip.

$S$  (20 - 30 ft.) =  $10 \times 12 \times 2.1 \times 0.00266 = .67$

$S$  (30 - 40 ft.) =  $10 \times 12 \times 1.95 \times 0.00266 = .62$

$S$  (40 - 50 ft.) =  $10 \times 12 \times 1.9 \times 0.00266 = .61$   
1.9 ins.

50 - 60 ft.

$S = H \frac{C_c}{1 + e_0} \log_{10} \frac{P_1}{P_0} = 10 \times 12 \times \frac{.315}{1.7} \log_{10} \frac{6300}{4400} = 3.56$

Therefore total estimated settlement of embankments approx. 10 inches.

APPENDIX 2 cont'd.IV Settlement at Edge of Embankment Adjacent to Ends of Wing WallsDepth

0 - 20 ft. negligible

20 - 50 ft.

(Applying pressure values for 30 feet to test result hole 1, 21 ft.)

$$S = H \frac{\Delta e}{1 + e_0} = 30 \times 12 \times \frac{.001}{1.505} = 0.24 \text{ ins.}$$

50 - 60 ft.

$$S = H \frac{C_c}{1 + e_0} \log_{10} \frac{P_1}{P_0} = 10 \times 12 \times \frac{.315}{1.7} \log_{10} \frac{4820}{4400} = 0.89 \text{ ins.}$$

Total estimated settlement at outside end of wing wall approx. 1 inch.

V Settlement of Centre Pier

Assume footing 40 x 6 feet founded at depth of 20 feet.

Assume net footing pressure of 4000 psf (no negative friction applicable here.)

Value of  $M_v$  at depth of 30 feet (using consolidation curve hole 1, 20 ft.)

$$= \frac{\Delta e}{\Delta P (1 + e_0)} = \frac{1.504 - 1.501}{1.06 (1.505)} = 0.00188 \text{ sq.ft./kip.}$$

Settlement

$$20 - 30 \text{ ft.} = 1.78 \times .00188 \times 10 \times 12$$

$$30 - 40 \text{ ft.} = 0.716 \times .00188 \times 10 \times 12$$

$$40 - 50 \text{ ft.} = 0.397 \times .00188 \times 10 \times 12$$

$$= 0.648$$

Settlement

50 - 60 ft.

$$S = 10 \times 12 \times \frac{.315}{1.7} \log_{10} \frac{4661}{4400} = 0.555$$

Total estimated settlement of centre pier = 1  $\frac{1}{4}$  inches.

VI Computation of Settlement RateAbutments

The rate of consolidation will be determined by the rate of drainage of the soil below 20 feet. For computation purposes the coefficient of consolidation of this material has been taken = 0.07 sq.ft./day. Drainage in two directions, upward to more permeable soil above 10 feet and downward to bedrock, has been

APPENDIX 2 cont'd.

assumed. Bedrock is felt to be permeable because water rose quickly in hole 4 after the augers reached refusal at a depth of about 70 feet. The stabilized water level in the bedrock was lower than the estimated ground water level.

Fifty percent consolidation or about  $2\frac{1}{4}$  inches of settlement of the compressible soil between 20 and 60 feet should be complete in time

$$t = \frac{0.197 \times 30}{0.07 \times 365} = 7 \text{ years.}$$

Ninety percent consolidation or about 4 inches movement should be complete in

$$t = \frac{0.848 \times 30}{0.07 \times 365} = 30 \text{ years.}$$

Centre Pier

The centre pier should settle at the same percentage rate as the abutments but, of course, the movements will be much less.

Embankment Adjacent to Abutment

Adjacent to the abutments the embankments should settle an estimated 6.3 inches or about 1.8 inches more than the abutments. This additional movement results from the compression of clay above 20 feet. Taking a value for coefficient of consolidation  $C_v = 0.12$  sq.ft./day this upper soil should be virtually consolidated after a time

$$t = \frac{0.8 \times 10}{0.12 \times 365} = 1.8 \text{ years approx. } *$$

Summary of Comments Regarding Settlement

The foregoing estimates of settlement are summarized as follows:

Abutment footings (founded at 20 feet)	= $4\frac{1}{2}$ inches.
Adjacent embankment fill	= $6\frac{1}{4}$ inches.
Embankment fill well back from the abutments	= 10 inches.
Edge of embankment near end of wing walls	= 1 inch.
Centre pier	= $1\frac{1}{4}$ inch.

The embankment adjacent to the abutments should settle about  $1\frac{3}{4}$  inches in about 2 years and hence a slight bump will be experienced for traffic passing onto the bridge. After this time both the embankment and abutments should settle about the same amount. All settlement should be complete in about 30 years. Somewhat more settlement of the embankment will occur at locations farther back from the bridge.

\* Fundamentals of Soil Mechanics P. 235 - D Taylor.

APPENDIX 2 cont'd.

A differential settlement of the order of 3 inches will exist between the centre pier and the end abutments. This can be avoided only by supporting the abutments on end-bearing piles to bedrock. Reducing the permissible bearing pressure below 4000 psf will not have much effect because the compressing force of the embankment fill still remains. If the abutments are supported on friction piles driven to a depth of 40 feet some slight reduction in settlement will result.

A differential settlement of approximately 5 inches will occur along the length of the wing walls if they are supported on the natural ground. In view of the weakness of the soil above 20 feet and the fact that differential settlement will occur, it would appear desirable to form the wing walls as integral parts of the abutments.

SUMMARY OF LABORATORY AND FIELD TEST RESULTS

Hole No.	Sample No.	Depth Ft.	Description	Shear Strength		Natural Moisture % dry wt.	Atterberg Limits		Natural Unit Weight pcf
				Field Vane psf	Undrained Triaxial psf*		PL	LL	
1		7	Beyond vane capacity.	2100+					
	2	6-8	Grey silty clay with 5" med. sand seam noted from 6'11". Many 1/8" fine sand seams irregularly spaced.		1320 unconfined	24.6	17.2	33.4	120
		10		882					
	3	10-12	Grey silty clay with silt pockets and occasional gravel sizes.		370 unconfined	29.0	17.1	34.2	122
		13		588					
	4	13-14 1/2	Grey silty clay, much fine to coarse gravel from 13 1/2 ft.		1080	28.7	18.1	38.3	134
		16 1/4		1050					
	5	17-18 1/2	Grey silty clay with some fine to med. gravel sizes, little fine sand.		900	21.6			131
		19 1/4		1008					
	6	20-21 1/2	Grey silty clay, sandy with sand and gravel sizes.		1650	18.0	18.6	31.0	131
		22	Beyond vane capacity.	2100+					
	7	24-25 1/2	Grey clayey silt with some sand and fine subangular gravel.		1670	17.9			131
	8	28-29 1/2	Grey silty clay with many fine to coarse gravel pieces.		2500	17.5	16.3	30.8	134

Table 1



SUMMARY OF LABORATORY AND FIELD TEST RESULTS cont'd.

Hole No.	Sample No.	Depth Ft.	Description	Shear Strength		Natural Moisture % dry wt.	Atterberg Limits		Natural Unit Weight pcf
				Field Vane psf	Undrained Triaxial psf*		PL	LL	
2		9 1/3	Could not insert vane.	2100+					
		10 1/2		1090					
		13 1/3		847					
	4	13-14 1/2	Grey silty clay, alternating layers of silt and clay approx. 1/32 to 1/16 inch thick.		750	32.5			118
		15 1/2		673					
		16		630					
	5	18-19 1/2	Grey silty clay containing many fine to med. gravel sizes, some silt and fine sand.		too gravelly for test	17.8			
		19 1/4		1092					
		20 1/2		1010					
	6	22-23 3/4	Grey silty clay, some sand and fine to med. gravel, clay seams noted from 22 1/2 to 23 ft., hair-line sand seam at 23 1/2 ft.		2025	16.0			133
		25		2100+					
3		9	Beyond vane capacity.	2100+					
		10 1/2		1010					
		12		756					
	4	13-14 1/2	Grey silty clay with occasional sand partings and lenses.		435	49.5	21.4	51.9	110
		15		462					
		15 1/2		630					
		18	Beyond vane capacity.	2100+					

SUMMARY OF LABORATORY AND FIELD TEST RESULTS cont'd.

Hole No.	Sample No.	Depth Ft.	Description	Shear Strength		Natural Moisture % dry Wt.	Atterberg Limits		Natural Unit Weight pcf
				Field Vane psf	Undrained Triaxial psf*		PL	LL	
3	5	18-19½	Grey silty clay containing sand and gravel sizes, middle 6" of sample grey clay.		1530	18.6			130
		20½	Beyond vane capacity.	2100+					
		21	Beyond Vane capacity.	2100+					
	6	22-23¾	Grey clayey silt with some sand and fine gravel.		1630	17.7			134
		25	Beyond vane capacity.	2100+					
	7	26-29½	Grey silty clay containing many fine to coarse gravel sizes and little fine sand.		2200	18.0			132
	4		Could not insert vane.	2100+					
			7½		882				
			8		840				
		3	8-9½	Grey clayey silt and clay. Clay is in layers inclined at irregular intervals, slightly lensed with fine sand.		580	36.6		
4		10½		462					
		11		504					
	4	13-14½	Grey slightly sandysilty clay containing occasional coarse sand sizes particularly below 14 feet.		550	26.6			123
		14		966					
		15½	Beyond vane capacity.	2100+					

SUMMARY OF LABORATORY AND FIELD TEST RESULTS cont'd.

