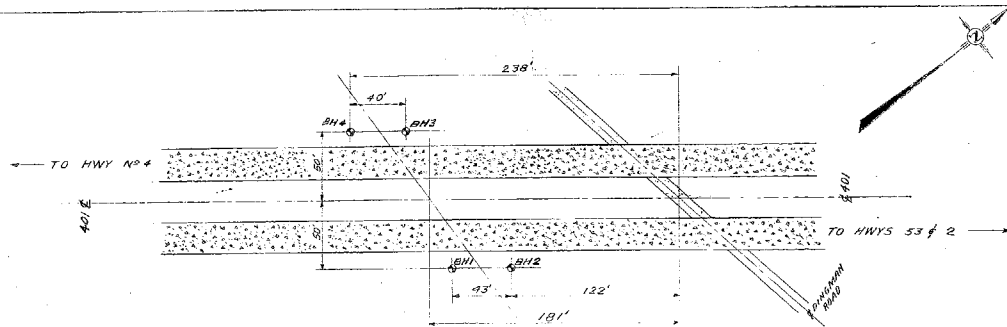
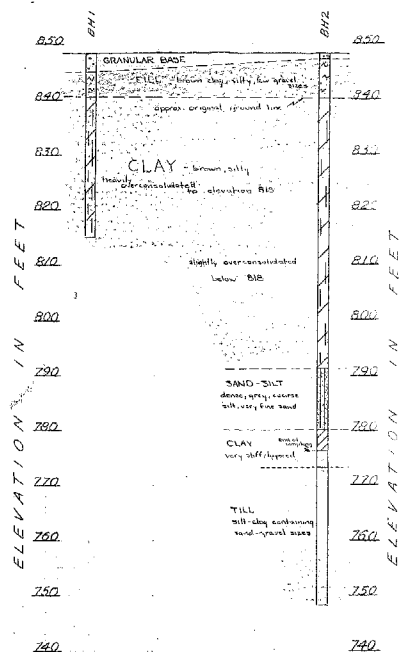


#59-F-237-C
W.P. 22-59
HWY. #401 UNDER-
PASS, DINGMAN
CREEK RD, S. OF
LONDON



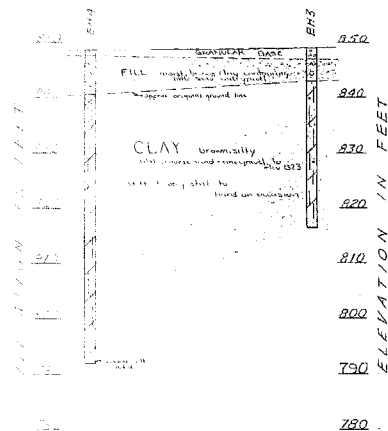
PLAN SHOWING LOCATIONS OF BOREHOLES

SCALE 1" = 10' H



PROFILE BETWEEN BH1 & BH2

HORIZONTAL SCALE 1" = 10' H
VERTICAL SCALE 1" = 10' V



PROFILE BETWEEN BH4 & BH3

HORIZONTAL SCALE 1" = 10' H
VERTICAL SCALE 1" = 10' V

PROPOSED UNDERPASS
KING'S HIGHWAY 401-DINGMAN CREEK ROAD
SOUTH-WEST OF LONDON
W.R. 22-59

William A. Trow & Associates Limited
JOB N° 422 OCT 5 1955

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac, Principal Foundation Engineer,
Room 107, Lab. Building

Mr. A.P. Watt,
Reg. Bridge Location Engineer,
London Regional Office

Bridge Division,
Downsview, Ontario

June 16, 1967

Dundas Creek Road Underpass
5 Miles East of Hwy. No. 4
W.P. 22-59-1, Site No. 19-368
Highway 401 (M.C.F.). District 2

Attached herewith are prints of the Preliminary Bridge
Plan Drawing D-6286-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is \$110,000.
This cost includes tender, materials, engineering and sundry
construction.

Any comments or revisions you may have should be submitted
within three weeks.

CSG:rd

C.S. Grebski,
Bridge Design Engineer

Attach.

c.c. S. McCombie
A. Stermac
E. Cross
R. Forrest

Materials and Research Division,

May 28, 1962.

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

Attention: Mr. W. A. Trow.

Re: W.P. 22-59-1, Hwy. #401, Dingman Creek Rd.
W.P. 98-57, Hwy. #401, Beestwich Rd.
District #2, London.

Dear Sir:-

Please consider this your authority to carry out one additional sampled borehole at each of the above sites, to provide more precise information concerning probable settlements.

Plans and profiles were provided to your representative on May 24, 1962. We are also attaching for your information, a copy of the original Foundation Report for the latter site. Would you please return this to our Office as soon as possible.

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

Fourteen copies of each of the completed foundation reports should be submitted to the Foundation Section as soon as possible.

Charges for the work performed will be in accordance with your Schedule of Rates, dated May 24, 1959, and the invoice should be addressed to the attention of Mr. A. Rutka, Materials and Research Engineer.

WDA/ade
Attach.

Yours very truly,

cc: Messrs. C. McCombie

A. Guter

W. L. Fraser

J. Roy

E. D. Smith (2)

Mrs. T. Tate

Foundations Office ✓

Gen. Files (2)

A. Rutka
A. Rutka,
MATERIALS AND RESEARCH ENGINEER

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. 5-5921Project: J422Mr. A. Rutka,
Department of Highways of Ontario,
Materials and Research Branch,
Parliament Buildings,
Toronto 5, Ontario.

October 8, 1959

59-F-237Attention: Mr. L.G. Soderman, P. Eng.,
Principal Soils and Foundation Engineer.Re: Foundation Investigation, Proposed Underpass Structure
Highway 401 at Dingman Creek Road, South of London, Ontario.WP 22-58-1
NST 2

Dear Sirs:

Enclosed herewith is our report on the soil conditions existing under the proposed Highway 401 underpass at Dingman Creek Road, near London, Ontario.

No foundation problem of consequence appears to exist at this location. Footings can be placed on the hard clay immediately underlying the present fill base of Highway 401 at a depth of about 10 feet. The recommended safe bearing value to apply at this depth is 6.00 psf. Some settlement of the abutments can be anticipated but this will result, in large part, from the weight of the adjoining embankment fill. A long term value of 3 inches has been computed. Farther back from the abutments the fill may eventually settle a total of about 7 inches. No embankment stability problem is anticipated.

We hope that the information contained in this report assists you in the selection of the required foundations for this structure.

Yours very truly,

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 DINGMAN CREEK ROAD
SOUTH OF LONDON, ONTARIO

Project: J422

Oct. 8, 1959

William A. Trow & Associates Ltd.

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FOUNDATION INVESTIGATION
PROPOSED UNDERPASS STRUCTURE
HIGHWAY 401 AT DINGMAN CREEK ROAD
SOUTH OF LONDON, ONTARIO

This report describes the soils investigation carried out at the above site. The safe bearing value of the soil at this location has been outlined. The stability of the embankment approaches to the underpass has been considered.

Description of Site

The intersection of Highway 401 and Dingman Creek Road lies in an area of level to very undulating ground. The diverted Dingman Creek passes immediately to the west of the underpass site and drains this region. Ground level is generally at elevation 841. The surface of Highway 401 is some 7 feet above this normal ground elevation.

It is understood that a rigid frame concrete structure is proposed for this site. Abutments will be located on either side of Highway 401 and there will be no centre pier. Maximum approach embankment height will be of the order of 30 feet.

Soil Types Encountered

All borings were located on the shoulders of Highway 401. This meant that from 6 to 8 feet of granular base and clay fill was penetrated before the original ground level was reached. The surface of the original ground was denoted by the presence of grasses and organic brown clay.

Immediately beneath the sod and about 1 foot of organic soil, hard brown clay was encountered. This clay deposit extends to approximate elevation 792. The soil above about elevation 818 has apparently been desiccated by weathering and drying resulting in the overconsolidation of this material. This overconsolidation is greatest in the first 7 feet of soil below original ground level where it is quite hard. Below this depth down to elevation 818 the clay can be classed as very stiff and at deeper levels it can be classified as stiff to medium stiff. The consistency or shear strength of the clay is best indicated by the strength-depth profile of drawing 8 on which the strength measurements from all four borings have been combined.

