

58-F-205C

W.P. # 702-56

PIKE CREEK

BRIDGE

TECUMSEH

c.c. Mr. A.M. Toye.

Mr. A.M. Toye.

January 16th, 1959.

Bridge Engineer.

Materials & Research.

Re: Subsoil Conditions - Proposed
Pike Creek Structure. Hwy 39. Essex
County. W.P. 702-56.

Attention: Mr. C. Grebski.

As a result of our review of a foundation report submitted by Dominion Soil Test Incorporated, we asked for additional information with respect to spread footing conditions. This factor was not completely looked into in the consultant's report as submitted. The additional information requested is hereto appended.

Footing dimensions have been chosen so that the net bearing pressure to the subsoil does not exceed the safe permissible bearing capacity with respect to shear failure. This value is 1 ton/sq.ft., incorporating a safety factor of 3. As indicated in the supplemental data appended, a total load application of 500 tons through spread footings will give rise to a settlement of the order of 4 to 6 inches.

If additional queries come to mind with respect to footing design of this structure, please contact us.

A. RUTKA.
Mats'ls & Research Engineer.

per:

L. G. Soderman

L.G. Soderman.
Foundation Engineer.

lgs/zw.

enc.

c.c. Mr. A.M. Toye.
Mr. H. McMillan.
Gen. File.

Job No. 105/F85-2

December 29th, 1958

PROPOSED PIKE CREEK BRIDGE

ADDITIONAL SETTLEMENT CALCULATIONS
BASED UPON INFORMATION SUPPLIED BY
THE DEPARTMENT OF HIGHWAYS OF ONTARIO.

ASSUMPTIONS

1. Spread footings for support of abutment loads.
2. Elevations of bottom of footing 566 ft.
3. Footing dimensions 43 ft by 15.5 ft.
4. Abutment load 500 tons.
5. The average natural ground surface elevation is 577.5 ft.
6. The average ground water level is 573.5 ft.
7. The dry unit weight of the loose sandy upper layer is 120 p.c.f.
8. The effective weight of the saturated sandy layer and the silty clay is 60 p.c.f.
9. The silty clay layer is normally consolidated.
10. The Coefficient of Compressibility $C_c = .18$

The results of the following calculations should be viewed with the following considerations.

1. The coefficient of compressibility probably varies through the layers under consideration.
2. The upper clay layers tend to exhibit some dehydration and may therefore tend to consolidate less than the calculated amounts.

3. The foregoing condition may increase the safety against shear failure but will reduce the settlements less than might be expected due to the rafting effect on an otherwise relatively homogeneous layer and the consequent increase in depth to which the effects of consolidation may be expected.
4. The accuracy of the settlement predictions are not likely to be greater than $\pm 30\%$.

The clay subjected to consolidation has been considered in 4 layers. The upper two layers are of 5 ft thickness and the lower two layers of 10 ft thickness.

The overburden above elevation 566 ft consists of:-

4 ft of dry sand effective weight 120 pcf	= 480 psf
6 ft of sand or sandy clay below the water table at 60 pcf	= 240 psf
1.5 ft of silty clay at 60 pcf	= 90 psf
Total overburden at footing level	<u>810 psf</u>

The overburden pressures P_0 at the mid points of the four layers under consideration are therefore -

P_{01}	=	<u>960</u> psf	1080
P_{02}	=	1260 psf	1380
P_{03}	=	1710 psf	1830
P_{04}	=	2310 psf	2430

Assuming that the load distribution with depth is contained within planes inclined at 30° from the vertical then the effective surcharge pressures \bar{A}_p at the mid points of the layers are -

ΔP_1	=	1200 psf
ΔP_2	=	800 psf
ΔP_3	=	510 psf
ΔP_4	=	310 psf

Substituting the above values into the equation

$$S = \frac{C_c}{1 + E_o} \times H \cdot \log_{10} \frac{P_o + \Delta P}{P_o}$$

we have

S_1	=	2.1 ins
S_2	=	1.3 ins
S_3	=	1.3 ins
S_4	=	.65 ins

The total indicated settlement is therefore 5 - 6 ins.

Peter E. Monk
P.E.M. MONK, P.Eng.

BA844

The Department of Highways of Ontario
280 Davenport Road,
Toronto - Ontario

SB F 205 C

RE: SUBSOIL INVESTIGATION FOR
THE PROPOSED PIKE CREEK BRIDGE
NEAR TECUMSEH, ONTARIO

Reference 105/F85

Dominion Soil Investigation Ltd.

November 27th, 1958

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DOMINION SOIL INVESTIGATION LTD.TEST BORING • DIAMOND DRILLING
FOUNDATION DETERMINATION • SOIL MECHANICS

TORONTO 12, ONTARIO

Report No. 105/F85

November 27th, 1958

FOUNDATION INVESTIGATION FOR
PROPOSED PIKE CREEK BRIDGE
NEAR TECUMSEH, ONTARIOPURPOSE OF THE INVESTIGATION AND SCOPE OF THE REPORT

1. The investigation was undertaken to determine the foundation conditions at the above-mentioned site. The report covers the field work undertaken, and the results of laboratory tests on selected samples together with an analysis of the alternative methods supporting the proposed structure and estimates of the probable settlements.

LOCATION OF THE SITE AND BOREHOLES

2. The proposed Pike Creek Bridge is on the proposed rerouted Highway No. 39 approximately 2 miles East of Tecumseh in the County of Essex. A sketch plan of the area illustrating the location of the site is shown on Enclosure No. 1. The borehole locations are shown on the plan on Enclosure No. 2.

FIELD INVESTIGATION AND DESCRIPTION OF THE SUBSOIL

3. The field work was commenced on October 7th, 1958 and completed on October 17th. Six boreholes were drilled, two to a depth of 80 ft. and four to a depth of 50 ft. Two cone penetration tests were also

completed.

Drilling was carried out using a conventional diamond drill adapted for soil sampling. Samples were obtained by means of a 2 in. O.D. standard split spoon or using a 2 in. I.D. thin walled shelly tube. 'In situ' vane shear tests including both undisturbed and disturbed readings were taken at depths intermediate between the normal 5 ft. sampling intervals. Penetration of the standard split spoon and the 2 in. diameter cone was achieved by means of a 140 lb. hammer falling freely through a height of 30 ins.

The subsoil conditions over the whole site are relatively uniform and consist of an upper layer of approximately 10 ft. of loose clayey sand and silty sand underlain by soft or medium stiff silty clay or clayey silt to a depth of 80 ft. This silty clay and clayey silt contains some fine gravel and sand and is particularly silty below a depth of 70 ft. below ground surface.

RESULTS OF LABORATORY TESTS

4. The results of laboratory tests are shown in the Appendix. These tests indicate that the silty clay or clayey silt has a low plasticity index of between 9 and 18 and that the soil becomes generally somewhat more silty and granular with depth.

A comparison between the 'in situ' vane shear strength results and the values of cohesive shear strength as determined by the unconfined compression test indicates that the undisturbed vane shear strength is appreciably higher than the cohesive shear strength. Even allowing for the sensitivity indicated by the disturbed vane

strengths the 'in situ' strength is higher than the indicated cohesive strength. This suggests that the soil has some angle of internal friction, as might be expected from its silt and granular content.

DISCUSSION OF THE RESULTS

5. The silty clay and clayey silt underlaying the upper 10 ft. layer of silty sand is soft to medium stiff. The cohesive shear strength appears to be somewhat variable, but the general pattern is one of slightly decreasing cohesive shear strength with increasing depth below ground surface, but this may be due to the fact that below a depth of 45 ft. there is a tendency for vane shear test results to be more reliable than unconfirmed test results. The sand and gravel content suggests that the subsoil may be reworked glacial till. This could account for the somewhat irregular cohesive shear strength results. The results of the 'in situ' shear tests show much more consistency and as has been mentioned previously the 'in situ' shear strength is consistently markedly higher than the cohesive shear strength as indicated by the unconfirmed compression tests.