Hole No.	Sample No.	Depth Ft.	Description	Shear Strength		Natural Moisture % dry wt.	Atterberg Limits		Natural Unit Weight pcf
				Field Vane psf	Undrained Triaxial psf*		PL	LL	
4		17		1764					
		20	Beyond vane capacity.	2100+					
	6	22-23 $\frac{3}{4}$	Grey clayey silt with sand and fine to med. gravel.		2240	17.9			133
		25	Beyond vane capacity.	2100+					
7	55-56 $\frac{1}{2}$	55-55.2'	Hard grey clayey silt with small sand pockets, some fine gravel.		no test	15.9			136
		-55.7'	Dense grey fine sand.						
		-56.0'	Very stiff grey clayey silt with some sand and fine to medium gravel.						
9		15 $\frac{1}{2}$		1428					
		19		2100					
		21 $\frac{3}{4}$	Beyond vane capacity.	2100+					
	4	22-23 $\frac{3}{4}$	Grey silty clay with gravel sizes. Occasional sand pockets.		1430	18.8			132
10		25	Beyond vane capacity.	2100+					
		3 $\frac{1}{2}$	Could not insert vane.						
		7	Beyond vane capacity.	2100+					
		10		861					
		13		1008					
		16 $\frac{1}{2}$		924					
		19 $\frac{1}{2}$		672					

Table 1

SUMMARY OF LABORATORY AND FIELD TEST RESULTS cont'd.

Hole No.	Sample No.	Depth Ft.	Description	Shear Strength		Natural Moisture % dry wt.	Atterberg Limits		Natural Unit Weight pcf
				Field Vane psf	Undrained Triaxial psf*		PL	LL	
10		22 $\frac{1}{2}$		1057					
		25	Beyond vane capacity.	2100+					
11		3 $\frac{1}{2}$	Could not insert vane.						
		6	Could not insert vane.						
		9 $\frac{1}{2}$	Could not insert vane.						
		13	Beyond vane capacity.	2100+					
		16		1344					
		19 1/3	Beyond vane capacity.	2100+					

Legend

\* Tested at overburden pressure.

PL = Plastic Limit.

LL = Liquid Limit.

## WILLIAM A. TROW &amp; ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Highway 401 Underpass W.P. 297-59

LOCATION Blenheim Road SW of Chatham

HOLE LOCATION See dwg. No. 1

HOLE ELEVATION AND DATUM 590.3 C.L. of Road  
opposite hole = 591.7

BOREHOLE NO. 1

FIELD SUPERVISOR

DRILLER

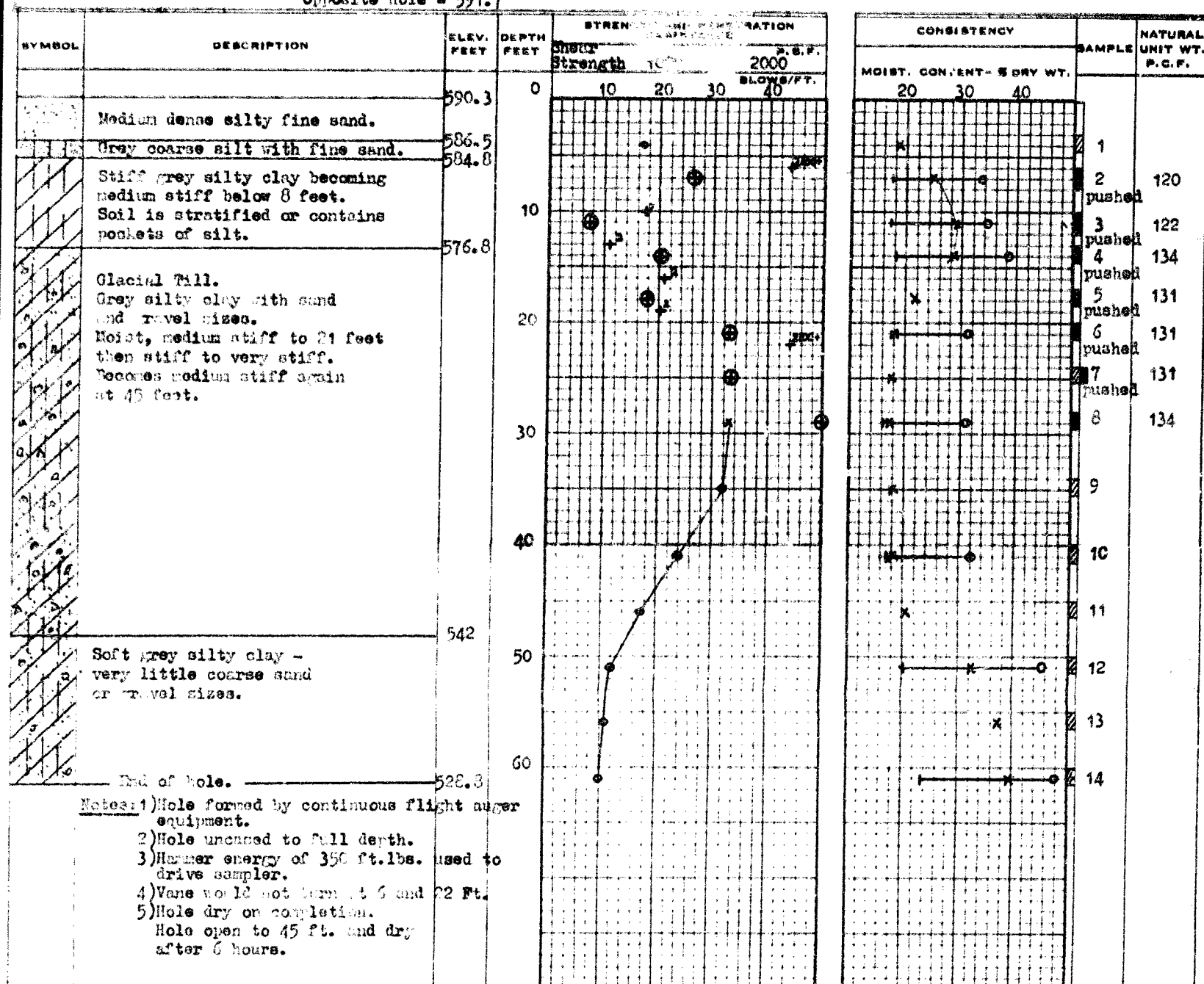
PREP.

DRAWING NO. 4

## LEGEND

1" 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

1" DIA. SPLIT TUBE  
 2" SHELBY TUBE  
 2" SPLIT TUBE  
 2" DIA. CONE  
 CASING  
 2" SHELBY



PROJECT NO. 3419

## WILLIAM A. TROW &amp; ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Highway 401 Underpass W.P. 207-50

LOCATION Bloemfield Road SW of Chatham

HOLE LOCATION See diag. No. 1

HOLE ELEVATION AND DATUM 589.4 C.L. of Road  
opposite hole No. 2 = 590.5

BOREHOLE NO. 2

FIELD SUPERVISOR

DRILLER

PREP.

DRAWING NO. 5

## LEGEND

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

CASING

2" SHELBY

1/2 UNCONFINED COMPRESSION (Qu)