A deposit of coarse silt to fine sand underlies the brown clay. This stratum is some 10 feet thick and extends approximately from elevation 791 to elevation 780. A very stiff varved or layered silty clay was encountered at about elevation 780 in hole 2. The exact extent of this deeper clay deposit is not known since sampling was not carried out much beyond its upper horizon. Hole 2 was augered to a depth of 100 feet below the surface of the highway but no samples were attempted below 72 feet depth. A definite change in resistance to augering was noted at 75 feet or about elevation 773. Augering was extremely difficult below this depth. The soil retained on the flights of the augers was a very stiff silty clay till containing coarse sand and gravel sizes. The depth to rock was not proven in this investigation.

Water level observations were taken for a considerable period of time after the borings were completed. However, the results are rather inconclusive since the water levels differed in each boring. In holes 2, 3 and 4 the water stabilized near the surface of the original ground while in hole 1, the level was about 10 feet lower down. Rain water lay in a shallow pool in the field to the south at the time of the boring program and it is thought that this water was seeping into the uncased holes along the topsoil between the fill and the impermeable clay of the original ground.

Foundation Considerations

a) Abutment Footings. The hard clay crust found immediately below the original ground surface is a very competent foundation medium for direct support of the proposed bridge structure. Spread footings should be placed as high up in this hard zone as possible and preferably at approximate elevation 838 feet. The soil cover required for frost protection will be provided by the approach embankments and the 7 feet of fill on which highway 401 rests.

The safe bearing capacity of spread footings placed on the hard clay at elevation 838 is 8000 psf. No ground water problems can be foreseen that would hinder excavation to this level.

b) Settlement Computations. An attempt has been made to compute the probable settlements that the abutments and the main portion of the embankments will undergo. Any computations for settlement involve an analysis of the change in effective stresses that the underlying soil will experience. Settlement only becomes appreciable when the preconsolidation load of the soil under consideration is exceeded. Judging from the strength and moisture content profiles of the foundation soils at this site, the clay is highly overconsolidated near the surface. The degree of overconsolidation decreases gradually with depth until, by elevation 818, the soil is either normally consolidated or only very slightly overconsolidated. From this observation it appears reasonable that any settlements, to be expected above about elevation 818, will represent a recompression of the clay and hence will be of a small order. Below approximate elevation 818 the degree of overconsolidation is not so apparent and therefore more settlement of this zone should be expected with an increase in effective stress. However, if it is reasoned that overconsolidation of the soil above about elevation 818 is due to desiccation or capillary forces set up subsequent to a lowering of the ground water, then the lowest possible water table must have been at elevation 818 approximately. This means that the soil below elevation 818 has been subjected to the pressure of at least the unbuoyed weight of the overlying soil. This observation assumes that the ground level has not been higher in the past. Therefore calculations for settlement, as outlined in appendix 2 are based on this assumption of overconsolidation below elevation 818 feet.

Newmark charts were used to compute the stresses transmitted to the soil at the end of construction. The water table existing at this site at the time of the investigation appeared to be at the original ground level. If this is

so then the application of the embankment and bridge footing loads will produce an increase in pressure below elevation 818 feet which will only exceed the assumed overconsolidation by a very small amount. On this basis the settlements to be expected at the abutments should be of the order of $1\frac{1}{2}$ inches only.

Since it is more probable that the normal water table of the area is at the same elevation as the nearby creek invert, effective stresses should, more realistically be determined on this basis. A settlement of the order of 3 inches has been computed for this condition.

Should the water table drop to elevation 818 as it must have at some time in the past, the most critical condition for settlement will exist. Settlements of the order of 6 inches would result for this circumstance.

Assuming a water table at elevation 835 - the approximate invert level of Dingman Creek - the embankments can be expected to settle 7 to 8 inches at a distance of about 50 feet back from the abutments.

The settlements outlined above will take place over a long period of time. The soil most affected by consolidation will be the material below elevation 818. This layer of clay will have to drain to the silt sand below elevation 791. This is a relatively long drainage path. Therefore the time required for 100% consolidation will be considerable. In the absence of consolidation tests on this soil, the rate of compression can be estimated only very approximately. About 30% of the total settlement should occur during construction while the remainder will be in the form of a gradual movement over a number of years.

c) Stability of Approach Embankments. Approach embankments of the order of 30 feet high will be required at this site. These embankments will rest on the soils outlined by this investigation. Large shearing stresses will be transmitted into these natural soils by the embankment loading. However, the shear strength of these materials is high especially near the surface. As a consequence, it appears that instability could only develop as deep-seated movements through the medium stiff clay below elevation 818. One trial circular arc analysis was carried out considering failure to take place through this relatively less stiff material. The arc chosen is shown on drawing 8. The factor of safety against failure, neglecting any resistance offered in the fill, was found to be 3.0.

With a factor of safety of this order no further analysis appeared to be warranted. The stability of the approaches therefore seems to be adequate.

d) The Abutments as Retaining Structures. A number of assumptions had to be made in order to obtain an estimate of the retaining capacity of the footing supported abutments. Granular soil with an angle of friction of 30° was assumed for the embankment fill. The active pressure from this fill will amount to the weight of soil times the active pressure coefficient, 0.33.

However, only the weight of soil was considered to act in the development of passive resistance to movement of the abutments towards the highway. This is thought to be conservative and can be considered to be the resistance available in the road shoulder when the lateral strain against it is negligible. The full passive resistance of this compressible fill will be utilized only after some movement of the abutment wall has taken place against it.

Accordingly

Active Pressure from 32 feet of fill (elevation 838 - 870)

$$P_a = \frac{1}{2} \times 32^2 \times 125 \times 0.33 = 21,100 \text{ lbs/lineal foot}$$

Passive Pressure from 10 feet of fill beneath Highway 401 (elev. 838 - 848)

$$P_p = \frac{1}{2} \times 10^2 \times 125 = 6,200 \text{ lbs/lineal foot}$$

Therefore the unbalanced force $P_r = 14,900 \text{ lbs/lineal foot.}$

From the long term point of view the clay under the base of the abutment can be considered to resist this unbalanced force by reverting to the properties of a cohesionless material. Assuming an angle of friction of 30° for this case, the load per lineal foot of abutment will have to equal $\frac{14,900}{\tan 30^\circ}$ approx. 26,000

lbs/lineal foot or horizontal movement may result. With a span of nearly 100 feet between supports, it is thought that loads of the order required will be available to resist failure of the abutments. In addition, the rigid frame characteristics of the proposed structure will also offer resistance to this embankment pressure.

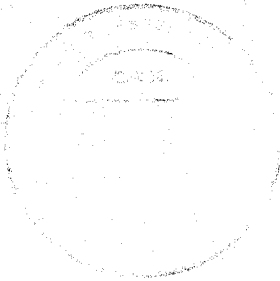
Conclusions

The foregoing comments can be summarized briefly as follows:

- 1) Heavily overconsolidated clay extends from the natural ground surface to a depth of some 20 feet at this site. Shear strengths in excess of 5000 psf were recorded down to approximate elevation 832. The shear strength generally decreased with depth, the soil reaching a medium stiff consistency below about elevation 818. This clay deposit extends to elevation 791 below which lies a 10 foot layer of dense silt-sand. Stratified stiff clay was encountered beneath the silt-sand. This is thought to be succeeded by a silt-clay glacial till deposit at approximately elevation 773.
- 2) It is recommended that the overpass structure be supported by spread footings placed near the surface of the hard clay. The safe bearing value for footings placed at approximate elevation 838 is 8000 psf.
- 3) Only the approximate order of settlement of the bridge and embankments can be determined because of the difficulty of ascertaining the existing ground water level and the variations in degree of overconsolidation. Most probable values of settlement of the abutments and approach embankments are 3 inches and 7 inches respectively.