There appear to be two methods of obtaining support for the proposed bridge (a) by means of friction piles

(b) by supporting the structure on large box caissons.

The span of the proposed bridge is approximately 80 ft. As an initial estimate it will be assumed that the reactions for a single span will be of the order of 250 kips.

STRUCTURE SUPPORTED ON PILES

Friction piles may be used for the support of this structure, but it is felt that the value given to the skin friction should be very conservative. The reasons for a conservative outlook are

- (1) The somewhat variable nature of the cohesive shear strength results.
- (2) The silt content and the presence of sand and fine gravel.
- (3) The evidence that bridges in Essex County have failed due to either individual or pile group failure.

Consideration must also be given to the sensitivity of the subsoil as indicated by the 'in situ' vane tests. The indicated sensitivity is less than 2, but it seems fairly certain that an appreciable percentage of the 'in situ' shear strength is due to the internal friction of the soil.

It follows therefore that the sensitivity of that part of the strength which is due to cohesion may be higher than 2. Cohesive shear strength values between depth of 15 ft. and 30 ft. in borehole No. 6 are of the order of 600 p.s.f. Below 30 ft. the strength drops to the order of 300 - 400 p.s.f. In borehole No. 3 however, the cohesive shear strength in the upper clay layers is 300 - 400 p.s.f.

It seems advisable therefore to assume that the soft clay has a cohesive shear strength not in excess of 300 p.s.f. ultimate for design purposes. Assuming an overall sensitivity of 1.3 and a factor of safety of 3, the allowable cohesive shear stress is

$$\frac{300}{3 \times 13} = 77 \text{ p.s.f.}$$

150 lb/ft² O.K. verbally by P. Monk
Dec 11th.

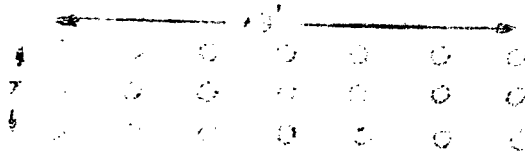
If it is assumed that wood piles of 12 in. diameter are to be used the allowable load per pile is $L \times 77$ lbs.

Where L is length of pile in the clay.

If L is 50 ft. then the allowable load per pile is 12,000 lbs.

Assuming that the abutment reaction is 250 Kips then the number of piles per abutment is 21.

Assuming that the piles are to be placed at a maximum spacing of 3 diameters on a square pattern then a convenient layout is 3 rows of 7 piles.



Checking for group action the group perimeter is 52 ft.

Group shear is therefore $50 \times 77 \times 52$ lbs. or 200 Kips i.e. the piles would fail as a group.

In order to prevent group failure the perimeter must be increased to 65 ft. This could be achieved by placing the piles at 4 ft. centres.

SETTLEMENT ESTIMATE FOR PILES

It is usual to assume that the load on the subsoil due to the pile friction acts at a distance one third of the pile length from the bottom of the pile or at a depth into the clay of 33 ft. i.e. a depth of approximately 43 ft. below ground surface.

Consider the consolidation characteristics of a sample taken from a depth of 35 ft. (see Enclosure No. 12) it will be noticed that the preconsolidation pressure indicated by the test results is approximately 1300 lbs. This does not agree with the existing overburden pressure of approximately 2100 lbs. This loss in apparent preconsolidation pressure may be due to the sensitivity of the material.

Considering the settlement characteristics of a sample representative of the more clayey soil with a Plasticity Index of 17. The results of a consolidation test on such a sample are to be found on Enclosure No. 12.

The settlement S of a strip of soil H thick is given by the expression -

$$S = \frac{C_c}{1+e_0} H \log \frac{p_0 + \Delta p}{p_0}$$

Where C_c is the Compression Index

e_0 is the initial voids ratio of the soil under p_0

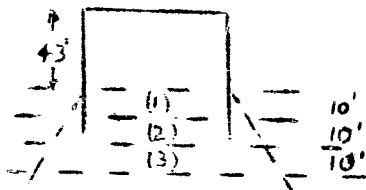
H is the depth of the strata under compression

p_0 is the initial pressure

Δp is the increase in pressure

Considering the previous tentative design where the length of pile in the clay is 50 ft. and the pile group has dimensions of 25 ft. by 9 ft. then the accompanying figure illustrates the three stages

for the calculation of the settlement.



Calculating P_0 for the mid-point of each of sections (1) (2) and (3)

$$P_{01} = 5 \times 120 + 5 \times 60 + 38 \times 60 = 3130 \text{ lbs./ft}^2$$

$$P_{02} = 3130 + 10 \times 60 = 3730 \text{ "}$$

$$P_{03} = 3730 + 10 \times 60 = 4330 \text{ "}$$

$$\Delta P_1 = 750 \text{ lbs./ft}^2$$

$$\Delta P_2 = 420 \text{ "}$$

$$\Delta P_3 = 270 \text{ "}$$

Then

$$S_1 = \frac{10 + 12}{1.67} \times .18 \quad \log \quad \frac{3130 + 750}{3130}$$

$$= \frac{120}{1.67} \times .18 \times .09 = 1.16 \text{ ins.}$$

$$S_2 = \frac{10 + 12}{1.66} \times .18 \quad \log \quad \frac{3730 + 420}{3730}$$

$$= \frac{120}{1.66} \times .18 \times .046 = .6 \text{ ins.}$$

$$S_3 = \frac{10 + 12}{1.65} \times .18 \quad \log \quad \frac{4320 + 270}{4330}$$

$$= \frac{120 + 18}{1.65} \times .025 + .33 \text{ ins.}$$

Total indicated settlement therefore - 2 to 2.5 ins.

If Piles with a shorter embedded length are used the settlement will be slightly higher.

BOX CAISSONS

Another alternative is to place the structure on box caissons. Such caissons would have to be placed at least 15 ft. below existing ground level and at the abutment this would mean the removal of a soil load at the 15 ft. depth of approximately 1150 lbs. Replacing the earth load by an abutment load of 1500 lbs. would result in settlements of the order of 2 - 3 ins. assuming that the soil consists of material with a plasticity index of 17. The Atterberg results indicate that 17 is the upper limit of Plasticity Index for the material encountered.

For piers placed in the form of box caissons the conditions are somewhat less suitable. Firstly because the pier loads would be greater than the abutment loads and secondly because the overburden in the river bed is some 10 ft. less than at the abutment. The subsoil below the river bed has in the past been subjected to pressures similar to that existing at the abutments but there is bound to have been some loss of preconsolidation pressure.

It would therefore be desirable to place pier caissons deeper than the abutment caissons, say at a depth of 10 ft. below the river bed. At this depth the initial preconsolidation pressure was probably of the order of 1400 - 1500 p.s.f. The existing overburden pressure is 600 p.s.f. A reasonable estimate of the

existing apparent preconsolidation pressure is therefore probably about 1000 p.s.f.

If the pier caisson loading were kept to 1500 p.s.f. then the expected settlement would probably be of the order of 3 - 4 ins. The increase in settlement over the abutments being due to the increased size of the caisson required for the pier. It has been assumed that simply supported spans would be used.

It is apparent that piles are more convenient than caissons and therefore no consideration has been given to the stability of caissons against general shear failure since this would require a determination or estimation of the internal friction of the silty clay.

CONCLUSIONS

6. From the foregoing, the following major points may be determined:-
 1. The value of cohesive shear strength used for pile calculations should be conservative.
 2. The value of allowable pile skin frictions should be taken to be 75 p.s.f. for all depths.
 3. The 'in situ' shear strength is considerably higher than the cohesive shear strength even in the disturbed condition. This indicates that the material has an angle of internal friction greater than zero.
 4. The use of box caissons is possible, but it appears that the cost of such an arrangement would probably be higher than piles, and further the probable settlements are higher.

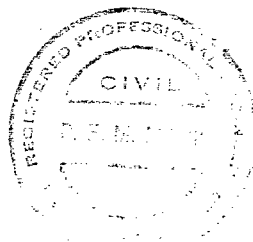
5. Using piles the structure may be either single or multispan, but if multispan, it is felt that the design should be simply supported construction since the possibility of differential settlement greater than 1 ins. is high.