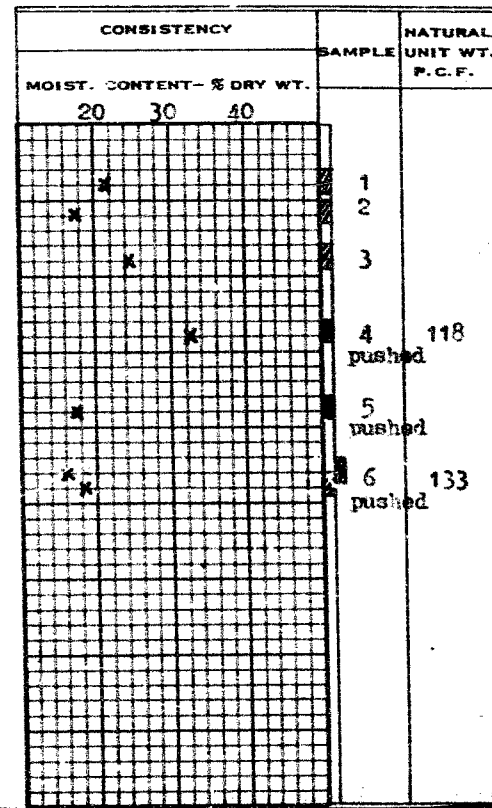
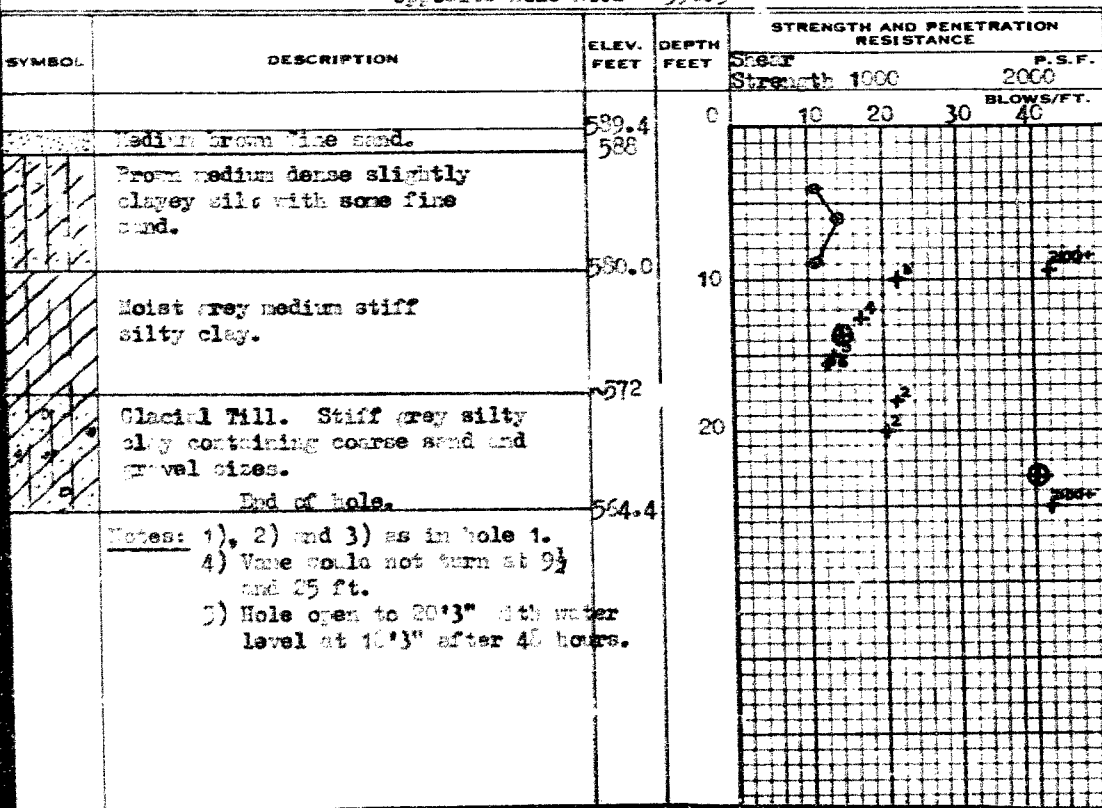
VANE TEST (C) AND SENSITIVITY (S)

NATURAL MOISTURE AND

LIQUIDITY INDEX

LIQUID LIMIT

PLASTIC LIMIT



Notes: 1), 2) and 3) as in hole 1.  
 4) Vane could not turn at 9' and 25 ft.  
 5) Hole open to 20'3" with water level at 10'3" after 48 hours.

**WILLIAM A. TROW & ASSOCIATES LTD.**

## SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Highway 401 Underpass W.P. 297-52

LOCATION: Glenfield Road SW of Chatham

**HOLE LOCATION** See dwg. No. 1

HOLE ELEVATION AND DATUM... 588.9 C.L. of Road  
opposite hole - 590.5

BONSHOLE NO. 3

**FIELD SUPERVISION.**

**GRILLER** grill /grill/

**PREP.** \_\_\_\_\_

**DRAWING NO.**

## LEGEND

2" DIA. SPLIT TUBE

# SHIRLEY TUNE

## 2.11 SPLIT TUBE

2<sup>o</sup> DIA. CONT.

## CABIN

21 **SHERRY**

1/2 UNCONFINED COMPRESSION (QU)

VANE TEST (C) AND SENSITIVITY (S).

## NATURAL MOISTURE AND

## LIQUIDITY INDEX

### LIQUID LIMIT

### PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				Strength	P.S.F. BLOWS/FT.
		508.9	0	10	20
	Medium brown silty sand.	505.9		30	40
	Very dry dense brown silt containing pockets of fine sand. 6" seam of clay noted at 8 ft.	580	10		
	Medium stiff grey silty clay - few thin sand lenses noted.	571	20		
	Glacial Till. Stiff to very stiff grey silt- clay containing sand to coarse gravel sizes. Medium stiff zone noted at 40 ft. Thin sand partings noted from 40 to 45 ft. Material is medium stiff below 55 feet.		30		
	End of hole.	527.4	60		

Notes: 1), 2) & 3) as in hole 1.  
4) Vane would not turn at 10, 20 and 25 feet.  
5) Hole open to 59½ ft. and dry after 48 hours.

CONSISTENCY			SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.				
20	30	40		
*			1	
*			2	
	*		3	
		*	4	110
			pushed	
	*		5	130
			pushed	
*			6	134
			pushed	
*			7	132
			pushed	
*			8	
	*		9	
*			10	
*			11	
	*		12	
		*	13	

## WILLIAM A. TROW &amp; ASSOCIATES LTD.

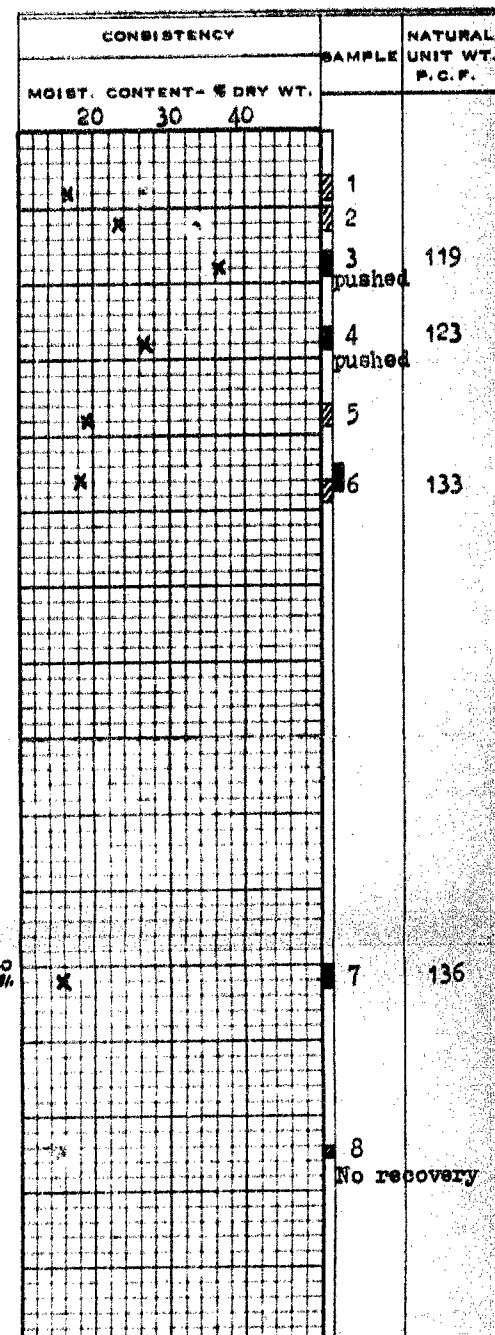
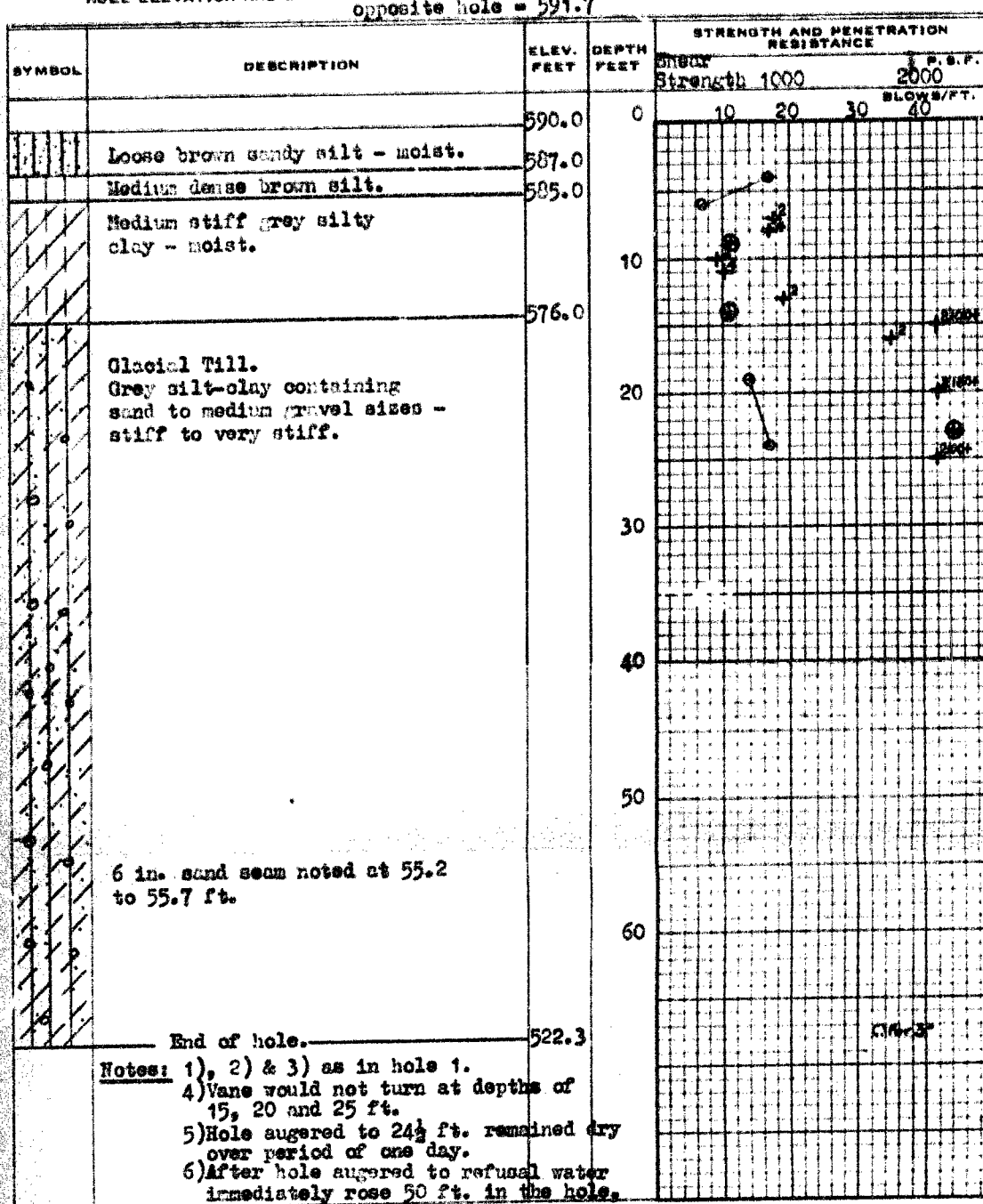
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Highway 401 Underpass W.P. 297-59  
 LOCATION Bloomfield Road SW of Chatham  
 HOLE LOCATION See Avg. No. 1  
 HOLE ELEVATION AND DATUM 590.0 C.L. of Road  
 opposite hole = 591.7