- 4) The stability of the approach embankments is not a problem at this site.
- 5) The stability of the abutments acting as retaining structures for the embankments has been considered. The computations involved are outlined in the report and should be referred to after the bridge loading, foundation size etc. have been determined.

DHS/kb
Oct. 8, 1959
J422



D.H. Shields
Donald H. Shields, P. Eng.

Field Work

A total of four borings were put down at this site, two under each abutment location. Drawing 1 shows the locations of the four borings. Continuous flight auger equipment was used to form the boreholes. The holes were 5 inches in diameter and 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. 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 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 condition in each boring both during the advance of the hole and for a period of time after its completion.

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 1 to 5 are the logs for boring 1 to 4 respectively.

APPENDIX 1 cont'd.

All elevations are referenced to the centreline elevation of the south lane of Highway 401 at Station 376. The pavement surface has an elevation of 848.8 at this location. This information was supplied by the District Engineer's office of the Dept. of Highways in London.

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 measurement of the Shelby tube samples.

An undrained triaxial test was performed on each representative 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.

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 6 and 7.

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

APPENDIX 2SETTLEMENT COMPUTATIONSAssumptions

- 1) Soil between elevations 818 and 791 subject to consolidation.
- 2) Compression index of this zone based on liquid limit of approx. 30 percent, $C_c = 0.2$.
- 3) Abutment footing 4 feet wide carrying 8000 psf.
- 4) Load spread from surface loading in accordance with Newmark diagram.
- 5) Water level at elevation 818 at some time in past; underlying soil consolidated under wet weight of soil above this elevation plus Highway 401 embankment weight.

Settlement of Abutments with the Following Water Table Conditions

a) Water level at ground surface

- computations for 9 foot thick layer with effective depth of 35 feet are shown.

Maximum overburden pressure before construction

$$\begin{array}{rcl}
 23' \times 135 \text{ pcf} + 12' \times 72 \text{ pcf} & = & 3970 \\
 \text{load from 401 embankment} & = & \frac{580}{4550} \quad (\text{from Newmark diagram})
 \end{array}$$

Pressure acting after construction

$$\begin{array}{rcl}
 35 \times 72 & = & 2520 \\
 \text{load from Hwy 401 embankment} & = & 390 \\
 \text{" " approach} & = & 1460 \\
 \text{" " abutment} & = & \frac{280}{4650}
 \end{array}$$

$$\text{Settlement of 9 ft. layer} = \frac{C_c}{1+e_0} H \log_{10} \frac{P_1}{P_0}$$

$$\text{where } C_c = \text{compression index} = 0.2$$

$$e_0 = \text{initial void ratio} = 0.6 \text{ from water content}$$

$$H = 9 \text{ feet} = 108 \text{ inches}$$

APPENDIX 2 cont'd. P_0 = previous pressure P_1 = after construction pressure

$$S = \frac{0.2}{1 + 0.6} \times 108 \log_{10} \frac{4650}{4550} = 0.14 \text{ in.}$$

Similarly the total settlement of 27 foot soft zone from elevation 818 to elevation 791 feet = 0.5 inches.

Allow 1 inch for recompression of the heavily overconsolidated soils above elevation 818 feet.

Total Settlement of Abutments = 1.5 inches

b) Water table at creek invert, elevation 835

 P_0 = 4550 as before

$$P_1 = 6 \times 135 + 29 \times 72 = 2,900$$

load from Hwy 401 embankment = 390

" " approach " = 1,460

" " abutment = 280

5,030

$$S = \frac{0.2}{1 + 0.6} \times 108 \log_{10} \frac{5030}{4550} = 0.58 \text{ inches}$$

Total settlement of zone = 1.8 inches

Allow 1 inch for recompression
above elevation 818 feet = 1.0 inch

Total Settlement of Abutment = 2.8 inches.

c) Water table at elevation 818

 P_0 = 4550 as before

$$P_1 = 23 \times 135 + 12 \times 72 = 3970$$

load from Hwy 401 embankment = 390

" " approach " = 1460

" " abutment = 280

6100

APPENDIX 2 cont'd.

$$\begin{aligned}
 S &= \frac{0.2}{1 + 0.6} \times 108 \log_{10} \frac{6100}{4550} = 1.73 \text{ inches} \\
 \text{Total settlement of zone} &= 5.1 \text{ inches} \\
 \text{Recompression} &= 1.0 \text{ inch} \\
 \text{Total Settlement of Abutment} &= 6.1 \text{ inches.}
 \end{aligned}$$

Settlement of Embankment 50 feet back of Abutments

Water table at elevation 835 feet

$$\begin{aligned}
 P_0 &= 23 \times 135 + 12 \times 72 = 3970 \\
 P_1 &= 6 \times 135 + 29 \times 72 = 2900 \\
 \text{load from embankment} &= \frac{2720}{5620} \text{ (from Newmark diagram)}
 \end{aligned}$$

$$S = \frac{0.2}{1 + 0.6} \times 108 \log_{10} \frac{5620}{3970} = 2.0 \text{ inches}$$

$$\text{Total settlement of compressible zone} = 6.0 \text{ inches}$$

$$\text{Recompression of overconsolidated soils approximately } 1 - 2 \text{ inches}$$

$$\text{Maximum Embankment Settlement} = 7 - 8 \text{ inches.}$$

SUMMARY OF LABORATORY TEST RESULTS

Hole No.	Sample No.	Depth Ft.	Description	Undrained Shear Strength psf*	Natural Unit Weight pcf	Natural Moisture % dry wt.	Atterberg LL %	Limits PL %
2	3	7-8½	Top 5": Brown fine sandy silt and clay (fill). Lower 12": Brown organic med. sand slightly silty.					
	4	10-11½	Brown silty clay, slightly sandy with some gravel sizes.	5120	138	16.5	32.0	15.8
	5	13-14½	Brown-grey slightly silty clay, few sand particles and coarse gravel sizes throughout.	5080	137	16.9		
	6	16-17½	Brown silty clay with a trace of fine to medium sand.	3500	135	17.9		
	7	19-20½	Brown silty clay with a little fine to med. sand (5" recovered).			17.5	31.4	15.4
	8	22-23½	No Recovery.					
	9	25-26½	No Recovery.					
	11	32-33½	Grey fine sandy silty clay, some silt and fine sand seams.	1530	132	21.1		
	12	35-36½	Brownish-grey slightly silty clay. ¼' silt layer in centre of test specimen.	623	128	28.0	38.3	17.4
	13	38-39½	Grey silty clay with silt pockets - top 6" (tested). Grey clayey silt with silt pockets - next 6". Grey saturated silt - bottom 6".	1390	133	20.5	24.0	14.6
	14	41-42½	Brownish-grey clay containing silt seams - top 9". Plastic clay containing silt pockets - bottom 9".	1710	131	21.6		

SUMMARY OF LABORATORY TEST RESULTS cont'd.

Hole No.	Sample No.	Depth Ft.	Description	Undrained Shear Strength psf*	Natural Unit Weight pcf	Natural Moisture % dry wt.	Atterberg LL %	Limits PL %
2	15	44-45½	No Recovery.					
	16	47½-49	Brownish-grey slightly silty clay with occasional thin silt seams - top 10". Closely varved silt-clay - bottom 8".	1390	127	25.0	35.2	17.3
	17	51-52½	Brownish-grey clay with hairline seams of silt at irregular intervals.	2710	124	27.9		
	18	54-55½	Brownish-grey silty clay with silt pockets.	2000	134	19.2		
	19	57-58½	Grey silty very fine sand.					
	10	28½-30	Spoon sample.			20.7	28.1	14.9
	22	70-72	Spoon sample.				22.6	15.6
4	3	7-8½	7-8': Fill. Soft grey silty clay with sand and gravel. 8-8½': Original topsoil.	No test performed				
	4	10-11½	Very stiff to hard brown silty clay.	"	134	23.9		
	5	14-15½	Hard brown silty clay with some coarse sand to fine gravel sizes.	"	136	16.8		
	7	24-25½	Very stiff brown silty clay with few pebbles.	2800	134	19.5	35.9	16.1
	8	29-30½	Grey, slightly silty clay, few coarse sand and fine gravel sizes.	2100	136	17.8		
	9	35-36½	Medium stiff brown silty clay.	600	134	21.3		

SUMMARY OF LABORATORY TEST RESULTS cont'd.