6. Using piles the settlement should be of the order of 2 to 2.5 ins. for the tentative design suggested in this report.

It should be noted however that for different pile groupings and lengths of piles the settlements should be rechecked.

From the foregoing it appears that a conventional friction pile foundation using a conservative value for the cohesive shear strength of the silty clay is likely to prove the most economic and convenient form of foundation support.

Robert G. Monk
P.E.M. Monk, P.Eng.



A P P E N D I X

RESULTS OF LABORATORY TESTS

-11-

Borehole No.	Sample	Depth in ft.	Unconfined Cohesive Shear Strength p.s.f.	Moisture Content Percent	Liquid Limit	Plastic Limit	Plasticity Index	Unit Weight p.c.f.	Description
1	SS 4				29.0	16.5	12.5		Grey silty clay with fine gravel
1	SS 5				28.8	15.2	13.6		" "
1	SS 6				31.8	17.1	14.7		" "
1	SS 7				29.6	16.7	12.9		" "
1	SS 8				31.4	18.0	13.4		" "
1	SS 9				32.1	17.0	15.1		" "
1	SS 10				31.3	16.8	14.5		" "
1	SS 11				29.7	16.1	13.6		" "
1	SS 12				29.3	16.4	12.9		" "
1	SS 13				30.9	17.8	13.1		" "
1	SS 14				29	16.3	12.7		" "
3	TW 5	20	332	22.3	29.7	16.9	12.8	120.8	Soft grey silty clay
3	TW 6	25	430	21.2	31.2	17.0	14.2	116	" "
6	SS 3	11			28.6	13.5	15.1		Soft grey Silty clay with some fine gravel
6	TW 4	15	675	21.7	31.8	14.5	17.3	123.5	" "
6	TW 5	20	630	24.3	29.7	14.5	15.2	129.1	" slightly more silt

-12-

Borehole No.	Sample	Depth in ft.	Unconfined Cohesive Shear Strength p.s.f.	Moisture Content Percent	Liquid Limit	Plastic Limit	Plasticity Index	Unit Weight p.c.f.	Description
6	SS 6	24			29.5	19.3	10.2		Soft grey silty clay with some fine gravel slightly more silt
6	TW 8	35	630	25.8	32.8	16.7	16.1	125.0	grey silty clay some fine gravel
6	TW 9	38	595	30.7	39.7	21.8	17.9	124	" "
6	SS 10	45			32.1	17.2	14.9		" more silt
6	TW 11	50	305	27.2	26.5	15.4	11.1		" "
6	TW 12	59		31.7	29.5	17.0	12.5		soft grey silty clay with some very fine gravel
6	SS 13	63			31.4	18.4	13.0		" "
6	SS 15	71			32.7	17.5	15.2		" "
5	TW 16	75	194	25.0	23.7	14.0	9.7	125.0	Grey sandy silt
6	SS 17	77			25.8	16.7	9.1		Grey sandy silt with fine gravel

Borehole No.	Sample	Depth in ft.	Unconfined Cohesive Shear Strength p.s.f.	Moisture Content Percent	Liquid Limit	Plastic Limit	Plasticity Index	Unit Weight p.c.f.	Description
6	SS 18	80			19.0	17.0	2.0		Grey sandy silt with fine gravel

The results of two consolidation tests are shown on Enclosures 11 and 12.



LAKE ST CLAIR

Hwy N° 39

C.N.R.

LINE 'B'

BRIDGE SITE

TO BELLE RIVER

C.P.R.

TRIE CREEK

201

TO Hwy N° 2

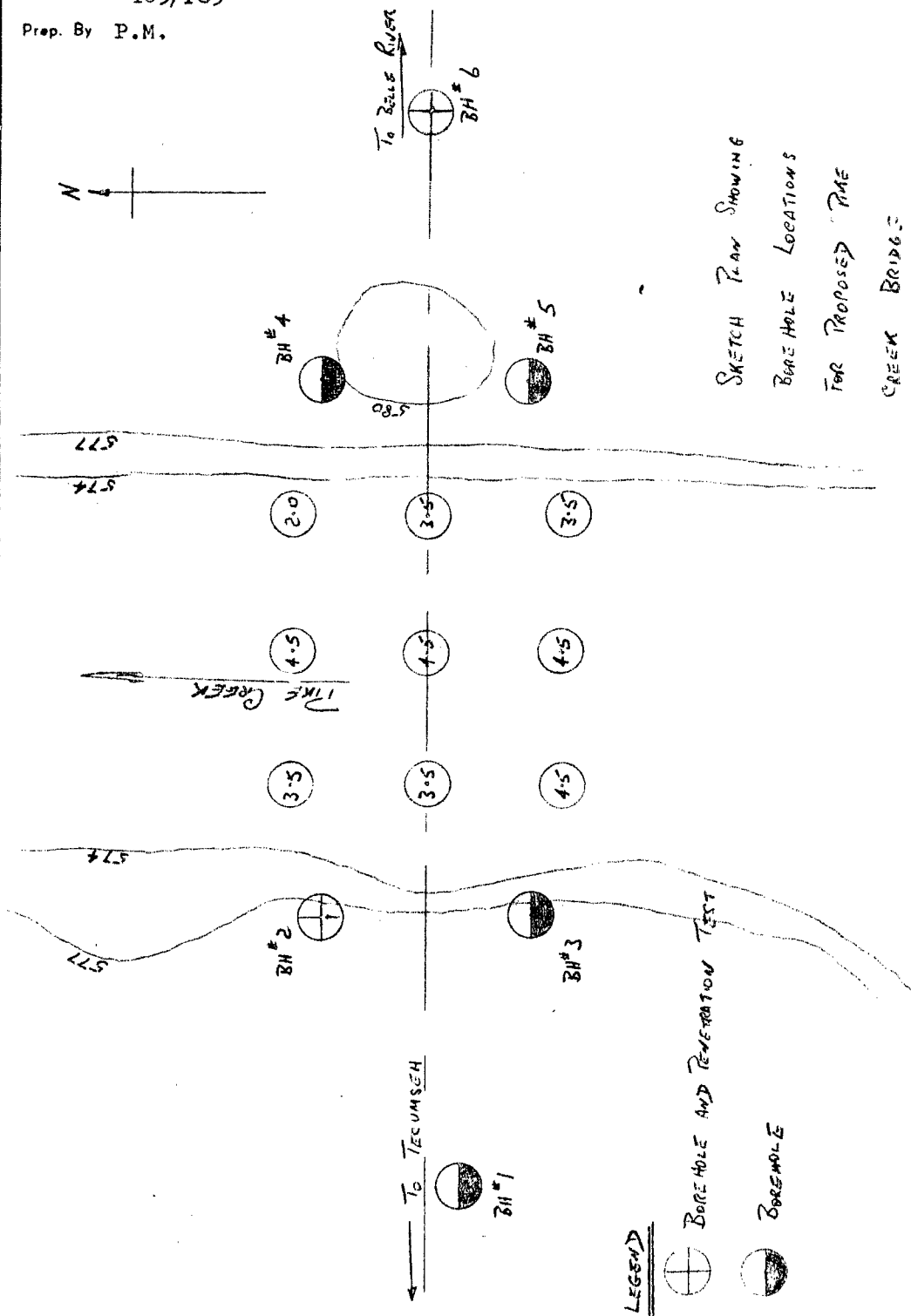
TO MADISON

TECUMSEH

SKETCH PLAN SHOWING
LOCATION OF PROPOSED
TRIE CREEK BRIDGE

SCALE 1 IN TO 1/2 MILE

Prep. By P.M.