BOREHOLE NO. 4  
 FIELD SUPERVISOR  
 DRILLER  
 PREP.

## LEGEND

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



Notes: 1), 2) & 3) as in hole 1.  
 4) Vane would not turn at depths of 15, 20 and 25 ft.  
 5) Hole augered to 24 1/2 ft. remained dry over period of one day.  
 6) After hole augered to refusal water immediately rose 50 ft. in the hole, stabilized at elevation 570.3 after 1 day. Natural gas emitted continuously.



PROJECT NO. J419

## WILLIAM A. TROW &amp; ASSOCIATES LTD.

## SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

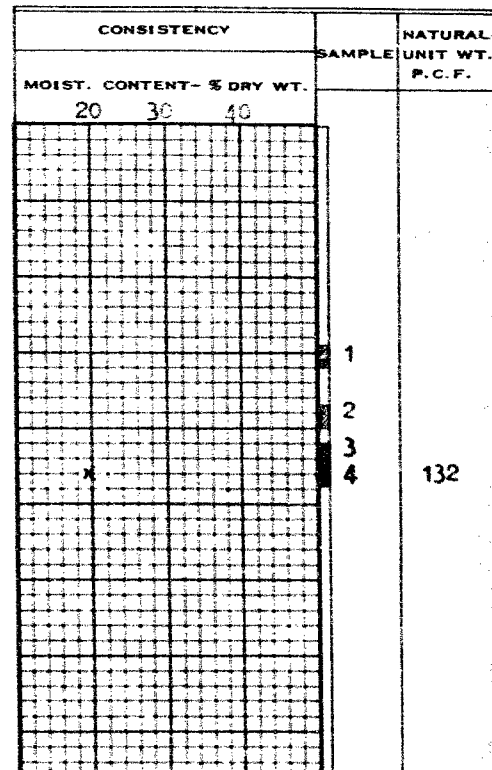
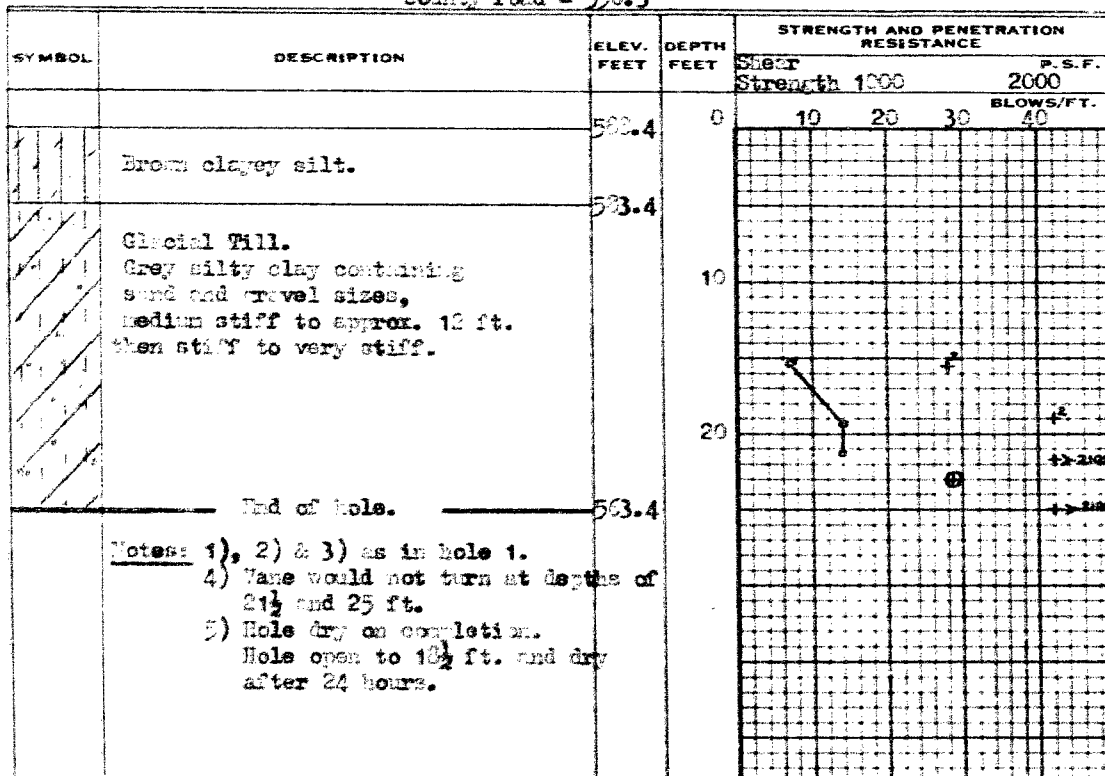
PROJECT Highway 401 Underpass W.P. 297-59  
 LOCATION Bloerfield Road SW of Chatham  
 HOLE LOCATION See dwp. No. 1  
 HOLE ELEVATION AND DATUM 588.4 C.L. of adjacent  
 county road = 590.3

BOREHOLE NO. 3  
 FIELD SUPERVISOR  
 DRILLER  
 PREP.

DRAWING NO.

## LEGEND

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



PROJECT NO. J419

## WILLIAM A. TROW &amp; ASSOCIATES LTD.

## SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

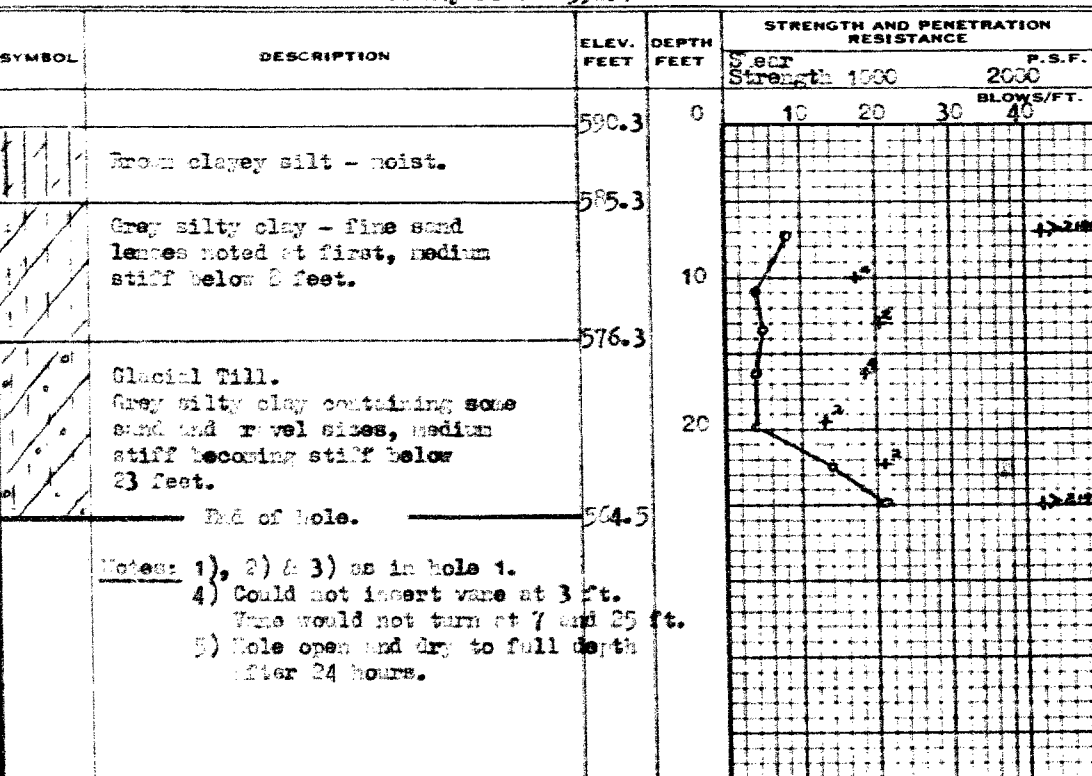
PROJECT Highway 401 Underpass W.P. 297-59  
 LOCATION Bloomfield Road SW of Cathlamet  
 HOLE LOCATION See diag. No. 2  
 HOLE ELEVATION AND DATUM 590.3 C.L. of adjacent  
 county road = 592.1

BOREHOLE NO. 10  
 FIELD SUPERVISOR  
 CRILLER  
 PREP.