Hole No.	Sample No.	Depth Ft.	Description	Undrained Shear Strength psf*	Natural Unit Weight pcf	Natural Moisture % dry wt.	Atterberg Limits LL %	Limits PL %
4	10	40-41½	Brown medium stiff clay - one silt pocket noted.		134	24.3		
	11	45-46½	Brown med.stiff varved silty clay. Varves at irregular intervals. Clay portion appeared fissured.			24.2		
	12	50-51½	Med. stiff brown clay. Vertical hairline sand seam noted. Lenses of fine sand at irregular widely spaced intervals.		123	28.1		
	13	55-56¾	55-56': Greyish-brown silty clay. 56' - Greyish-brown coarse silt.		138	21.5		

Legend

* Tested at overburden pressure.

LL = Liquid limit.

PL = Plastic limit.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Highway 401 - Dingman Road Underpass

LOCATION South of London

HOLE LOCATION See drawing 1

HOLE ELEVATION AND DATUM 848.4 C.L. of South
Lane Highway 401 Sta 376 = 848.8

BOREHOLE NO. 1

FIELD SUPERVISOR

DRILLER

PREP.

DRAWING NO.

2

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

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				Shear Strength 2000	P.S.F. 4000
				BLOWS/FT.	
		848.4	0		
	Granular Fill. Brown sandy gravel.				
	Fill and original ground surface.	844.9			
	Brown silty clay with organics.				
	Black topsoil.	840.4			
	Brown Clay - stiff to very stiff to hard on occasion.		10		
	Medium sand to medium gravel sizes noted to 22 ft. depth.		20		
			30		
	End of hole.	814.9			

Notes: 1) Continuous flight auger equipment
formed hole.
2) Hole 5 inches in diam. uncased to
full depth.
3) Hammer energy of 350 ft.lbs. used to
drive sampler.

4) Recorded water levels: Hole dry on completion, open to 30 ft.
after 26 hrs. - 30.2 ft. " " "
" 66 hrs. - 29.6 ft. " " "
" 90 hrs. - 28.7 ft. " " "
" 114 hrs. - 27.8 ft. " " " After 9 days = 24.2 ft. open to 29 ft.

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

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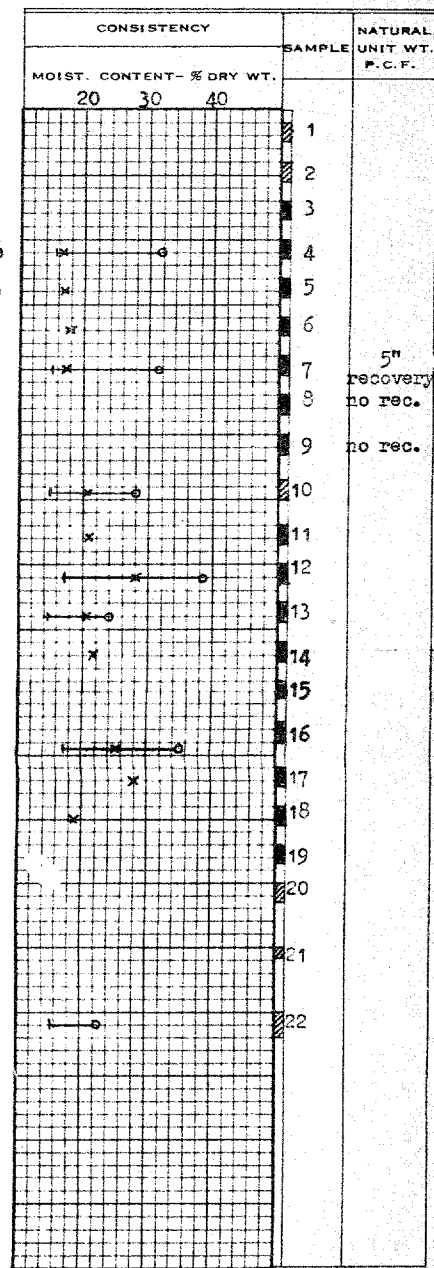
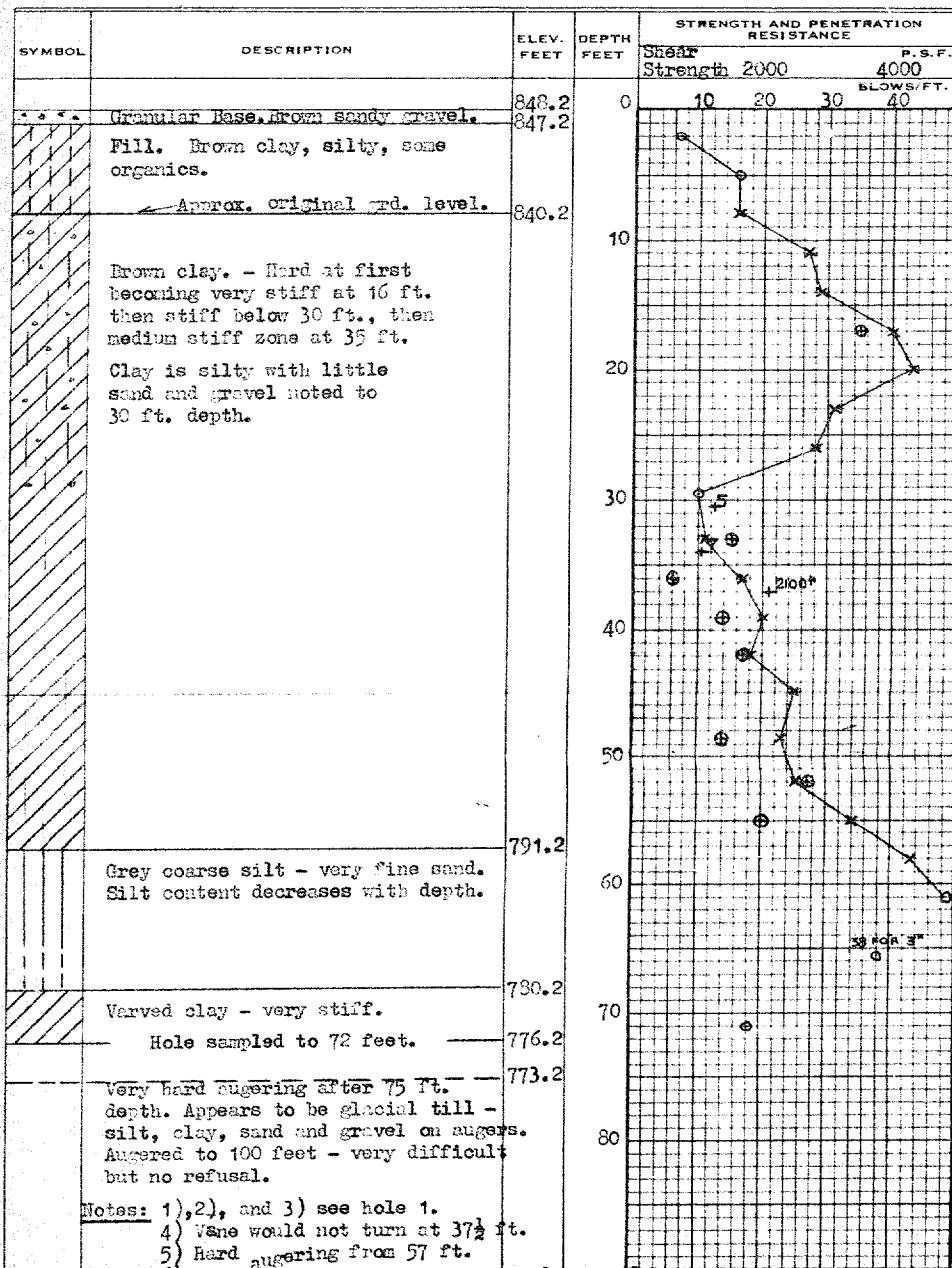
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Highway 401 - Dingman Road Underpass
 LOCATION South of London
 HOLE LOCATION See drawing 1
 HOLE ELEVATION AND DATUM 848.2 BM see hole 1

BOREHOLE NO. 2
 FIELD SUPERVISOR
 DRILLER
 PREP.