Dominion Soil Investigation Ltd.Engineering Data Sheet for Borehole: **I**Date: **16/10/58**

Project: **Proposed Pike Creek Bridge**
 Location: **Relocated Hwy 39 Nr Windsor**
 Hole Location: **See Enclosure No 2**
 Hole Elevation and Datum: **579 geo contours**
 Field Supervisor: **H.P.** Prep.: **P.M.**
 Driller: **C.S.** Checked:

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing

⊕

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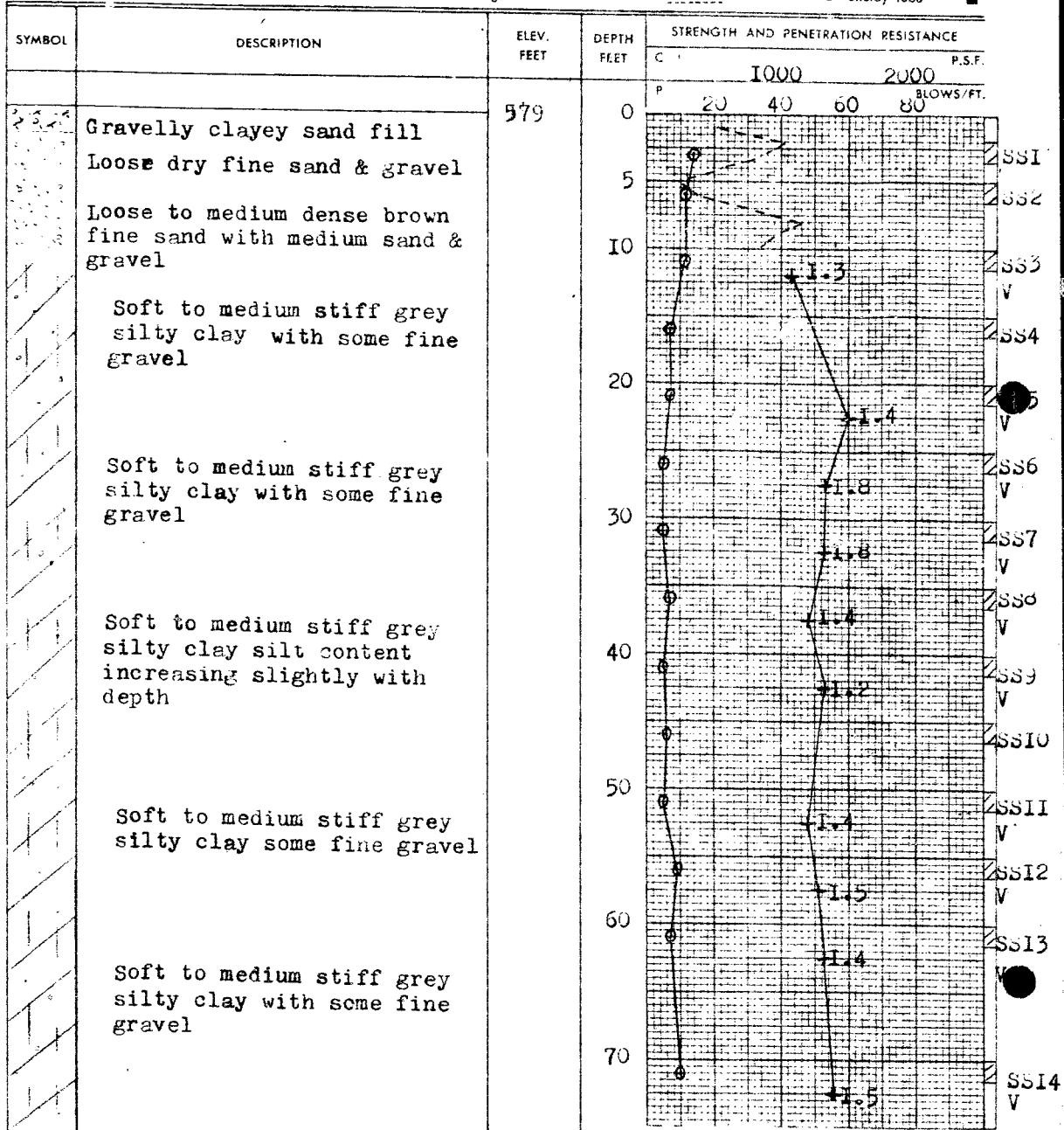
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Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Foundation Engineering Division

Engineering Data Sheet for Borehole: I

Date: 26/11/58

LEGEND

Consistency

Natural moisture and

Liquidity Index (%)

Liquid limit

Plastic limit

— — — — —

2:1

— 0

)—

Sampling Method

2" Dia. split tube

2" Shelby tube



DEPTH FEET	CONSISTENCY					SAMPLE	NATURAL UNIT WT. P.C.F.	REMARKS
	MOISTURE CONTENT, % DRY WEIGHT							
0	10	20	30	40	50			
						SS1		
						SS2		
10						SS3		
						SS4		
20						SS5		
						SS6		
30						SS7		
						SS8		
40						SS9		
						SS10		
50						SS11		
						SS12		
60						SS13		
70						SS14		

Dominion Soil Investigation Ltd.**Engineering Data Sheet for Borehole: I cont.**

Date: 16/10/58

Project: Proposed Pike Creek Bridge

Location: Relocated Hwy 39 Nr Windsor

Hole Location: See Enclosure No 2

Hole Elevation and Datum: 579 Geo. contour

Field Supervisor: H.P. Prep.: P.M.

Driller: A.B.

Checked:

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

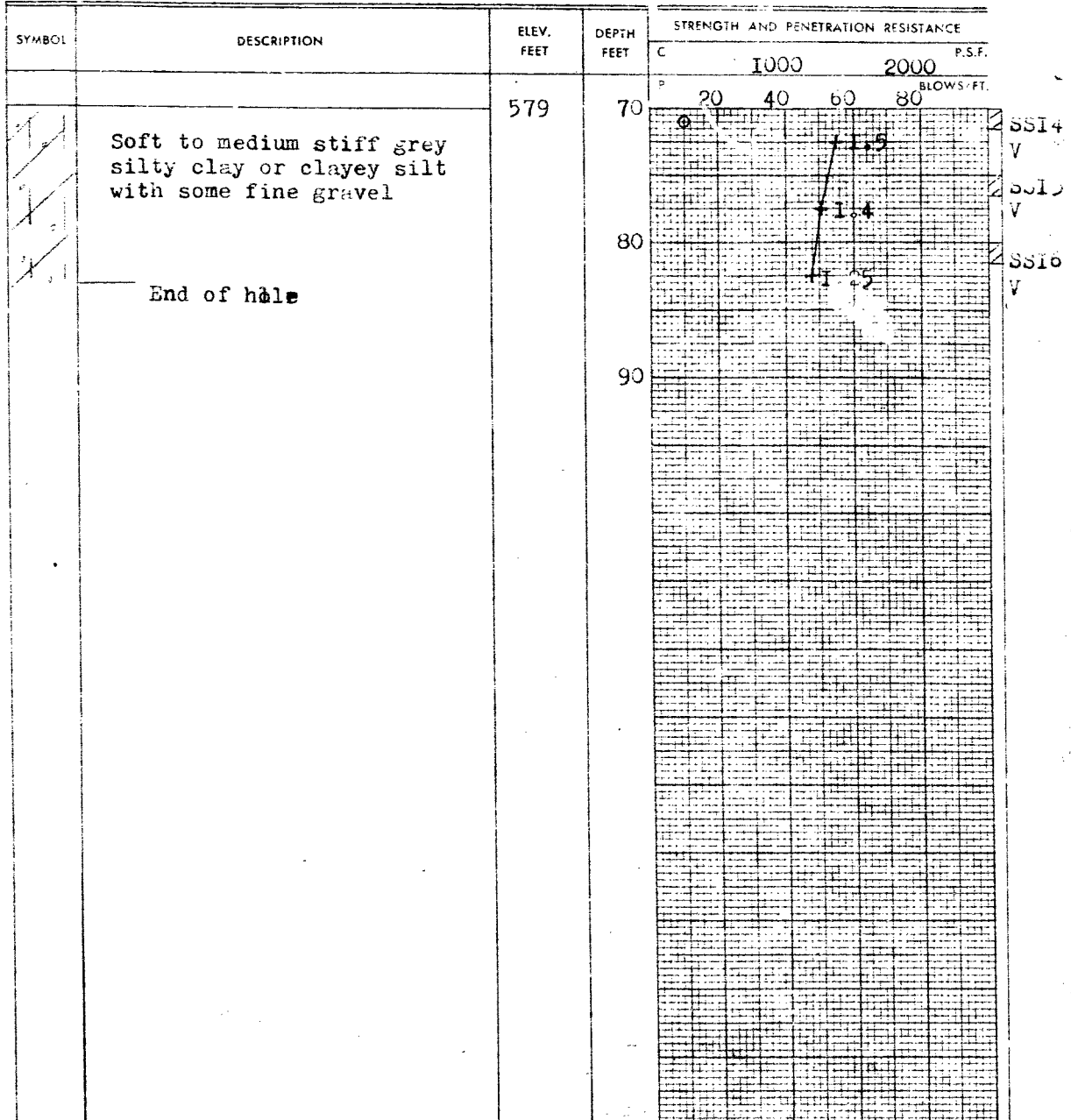
2" Dia. Cone

Casing

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 2

Date: 17/10/58

Project: Proposed Pike Creek Bridge
Location: Relocated Hwy 39 Nr Windsor
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 573 Geo. contour
Field Supervisor: H.P. Prep.: P.M.
Driller: C.S. Checked:

LEGEND
Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing.

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

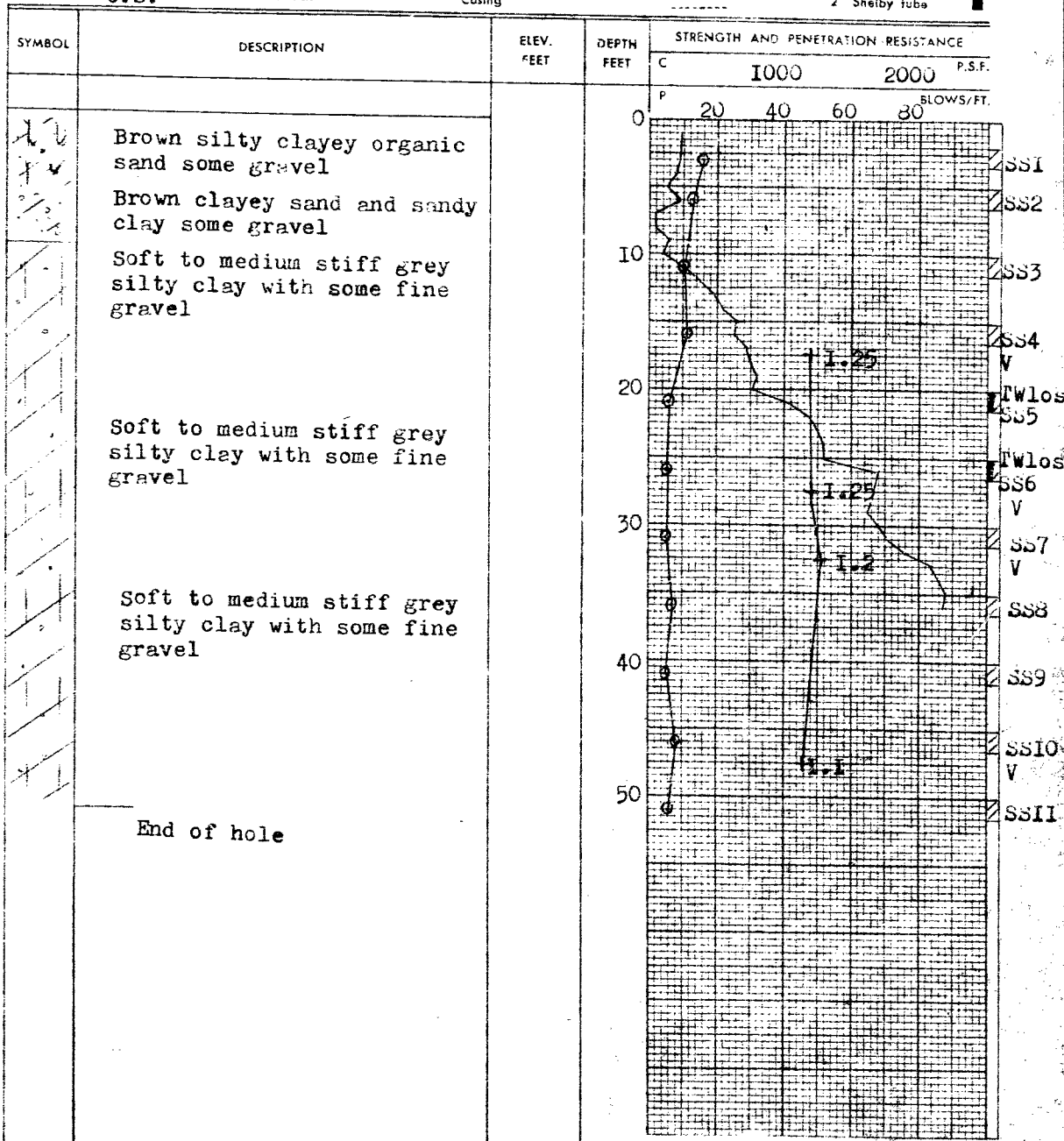
2" Dia. Cone

Casing

Sampling Method

2" Dia. split tube

2' Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 3

Date: 17/10/58

Project: Proposed Pike Creek Bridge.

Location: Relocated Hwy 39 Nr Windsor

Hole Location: See Enclosure No 2

Hole Elevation and Datum: 578 Geo. contour

Field Supervisor: H.P. Prep.: P.M.

Driller: C.S. Checked:

LEGEND

Shear Strength (C)