DRAWING NO. 9

## 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 (Si)  
 NATURAL MOISTURE AND  
 LIQUIDITY INDEX  
 LIQUID LIMIT  
 PLASTIC LIMIT



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.		
	1	
	2	
	3	
	4	
	5	
	6	
	7	

PROJECT NO. J419

**WILLIAM A. TROW & ASSOCIATES LTD.**

## SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

**PROJECT** Highway 401 Underpass W.P. 297.59

**LOCATION** Bloomfield Road SW of Chatham

**HOLE LOCATION** See dwg. No. 2

MOLE ELEVATION AND DATUM.. 589.7 C.L. of adjacent  
country road = 598.3

BOREHOLE NO. 11

**FIELD SUPERVISOR**

**OWELL**

**PREP.** \_\_\_\_\_

DRAWING NO. 10

### LEGEND

2" DIA. SPLIT TUBE

2" SHELBY TUBE

## 2" SPLIT TUBE

2<sup>nd</sup> DIA. CONE

## CASING

2" SHELBY

1/2 UNCONFINED COMPRESSION ( $Q_u$ )

### VANE TEST [C] AND SENSITIVITY [S]

### NATURAL MOISTURE AND

### LIQUIDITY INDEX

**LIQUID LIMIT**

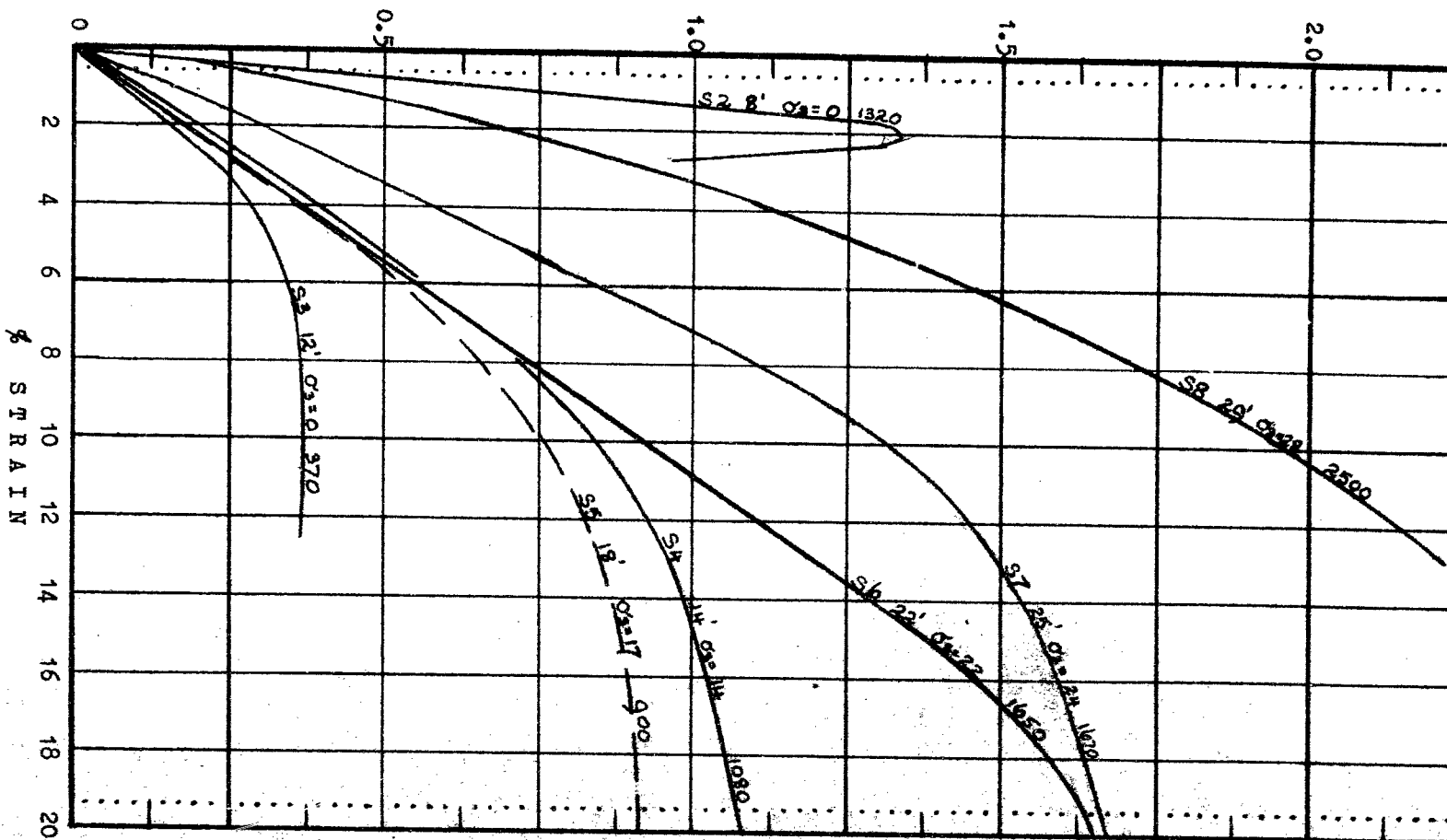
### PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				Shear Strength	1000	2000	P.S.F. BLOWS/FT.
	Clayey silt - brown dry.	589.7	0	10	20	30	40
		579.7	10				
	Silty clay - grey, contains sand and gravel sizes.	570.4	20				
	End of hole.						

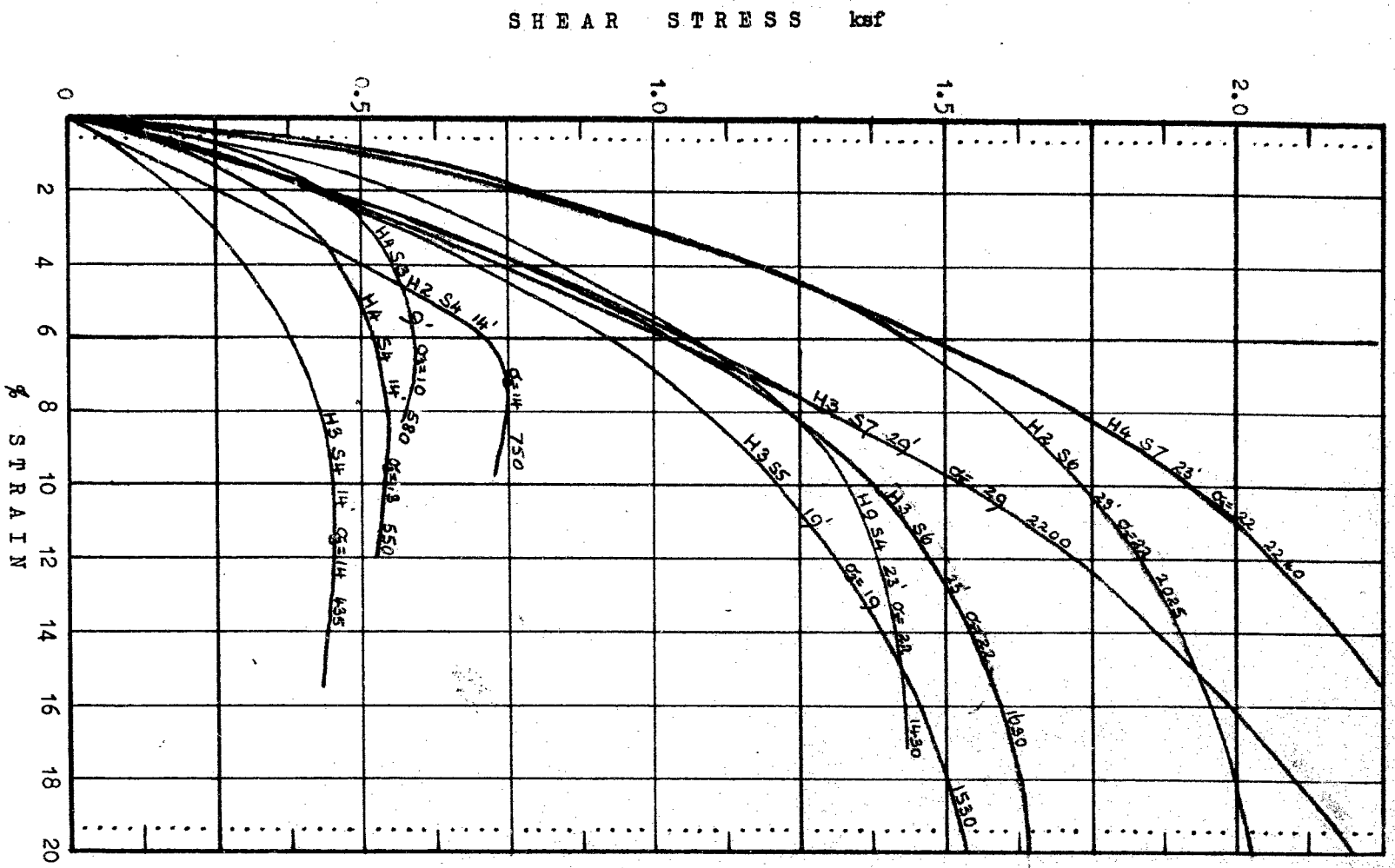
Notes: 1), 2) & 3) as in hole 1.  
 4) Could not insert vane at 3, 6 and 9 ft.  
 Vane would not turn at 13 and 19 ft.  
 5) Hole open to 17 1/2 and dry after 4 hours.