LEGEND

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



Notes: 1), 2), and 3) see hole 1.

4) Vane would not turn at 37½ ft.

5) Hard augering from 57 ft.

6) Water level soundings: After 18 hrs - 8.7 ft. hole open to 20 ft.

" 42 " - 8.5 " " " "

" 64 days - 8.2 " " " "

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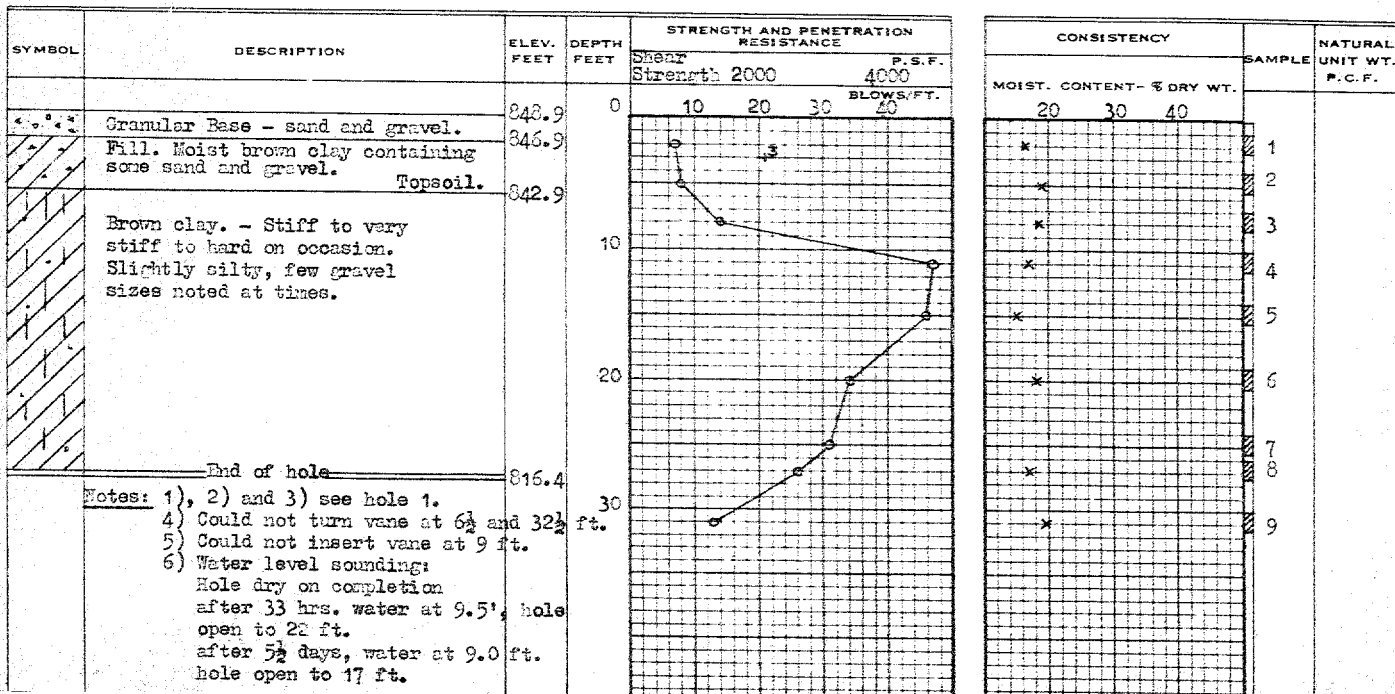
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

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 Highway 401 - Dingsen Road Underpass
 LOCATION South of London
 HOLE LOCATION See drawing 1
 HOLE ELEVATION AND DATUM 848.9 BM see hole 1

BOREHOLE NO. 3
 FIELD SUPERVISOR
 DRILLER
 PREP.



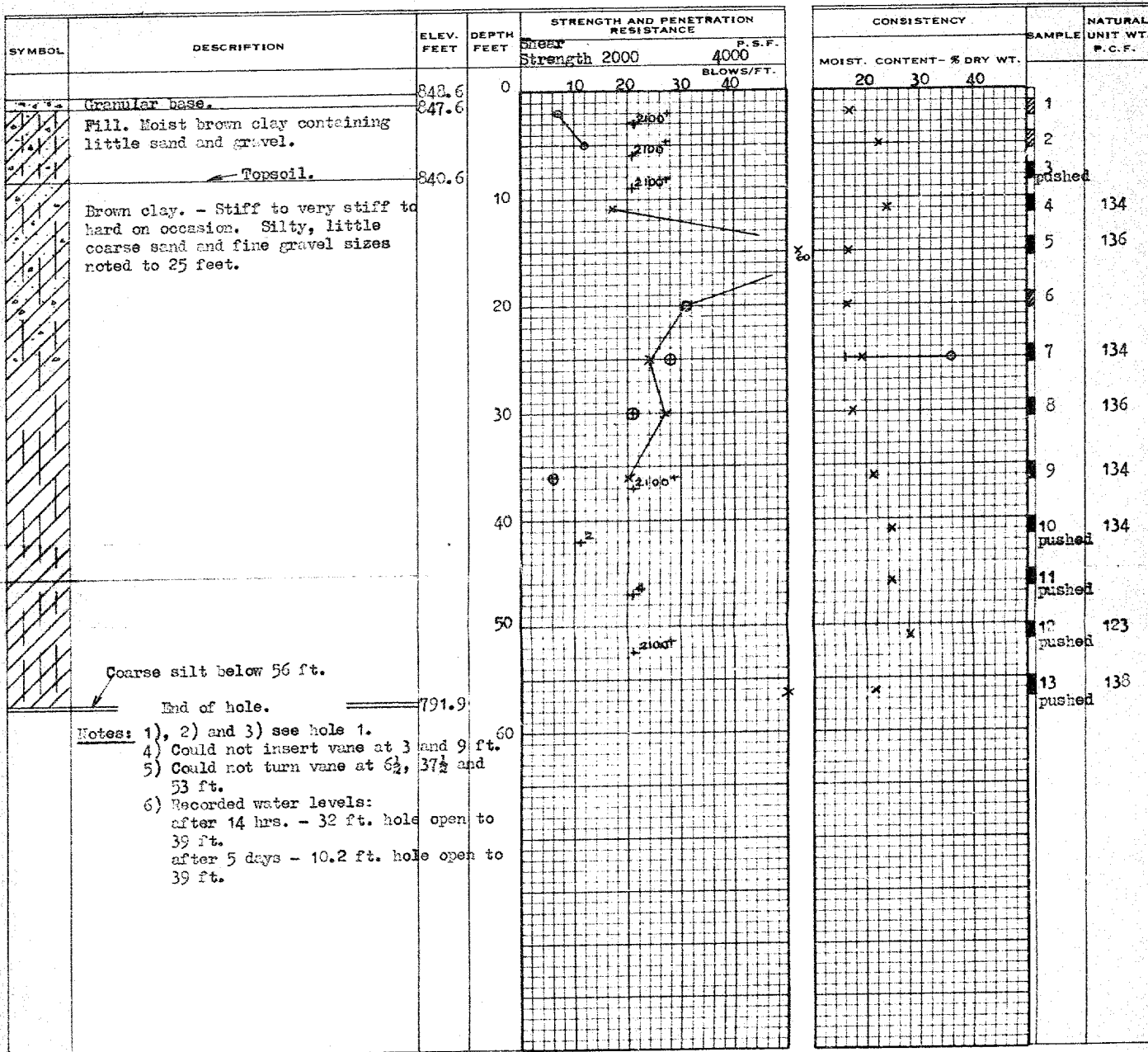
WILLIAM A. TROW & ASSOCIATES LTD.