Unconfined compression
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

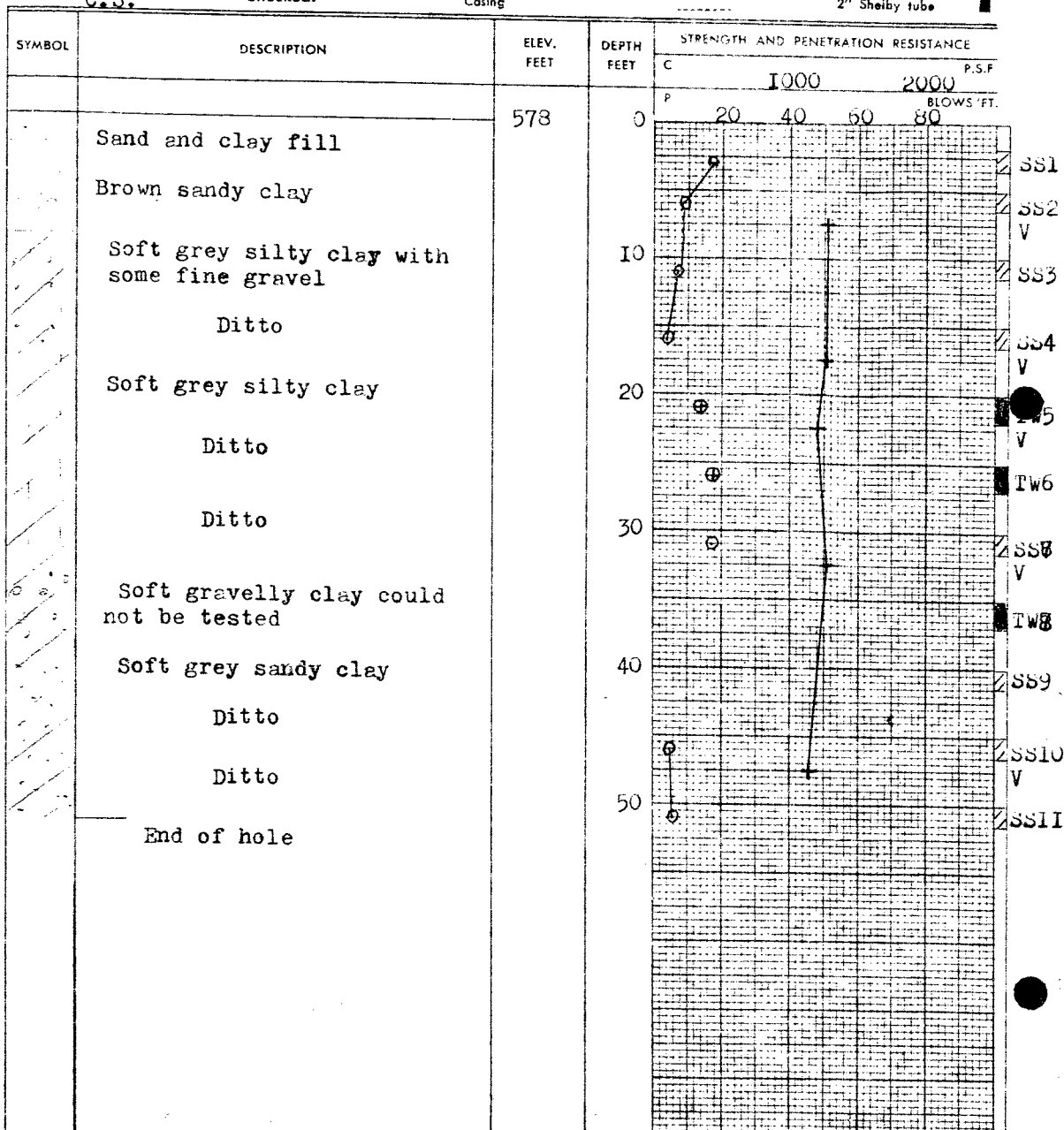
2" Dia. Cone

Casing

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Foundation Engineering Division

Engineering Data Sheet for Borehole:

Date: 26/11/58

LEGEND:**Consistency**

Liquid Limit and

Plasticity Index

Flow Value

Preconsolidation

Liquid Limit

Plasticity Index

Flow Value

Preconsolidation

Sampling Method

2" Dia. split tube

2" Shelby tube



DEPTH FEET	CONSISTENCY				SAMPLE	NATURAL UNIT WT. P.C.F.	REMARKS
	MOISTURE CONTENT % DRY WEIGHT						
0	10	20	30	40	50		
					SSI		
					SS2		
10					SS3		
					SS4		
20					TW5	120.8	
					TW6	116	
30					SS7		
					TW8		
40					SS9		
					SS10		
50					SS11		

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 4

Date: I7/10/58

Project: Proposed Pike Creek Bridge

Location: Relocated Hwy 39 Nr Windsor

Hole Location: See Enclosure No 2

Hole Elevation and Datum: 579.5 Geo. contour

Field Supervisor: H.P. Prep.: P.M.

Driller: C.S.

Checked:

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

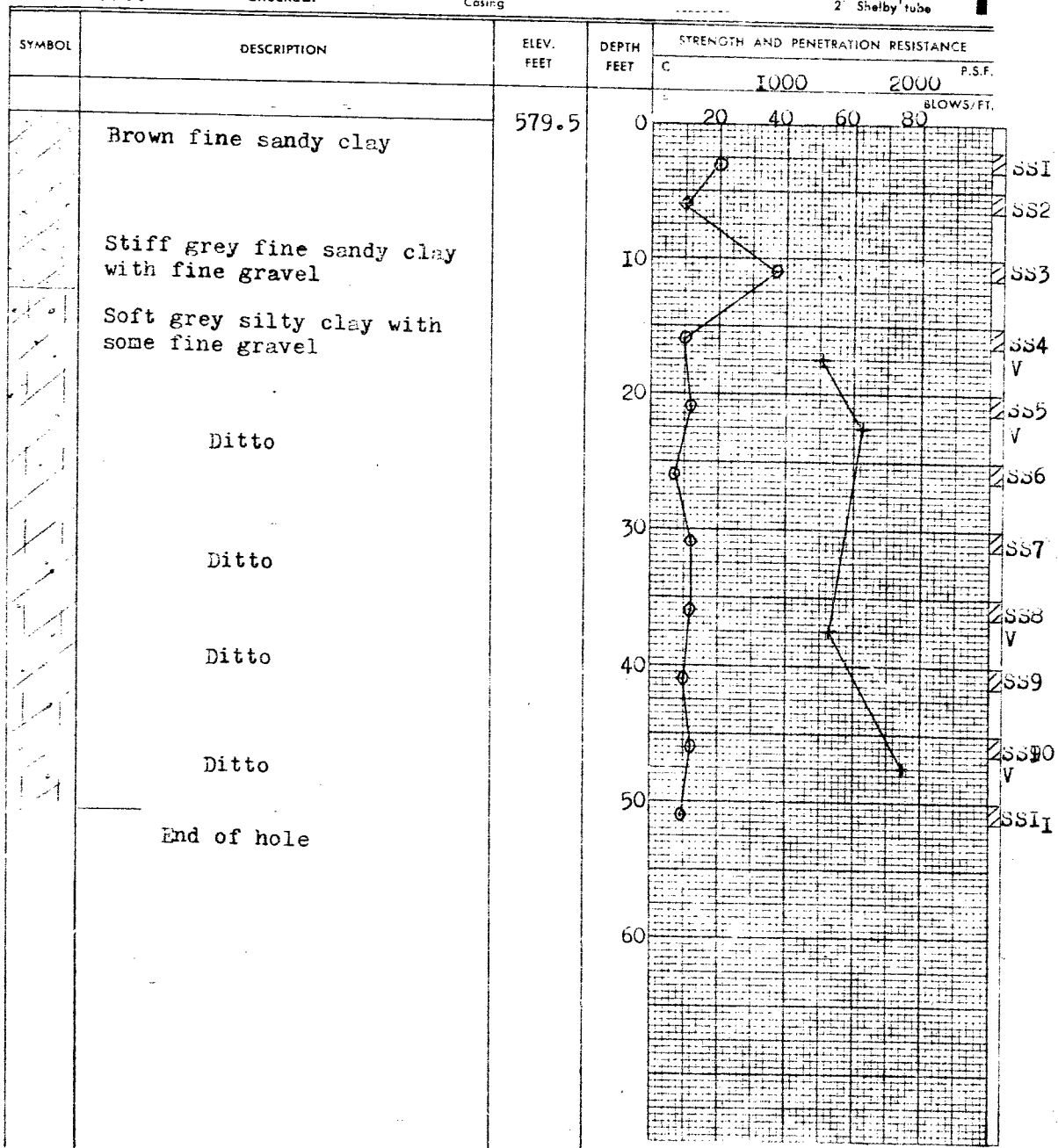
2" Dia. Cone

Casing

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 5

Date: 21/10/58

Project: Proposed Pike Creek Bridge
 Location: Relocated Hwy 39 Nr Windsor
 Hole Location: See Enclosure No 2
 Hole Elevation and Datum: 579 Geo. contour
 Field Supervisor: H.P. Prep.: P.M.
 Driller: C.S. Checked:

LEGEND

Shear Strength (C)

Unconfined compression
 Vane test and sensitivity (S)