CONSISTENCY		SAMPLE	NATURAL
MOIST. CONTENT- % DRY WT.			UNIT WT. P.C.F.

# SHEAR STRESS ksf



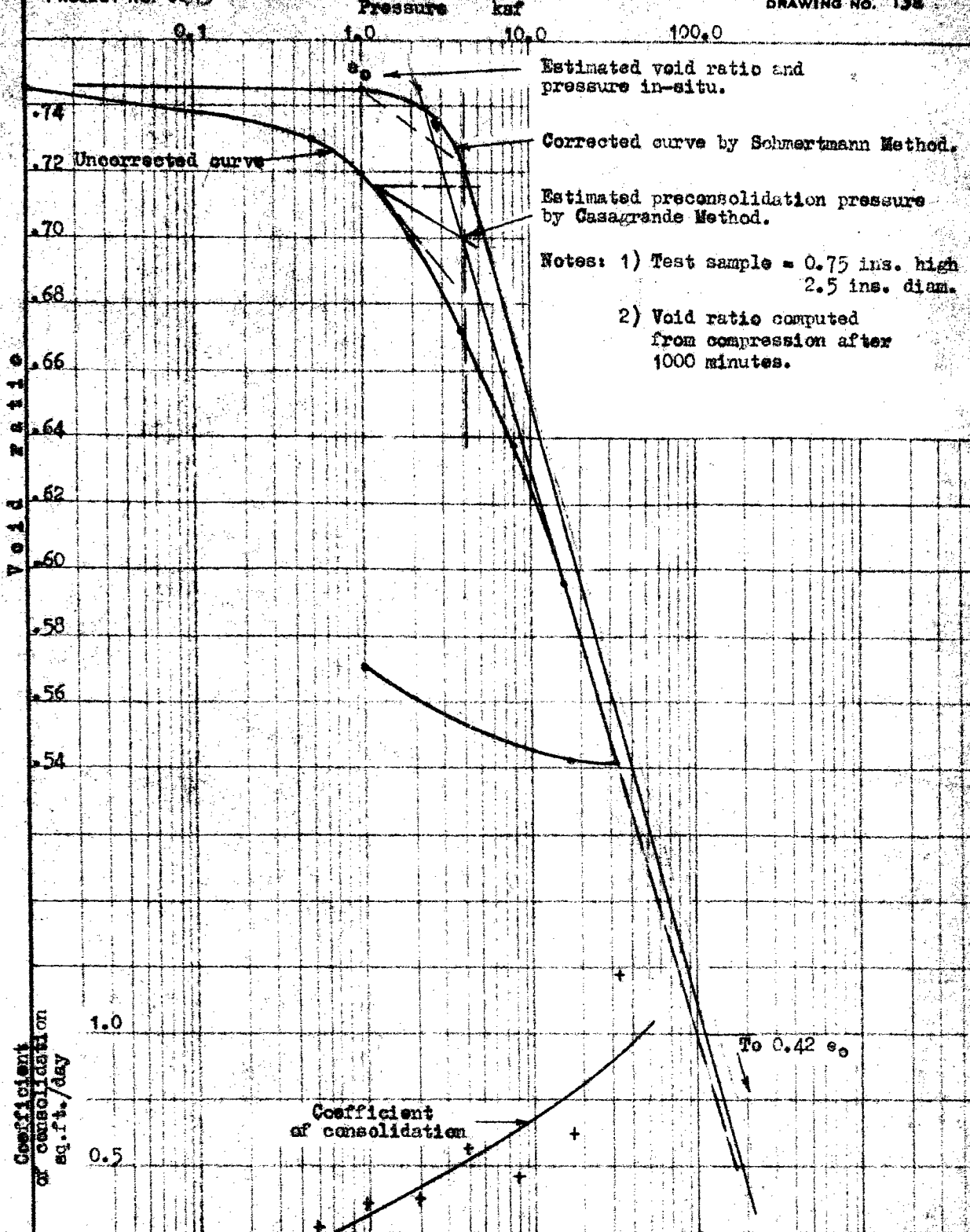
Stress-Strain Curves - Shear Tests on Samples from Hole 1