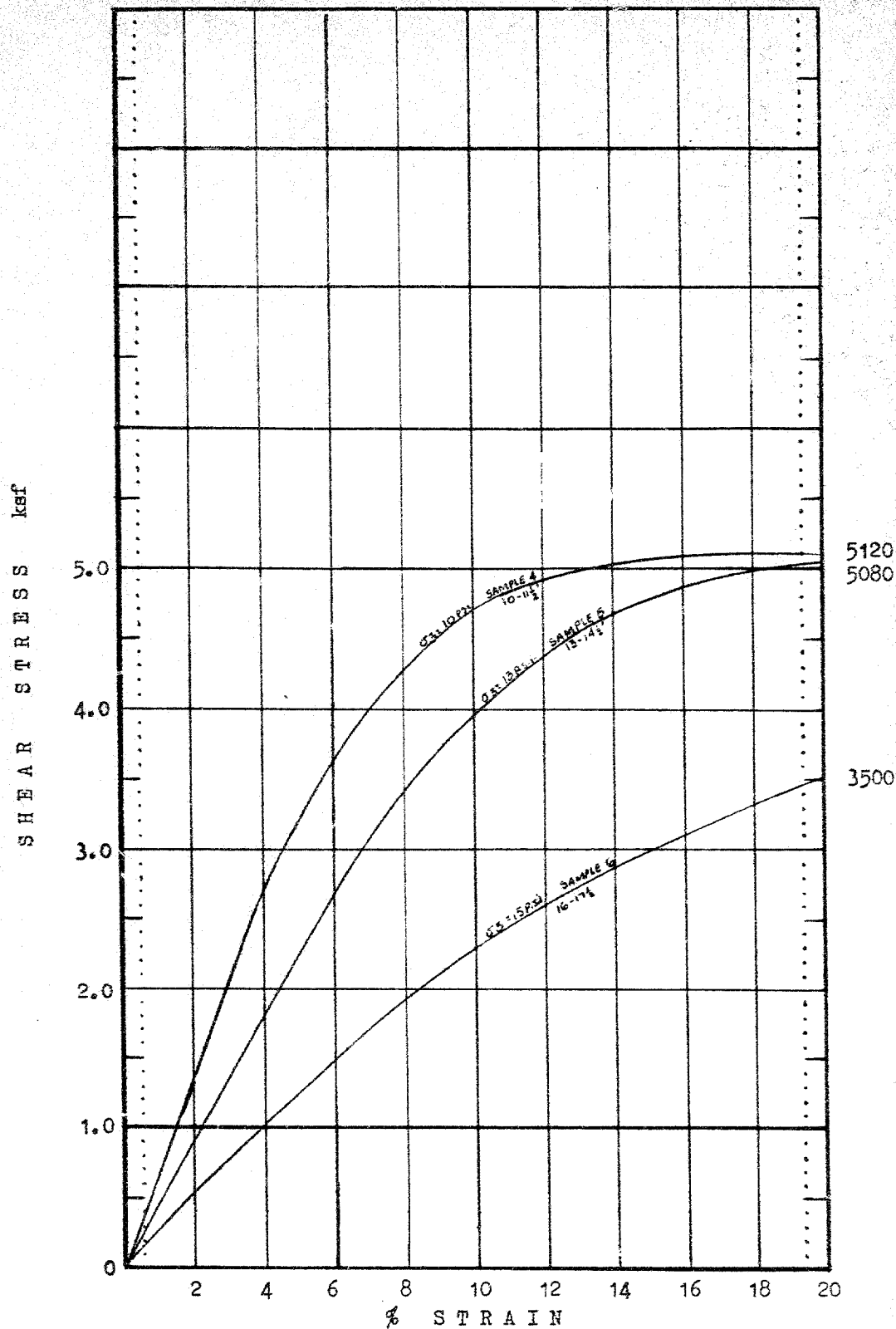
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Highway 401 - Dingman Road Underpass
 LOCATION South of London
 HOLE LOCATION See drawing 1
 HOLE ELEVATION AND DATUM 848.6 BM see hole 1

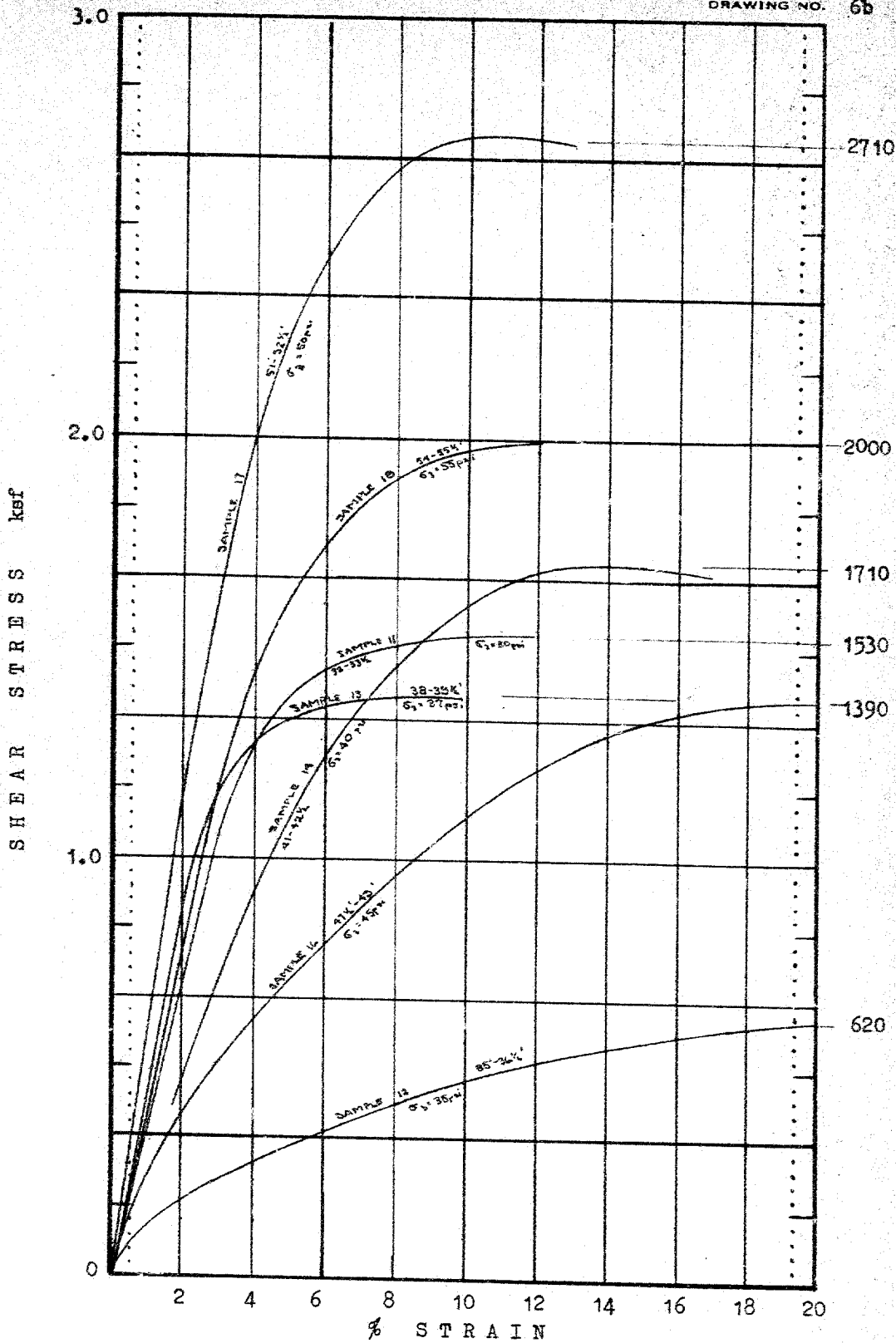
BOREHOLE NO. 4
 FIELD SUPERVISOR
 DRILLER
 PREP.