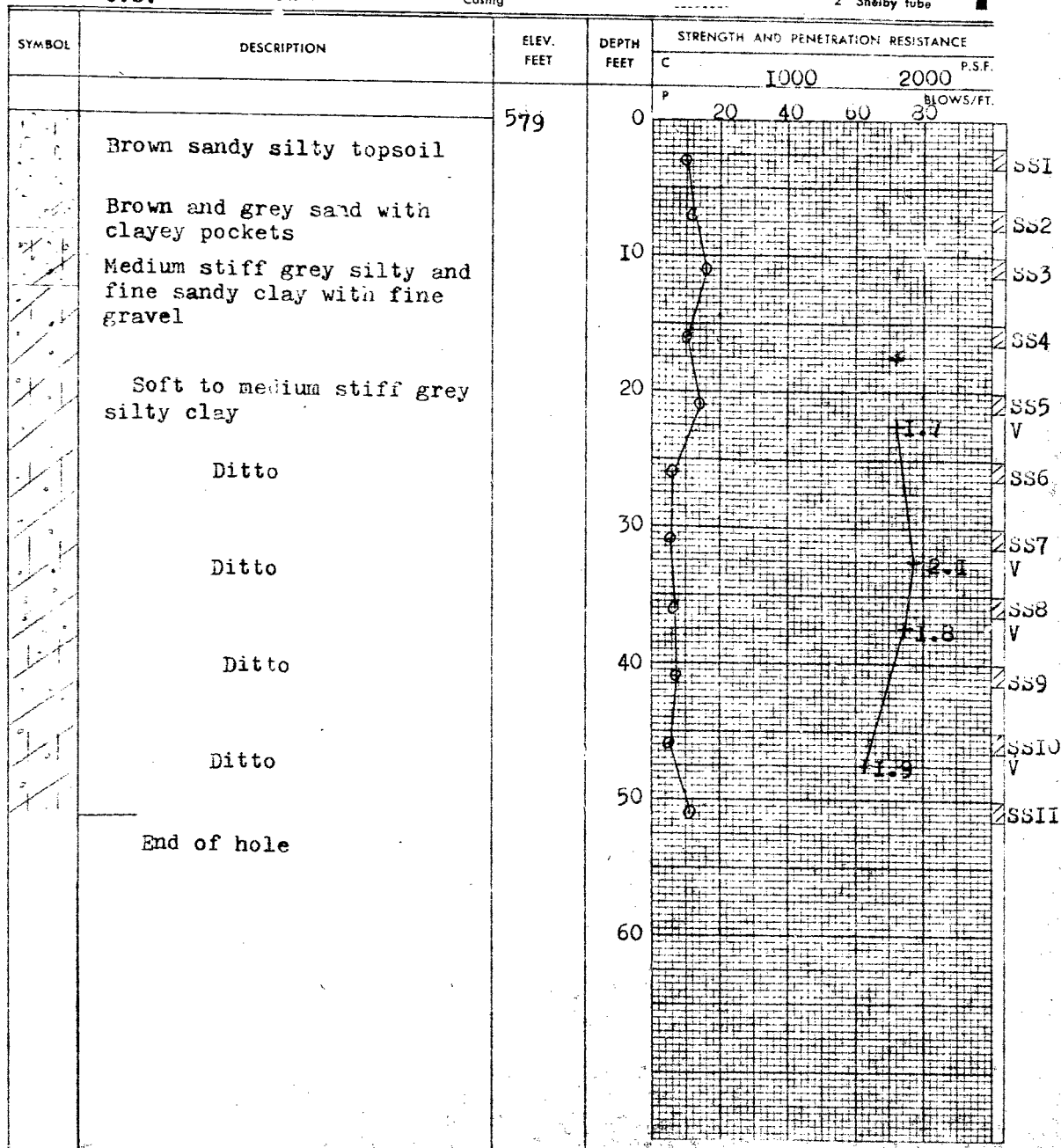
Penetration Resistance (P)

2" Split tube
 2" Dia. Cone
 Casing

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 6

Date: 21/10/58

Project: Proposed Pike Creek Bridge

Location: Relocated Hwy 39 Nr Windsor

Hole Location: See Enclosure No 2

Hole Elevation and Datum: 578.5 Geo. contour

Field Supervisor: H.P.

Prep.: P.M.

Driller:

C.S.

Checked:

LEGEND

Shear Strength (C)

Unconfined compression
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

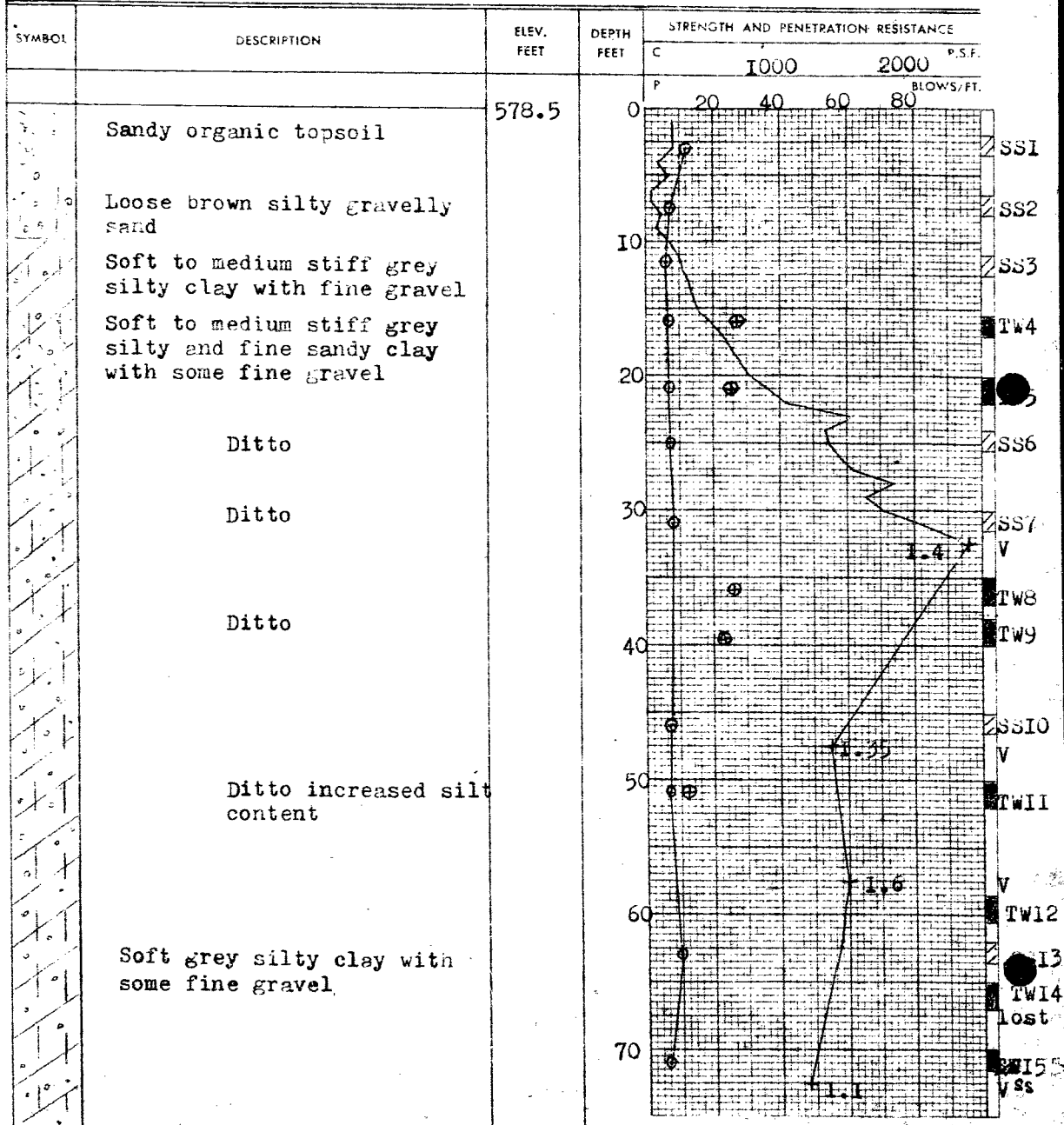
2" Dia. Cone

Casing

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 6

Date: 26/11/58

LEGEND**Consistency**Natural moisture and
Liquidity Index (LI)