SHEAR STRESS ksf

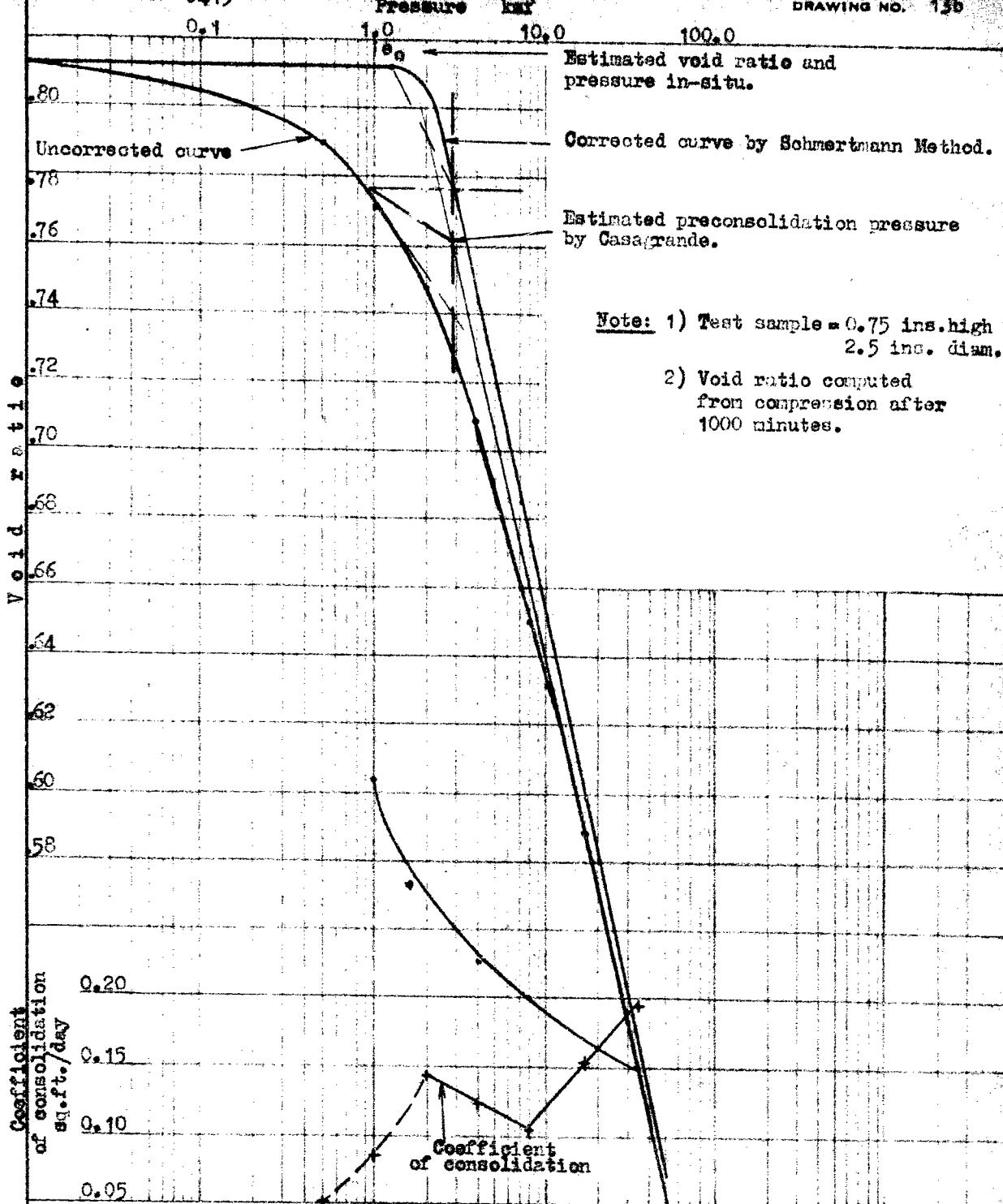
% STRAIN

Stress-Strain Curves - Shear Tests on Samples from Holes 2 - 3

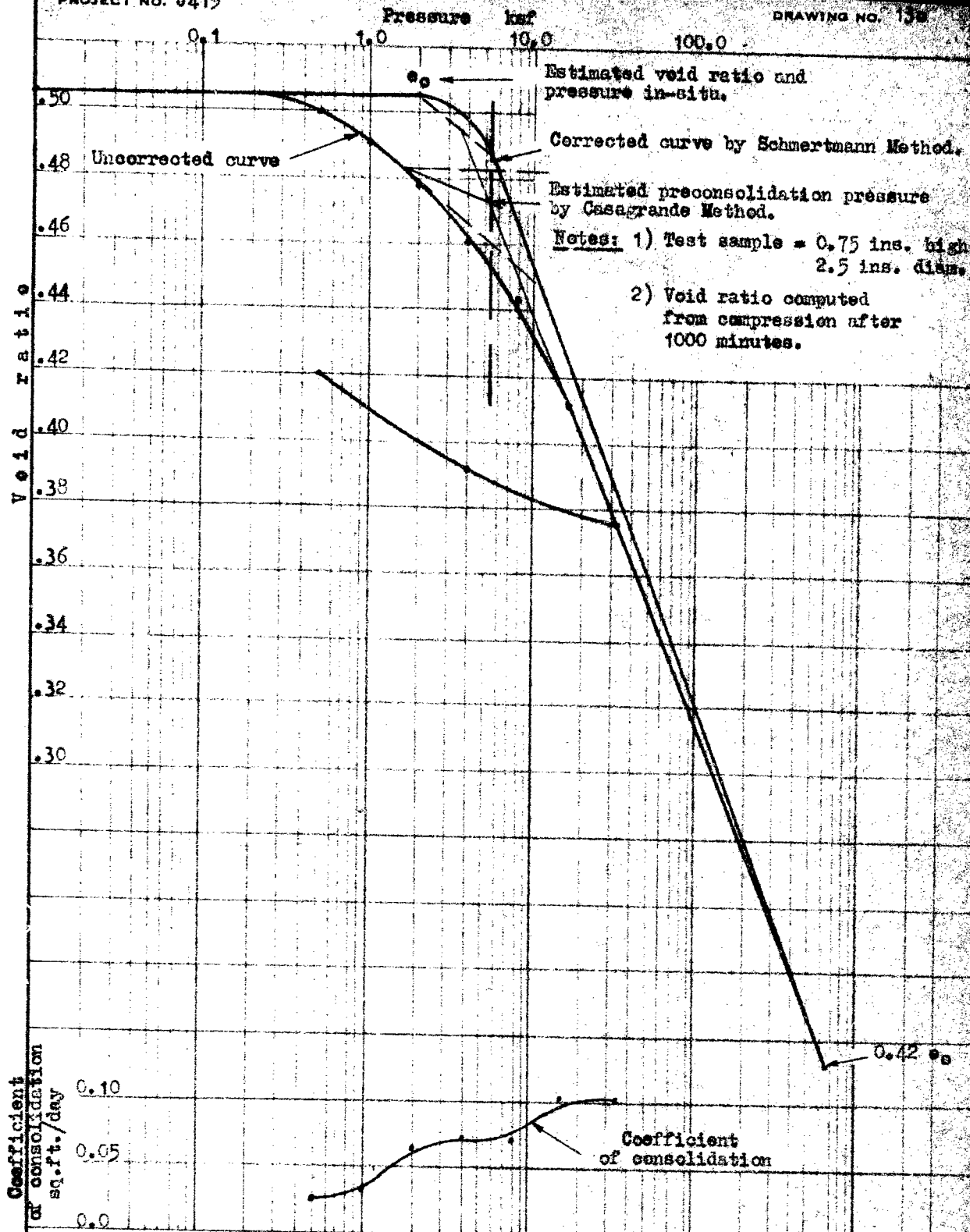


CONSOLIDATION TEST RESULT HOLE 1 - 7½ Ft. HIGHWAY 401 UNDERPASS  
W.P. 297-59 CHATHAM, ONTARIO.

WILLIAM A. TROW AND ASSOCIATES



CONSOLIDATION TEST RESULT HOLE 1 - 11½ Ft. HIGHWAY 401 UNDERPASS  
W.P. 297-59 CHATHAM, ONTARIO.



CONSOLIDATION TEST RESULT HOLE 1 - 21 Ft. HIGHWAY 401 UNDERPASS  
W.P. 297-59 CHATHAM, ONTARIO.





Underpass Site Looking North

Drill on Hole No. 3



Underpass Site Looking South

Drill on Hole No. 3



Underpass Site Looking North

Drill on Hole No. 3



Underpass Site Looking South

Drill on Hole No. 3

Department of Highways  
COPY  
For the Information of:

Mr. L. Francis,  
Bridge Design.

Bridge Division,  
July 12, 1960.

MEMORANDUM TO:

Mr. R. Fitzgibbon,  
Bridge Design Expediter.

RE: Malahigh Twp. Bridge #5,  
S.P. 897-52, Dist. #1, Hwy. 401

Please change the drawing number of our recently  
issued preliminary plan for this structure (issued  
July 8, 1960) from D 4444-73 to D 4719-71.

JLA/ea

c.c. P. Harvey  
A. Gray  
L. Francis  
C. Scott

  
J.L. Koon,  
Senior Engineer,  
Bridge Division.



ONTARIO

DEPARTMENT OF HIGHWAYS

Toronto 5,  
October 23, 1959.

MEMORANDUM TO:

Mr. J. Keen,  
Senior Engineer,  
Bridge Design Section.

RE: W.P. 297-59  
Raleigh TWP. Br. #5  
Hwy. #401 Dist. #1

Attached please find soil report BA 946A  
for the above structure.

A handwritten signature in cursive script, appearing to read "J. C. McAllister".

JCM:wd

J. C. McAllister,  
for S. McCombie,  
Bridge Planning Engineer.

ONTARIO  
DEPARTMENT OF HIGHWAYS

Memo to Mr. A. M. Toye, Date September 29, 1959.  
Bridge Engineer. Subject \_\_\_\_\_  
From Materials & Research Section. *Released by E*

Attention: Mr. S. McComble.

Re: Interim Report by W. A. Trow & Associates,  
on Foundation Conditions, Hwy. 401 Underpass,  
Southwest of Chatham, Ontario. W.P. 297-59.

In response to a request from your Mr. J. McAllister, I have asked the Consultants to submit their foundation recommendations for the above site, as quickly as possible. They have replied by submitting to our office, a letter which I have enclosed, generally outlining the soil conditions at the underpass location. It would appear from the preliminary information contained in this letter that the structure, supported on bearing piles driven to a depth of 70 feet below existing ground surface, is the obvious means of obtaining pier and abutment support.

Immediately the Consultants submit their formal report, we will review same and pass it on to you, with our comments.

LGS/MdeF  
Attach.

*L. G. Soderman*  
L. G. Soderman,  
PRINCIPAL SOILS & FOUNDATIONS ENGINEER.

# WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS  
AND  
SOIL MECHANICS CONSULTATION

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

884 WILSON AVE.  
DOWNSVIEW, ONT.  
ST. 8-8991

1419

Mr. A. Rytkin,  
Acting Materials and Research Engineer,  
Dept. of Highways of Ontario,  
Downsview, Ontario.

September 28, 1959

Att'n: Mr. L.G. Soderman,  
Principal Soils and Foundation Engineer.

Re: Interim Report on Foundation Conditions,  
Highway 401 Underpass, Southwest of  
Chatham, Ontario. W.P. 297-59.

Dear Sirs:

This letter outlines in a general way the soil conditions existing at this site and the problems associated with forming foundations and approach embankments for the proposed underpass structure. We hope to have our full report on this subject completed within the next 3 or 4 days.

A shallow stratum of brown clayey silt overlies gray silty clay at this site. The stratum of silt is of the order of 5 feet thick north of the proposed highway and approximately 9 feet thick to the south. This brown silt is very susceptible to changes in climatic conditions, becoming very wet and soft after rains and very dense after a period of drying.

The gray silty clay underlying the brown silt can be considered to exist in two distinct layers at most locations. The upper zone contains very little or no sand and gravel sizes and was found to be stratified or varved. The lower zone is a glacial till deposit consisting of a heterogeneous mixture of silty clay to clayey silt and numerous sand and gravel sizes. The upper horizon of this till begins generally at depths of from 14 to 16 feet.

The consistency of the upper gray silty clay soil ranges from stiff to medium stiff with strengths as low as 500 pcf at 10 feet depth. With some variations this deposit remains medium stiff to 20 feet depth. From depths of from 20 to 25 feet the material has a shear strength of the order of 1600 pcf. Below 25 feet the soil is very stiff and remains at this consistency to a depth of 45 feet. Below 45 feet the soil becomes weaker but its consistency is variable. This latter material appears to be normally consolidated.

Refusal to augering was reached at a depth of about 70 feet. This is felt to be bedrock elevation since rock was encountered at the same level in an adjacent investigation by the Dept. of Highways and by well drillers working in this vicinity.

#### Foundation Considerations

- 1) Although approach embankments will reach a maximum height of 20 feet, no embankment stability problems can be foreseen. The earth fill should remain stable both during and after construction.
- 2) Abutments and piers for the underpass structure can be supported either on spread footings or piling.
- 3) Spread footings must be placed a minimum of 20 feet below the ground surface at which level the safe bearing value of the soil is 4000 pcf. These footings will be subjected to a downward pull resulting from the adhesion of the medium stiff silty clay and from the friction of the overlying embankment fill as the softer clay above 20 feet consolidates under the weight of the approach fill. This negative skin friction in the clay can be reduced by placing a bentonite slurry between the original ground and the side of the foundation wall adjacent to the embankment. As the embankment settles negative friction forces will be transmitted only from the granular fill. The magnitude of this force will be of a relatively small order.
- 4) It is estimated that the abutments will settle a total of about 4 inches while the centre pier will settle approximately 1 inch.
- 5) Large displacement piles could be used to support the bridge structure. However, allowance will have to be made for the negative friction forces induced by settlement of the embankments. Piles, 1 foot in diameter, driven to a depth of 40 feet should have a working capacity of the order of 30 tons. Settlements will be of the same magnitude as with spread footings.
- 6) Steel H piles, 70 feet long, would be driven to refusal in the underlying rock. Their capacity would depend on the sectional area of the pile. Settlements would be almost non-existent. Some differential settlement should be anticipated if the abutments are supported on H piles and the intermediate pier is carried on a single footing.