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

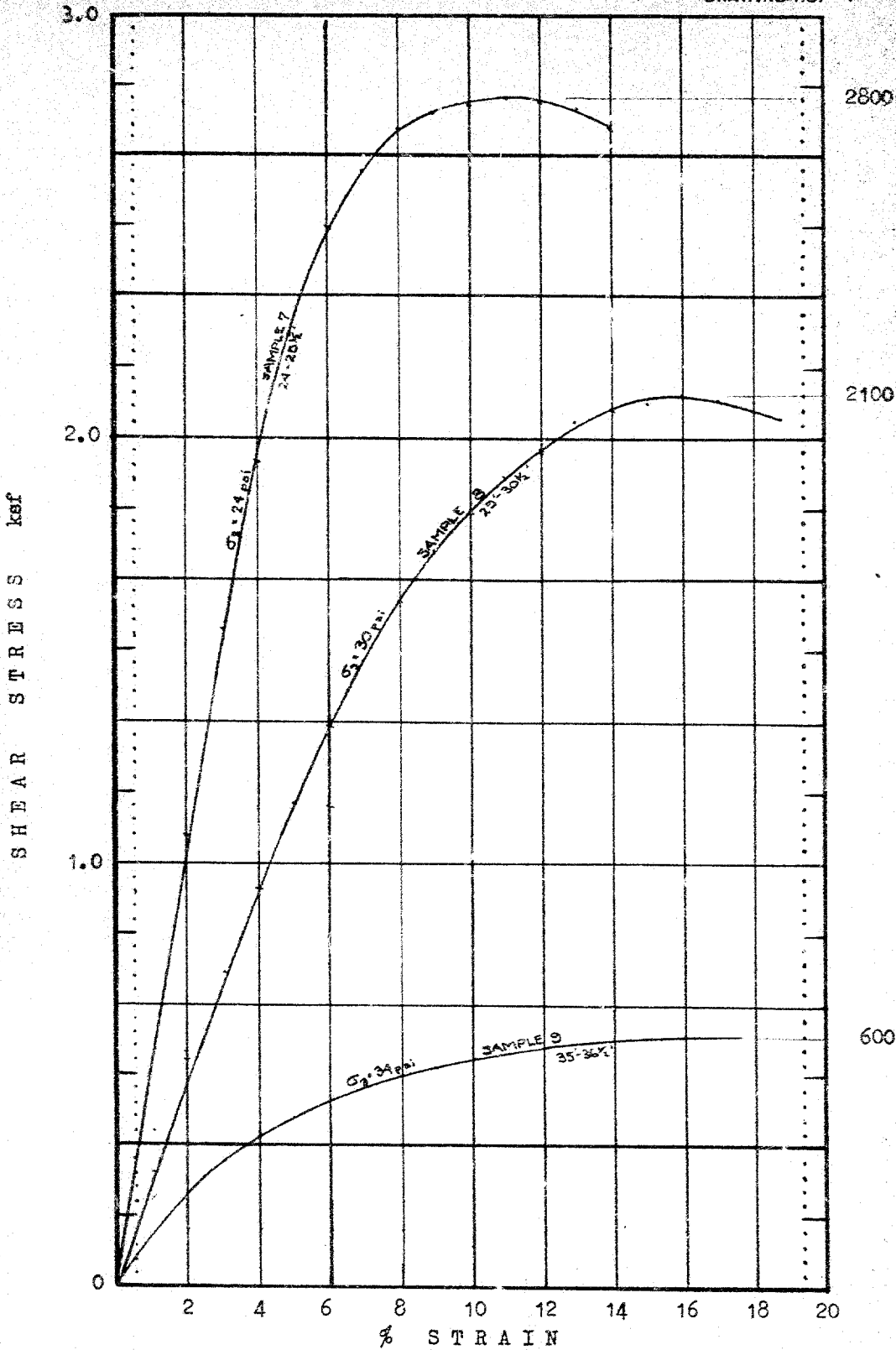




Stress-Strain Curves, Triaxial Tests on Samples Hole 2 Above 18 Ft.



Stress-Strain Curves, Triaxial Tests on Samples Hole 2 Below 20 Ft.