Liquid limit

Plastic limit

—*—

x LI

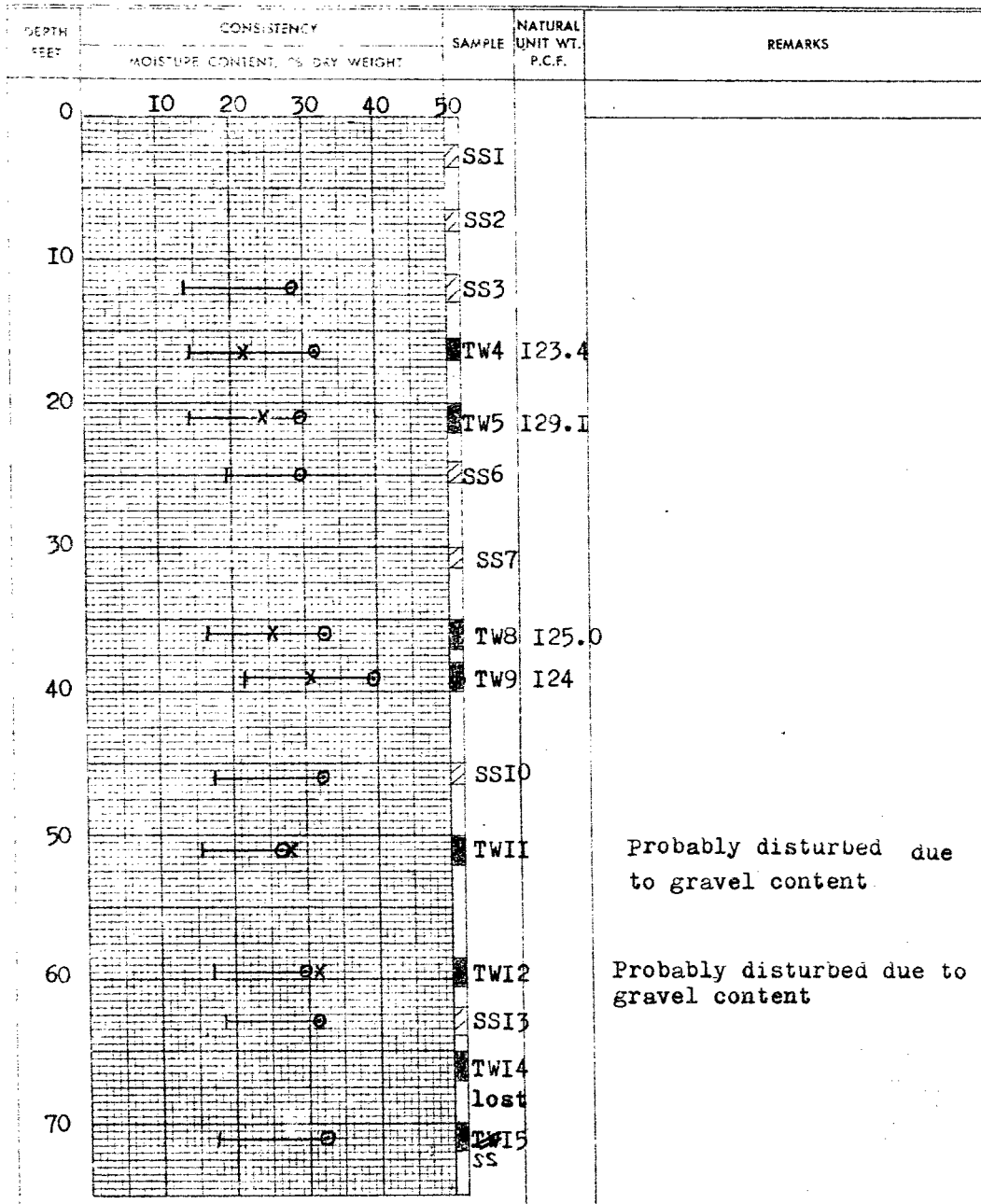
—o—

—

Sampling Method

2" Dia. split tube

2" Shelby tube



Engineering Data Sheet for Borehole: 6

Project: Proposed Pike Creek Bridge
Location: Relocated Hwy 39 Nr Windsor
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 578.5 Geo. contour
Field Supervisor: H.P. Prep.: P.M.
Driller: A.B. Checked:

LEGEND
Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing

Sampling Method

2" Dia. split tube

2" Shelby tube

[illegible]

Dominion Soil Investigation Ltd.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 6

Date: 26/11/58

LEGEND

Consistency

Natural moisture and

Liquidity Index (L)

liquid limit

Please Find

— 2 —

26. 100%

—

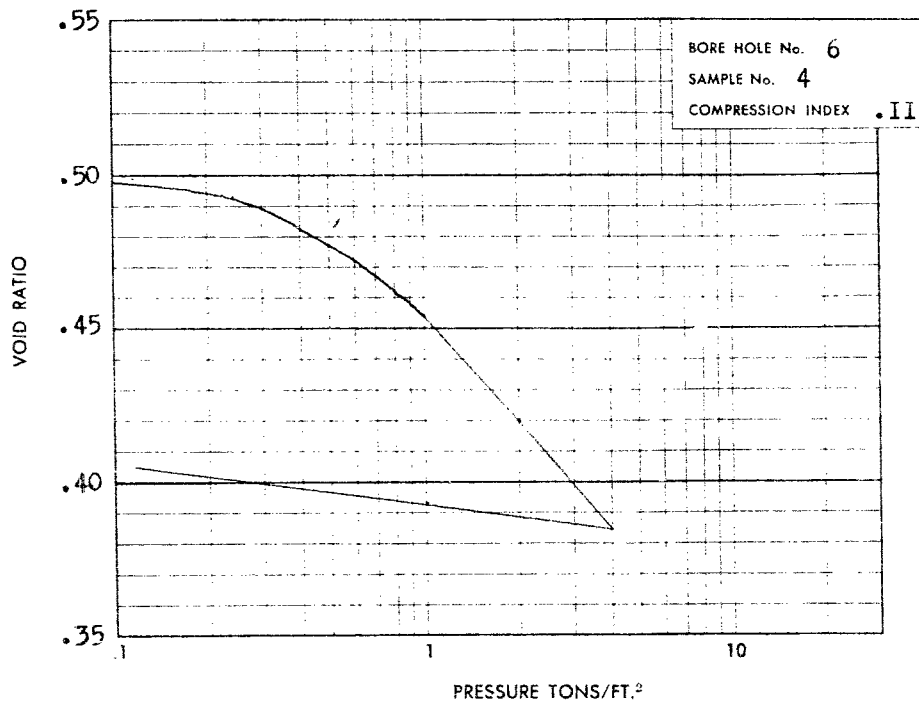
7

Sampling Method

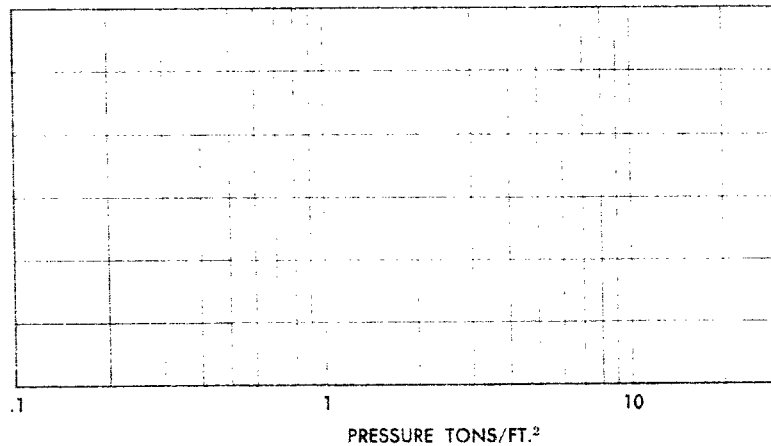
2" Dia. split tube

2" Shelby tube

DEPTH FEET	CONSISTENCY		SAMPLE	NATURAL UNIT WT. P.C.F.	REMARKS	
	MOISTURE CONTENT IN DRY WEIGHT					
70	10	20	30	40	50	
		1	0			SSI5
		1	0			TWI6
		1	0			SSI7
80		10				SSI8
90						

Dominion Soil Investigation Ltd.**CONSOLIDATION TEST**

COEFFICIENT OF CONSOLIDATION
FT.²/DAY



Dominion Soil Investigation Ltd.

CONSOLIDATION TEST