We trust that this brief review of soil and foundation conditions will enable you to form an appreciation of the soils conditions at this site.

Yours very truly,



Donald E. Shields, P. Eng.

*Encl. 10/1/59*

Mr. A. M. Toye,

September 29, 1959.

Bridge Engineer.

Materials & Research Section.

Attention: Mr. S. McCombie.

Re: Interim Report by W. A. Trow & Associates,  
on Foundation Conditions, Hwy. 401 Underpass,  
Southwest of Chatham, Ontario. W.P. 297-59.

In response to a request from your Mr. J. McAllister, I have asked the Consultants to submit their foundation recommendations for the above site, as quickly as possible. They have replied by submitting to our office, a letter which I have enclosed, generally outlining the soil conditions at the underpass location. It would appear from the preliminary information contained in this letter that the structure supported on bearing piles driven to a depth of 70 feet below existing ground surface, is the obvious means of obtaining pier and abutment support.

Immediately the Consultants submit their formal report, we will review same and pass it on to you, with our comments.

LGE/MdeF  
Attach.

  
- L. G. Soderman,  
PRINCIPAL SOILS & FOUNDATIONS ENGINEER.



# WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS  
AND  
SOIL MECHANICS CONSULTATION

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

884 WILSON AVE.  
DOWNSVIEW, ONT.  
ST. 8-8921

3419

September 28, 1959

Mr. A. Rythm,  
Acting Materials and Research Engineer,  
Dept. of Highways of Ontario,  
Downsview, Ontario.

Attn: Mr. L.D. Soderman,  
Principal Soils and Foundation Engineer.

Re: Interim Report on Foundation Conditions,  
Highway 401 Underpass, Southwest of  
Chatham, Ontario. W.P. 297-55.

Dear Sirs:

This letter outlines in a general way the soil conditions existing at this site and the problems associated with former foundations and approach embankments for the proposed underpass structure. We hope to have our full report on this subject completed within the next 3 or 4 days.

A shallow stratum of brown clayey silt overlies gray silty clay at this site. The stratum of silt is of the order of 5 feet thick north of the proposed highway and approximately 9 feet thick to the south. This brown silt is very susceptible to changes in climatic conditions, becoming very wet and soft after rains and very dense after a period of drying.

The gray silty clay underlying the brown silt can be considered to exist in two distinct layers at most locations. The upper zone contains very little or no sand and gravel sizes and was found to be stratified or varved. The lower zone is a glacial till deposit consisting of a heterogeneous mixture of silty clay to clayey silt and numerous sand and gravel sizes. The upper horizon of this till begins generally at depths of from 14 to 18 feet.

The consistency of the upper gray silty clay soil ranges from stiff to medium stiff with strengths as low as 500 pcf at 10 feet depth. With some variations this deposit remains medium stiff to 20 feet depth. From depths of from 20 to 25 feet the material has a shear strength of the order of 1600 pcf. Below 25 feet the soil is very stiff and remains at this consistency to a depth of 45 feet. Below 45 feet the soil becomes weaker but its consistency is variable. This latter material appears to be normally consolidated.

Refusal to augering was reached at a depth of about 70 feet. This is felt to be bedrock elevation since rock was encountered at the same level in an adjacent investigation by the Dept. of Highways and by well drillers working in this vicinity.

#### Foundation Considerations

- 1) Although approach embankments will reach a maximum height of 20 feet, no embankment stability problems can be foreseen. The earth fill should remain stable both during and after construction.
- 2) Abutments and piers for the underpass structure can be supported either on spread footings or piling.
- 3) Spread footings must be placed a minimum of 20 feet below the ground surface at which level the safe bearing value of the soil is 4000 psf. These footings will be subjected to a downward pull resulting from the adhesion of the medium stiff silty clay and from the friction of the overlying embankment fill as the softer clay above 20 feet consolidates under the weight of the approach fill. This negative skin friction in the clay can be reduced by placing a bentonite slurry between the original ground and the side of the foundation wall adjacent to the embankment. As the embankment settles negative friction forces will be transmitted only from the granular fill. The magnitude of this force will be of a relatively small order.
- 4) It is estimated that the abutments will settle a total of about 4 inches while the centre pier will settle approximately 1 inch.
- 5) Large displacement piles could be used to support the bridge structure. However, allowance will have to be made for the negative friction forces induced by settlement of the embankments. Piles, 1 foot in diameter, driven to a depth of 40 feet should have a working capacity of the order of 30 tons. Settlements will be of the same magnitude as with spread footings.
- 6) Steel H piles, 70 feet long, would be driven to refusal in the underlying rock. Their capacity would depend on the sectional area of the pile. Settlements would be almost non-existent. Some differential settlement should be anticipated if the abutments are supported on H piles and the intermediate pier is carried on a simple footing.

We trust that this brief resumé of soil and foundation conditions will enable you to form an appreciation of the soils conditions at this site.

Yours very truly,



Donald H. Shields, P. Eng.

DHS/kb

BA 946

Mr. A. M. Toye,

September 29, 1959.

Bridge Engineer.

Materials & Research Section.

Attention: Mr. S. McCombie.

Re: Interim Report by W. A. Trow & Associates,  
on Foundation Conditions, Hwy. 401 Underpass,  
Southwest of Chatham, Ontario. W.P. 297-59.

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LGS/MdeF  
Attach.

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

# WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS  
AND  
SOIL MECHANICS CONSULTATION

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

884 WILSON AVE.  
DOWNSVIEW, ONT.  
ST. B-5921

J419

September 28, 1959

Mr. A. Rytko,  
Acting Materials and Research Engineer,  
Dept. of Highways of Ontario.  
Downsview, Ontario.

Att'n: Mr. L.G. Soderman,  
Principal Soils and Foundation Engineer.

Re: Interim Report on Foundation Conditions,  
Highway 401 Underpass, Southwest of  
Chatham, Ontario. W.P. 297-59.

Dear Sirs:

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We trust that this brief resume of soil and foundation conditions will enable you to form an appreciation of the soils conditions at this site.

Yours very truly,



Donald H. Shields, P. Eng.

Department of Highways

COPY

For the Information of:

file

Bridge Division,  
July 11, 1960.

Mr. L. G. Soderman,  
Principal Soils & Foundation engineer,  
Laboratory Building.

RE: Raleigh Twp. Br. #5,  
W.P. 297-59, Hwy. 401,  
Dist. #1, Twp. Raleigh,  
Co. Kent, Lots 18 & 19, Con. VII

We enclose herewith one copy of our drawing D 4444-P3  
and would like to know the length of pile embedment re-  
quired to sustain a design load of 40 tons per pile.

LNF/dd

L. W. Francis,  
for J.L. Keen,  
Senior Engineer,  
Bridge Design Office.



DEPARTMENT OF HIGHWAYS

**Memo to** Mr. J. L. Keen,  
Senior Engineer,  
Bridge Division.  
**From** Materials & Research Section.

**Date** July 22, 1960.

**Subject** Raleigh Twp. Bridge #5,  
W.P. 297-59, District #1,  
Underpass Hwy. 401 (S.W. or Chatham.

In reply to your letter dated July 20th, 1960, we are advising you that it is our opinion that your second alternative - i.e., the front rip-rap slope at 2:1 and a transition of slope 2:1 to the side slope at  $2\frac{1}{2}$ :1 be retained as the final solution.

The end effect at the end of the embankment, justifies the reduction of the  $2\frac{1}{2}$ :1 slope to 2:1.

If there are any additional questions that you would like to discuss, please feel free to call on our Office.

L. G. Soderman,  
PRINCIPAL FOUNDATIONS ENGR.

Per:

*Astermac*

(A. Stermac,  
FOUNDATIONS OFFICE ENGR.)

AS/MdeF

cc: Foundations Office  
Gen. Files.

$$W_3 = \frac{1}{2} \times 19 \times 7 \times 39 = 3760$$

$$W_2 = 10.5 \times 19 \times 11.5 = 6290$$

$$W_1 = \frac{1}{2} \times 38 \times 19 \times 13.5 = 4270$$

$$14,620 \times 135 = 1,975,000$$

$$\frac{442}{120} \times \left( \frac{82.5 \times 554}{115 \times 100} \right) = 1530000$$

$$1530000$$

$$461000$$

$$76200$$

$$2,070,200$$

$$\textcircled{1} \text{ F.S. } \frac{2,070,200}{1,975,000} = 1.05 \text{ - safe}$$

$$\frac{14 \times 6}{2} \times 135 \times \cos 46^\circ + 62.5 \times \text{force } 30 \times 1.35$$

$$2,136,200$$

$$62.50$$

$$2,136,200$$

$$\textcircled{2} \text{ F.S. } = \frac{2,136,200}{1,975,000} = 1.08$$