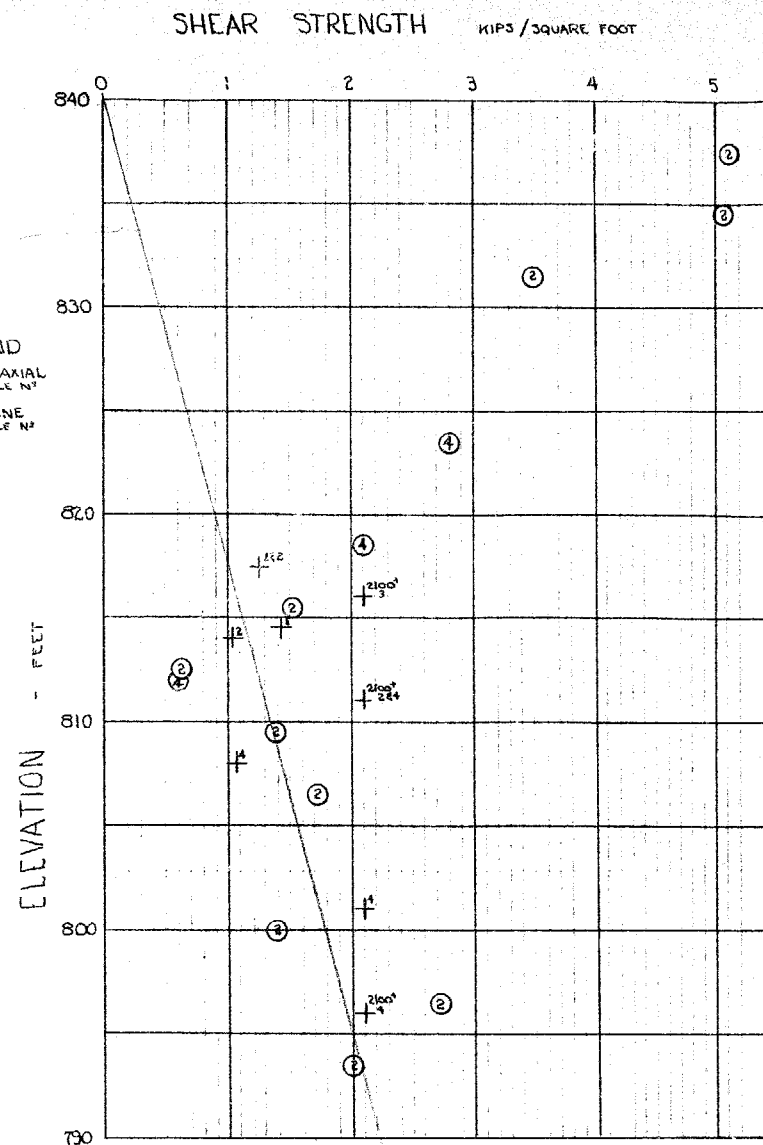


Stress-Strain Curves, Triaxial Tests on Samples from Hole 4



FACTOR OF SAFETY $\frac{18.67}{6.23} = 3$

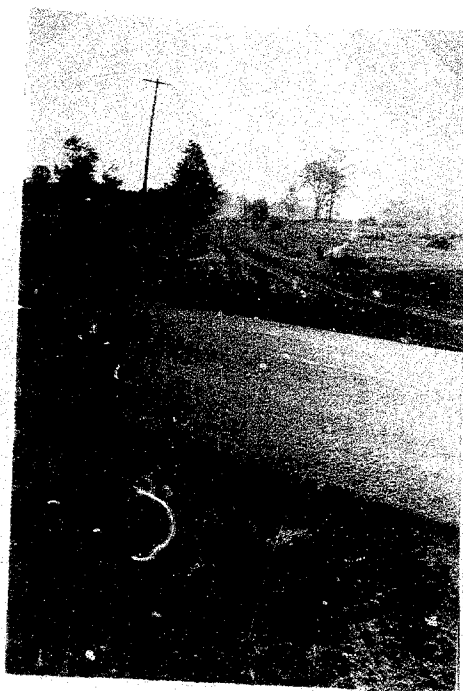
Stability Analysis



Strength-Depth Profile



Dingman Creek Road - Looking North



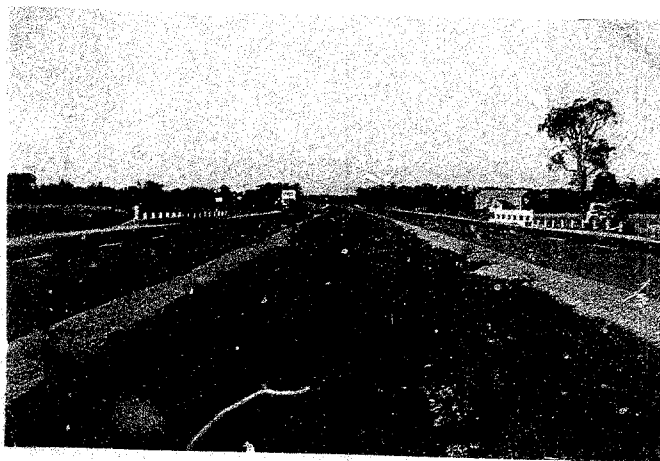
Dingman Creek Road - Looking South



Dingman Creek Road - Looking North



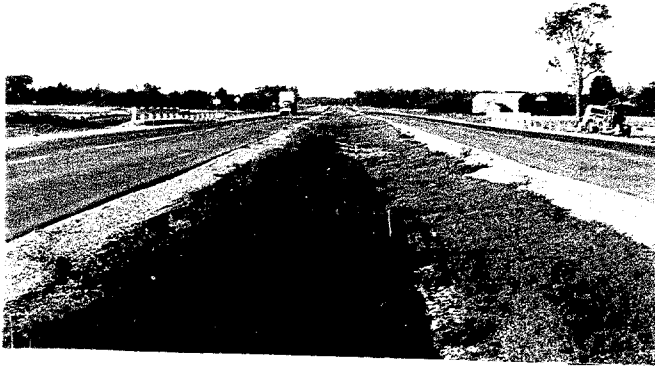
Dingman Creek Road - Looking South



Median Strip of Highway 401 - Looking West
Underpass Site in Foreground



Proposed Underpass Centreline Approaching
Highway 401 from South
Note Shallow Pond of Water in Foreground.



Median Strip of Highway 401 - Looking West
Underpass Site in Foreground



Proposed Underpass Centreline Approaching
Highway 401 from South
Note Shallow Pond of Water in Foreground.



North Shoulder of Highway 401 - Looking West
Hole 4 Located Left - Centre



South Shoulder of Highway 401 - Looking West
Spoil from Hole 2 in Foreground

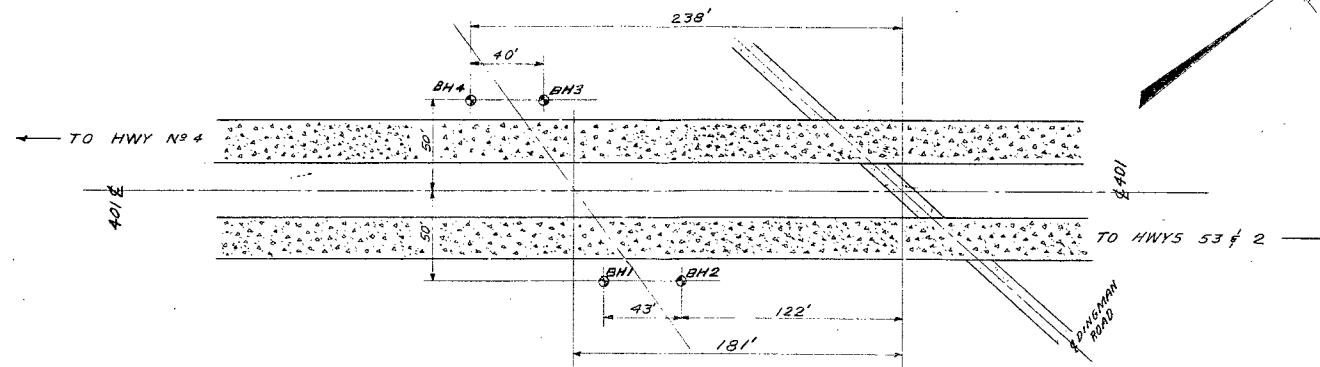


North Shoulder of Highway 401 - Looking West
Hole 4 Located Left - Centre

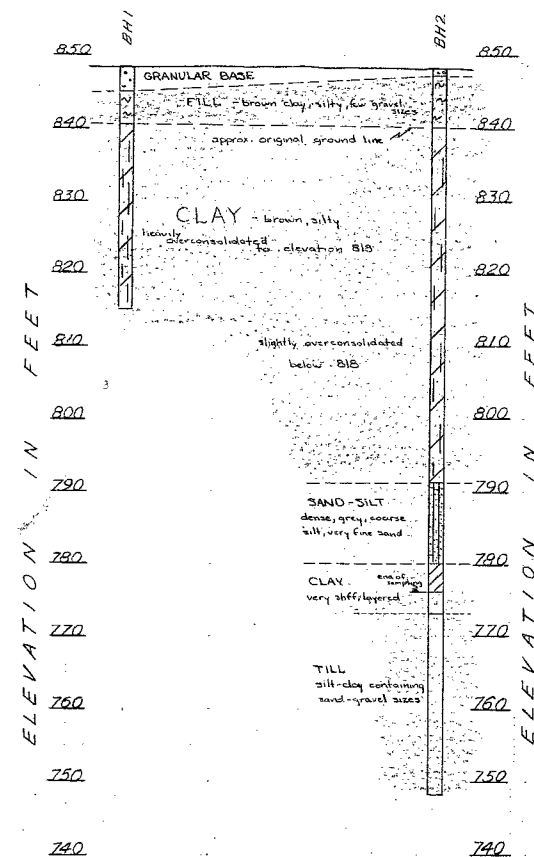


South Shoulder of Highway 401 - Looking West
Spoil from Hole 2 in Foreground

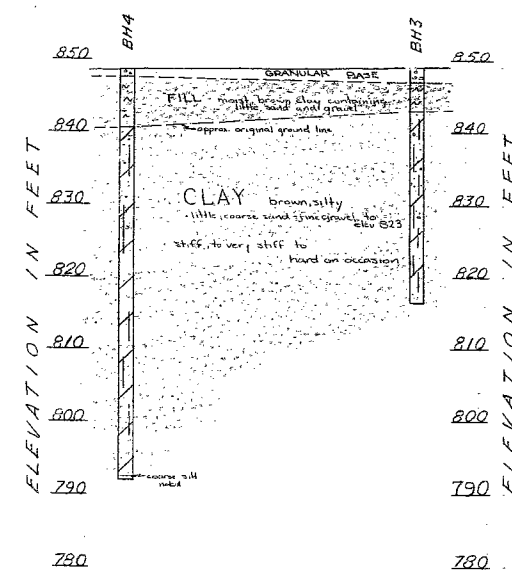
#59-F-237-C
W.P. 22-59
HWY. #401 UNDER-
PASS, DINGMAN
CREEK RD., S. OF
LONDON



PLAN SHOWING LOCATIONS OF BOREHOLES
SCALE 1IN = 40FT



PROFILE BETWEEN BH1 & BH2
HORIZONTAL SCALE 1IN = 10FT
VERTICAL SCALE 1IN = 10FT



PROFILE BETWEEN BH4 & BH3
HORIZONTAL SCALE 1IN = 10FT
VERTICAL SCALE 1IN = 10FT

PROPOSED UNDERPASS
KING'S HIGHWAY 401-DINGMAN CREEK ROAD
SOUTH-WEST OF LONDON
W.P. 22-59

William A Trow & Associates Limited
JOB N° 422 OCT 5 1955