

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

Attention: Mr. S. McGeeble

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

April 29, 1963

FOUNDATION INVESTIGATION REPORT --
By William A. Trow & Associates, Ltd.
for De Leuw, Cather of Canada, Ltd.
West Approaches to Hwy. 401 - 400
Interchange, District 6, Toronto, Ont.

Attached, we are forwarding you a copy of
the above-mentioned report for your use.

AGS/HMF
Attach. (1)

A. G. Sternac
A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. H. A. Tregaskes
H. D. McMillan
G. K. Hunter (2)
C. Fraser
T. J. Kovich

Foundations Office
Gen. Files ✓

W.P. = 232-50
63-216
Date
Site
Bit walls

23-63-216.

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

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

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

Project: J 1067

April 19, 1963.

De Leuw Cather of Canada Ltd.,
1491 Yonge St.,
Toronto 7, Ont.

Attention: Mr. H. Van Bodegom, P.Eng.

FOUNDATION CONDITIONS
WEST APPROACHES TO HWY. 401 - 400 INTERCHANGE

Dear Sirs:

63-F-220C

Enclosed herewith is our report on the foundation conditions existing under the sections of the Hwy. 401 development, west of the Hwy. 400 interchange.

The subsoil conditions encountered in this supplementary field program essentially confirmed the results of borings made in a preliminary study, the subject of our report of October 12, 1962. A stratum of very stiff to hard desiccated lake clay comprises the first ten to twelve feet of depth and this is underlain by a very deep deposit of less stiff clay till.

Calculations made for the multi-span Eastbound Collector Bridge over the West North Ramp showed that no differential settlement of consequence will occur in this structure. The recommended safe net bearing value for piers and abutments of the bridge, with the exception

of pier D, was given as 5000 p.s.f. Since two separate and smaller footings will be used for pier D, which spans the W. N. ramp, the use of a higher bearing pressure of 5500 p.s.f. was suggested, in order to obtain more uniform settlements along the length of the structure.

The recommended maximum bearing stress to use in the design of retaining wall footings is 6000 p.s.f. It was suggested that these structures be designed to resist the "at rest" earth pressure condition since even very small movements at the top of the wall may cause cracking of the asphalted road shoulders. Even though poorly drained ground exists adjacent to some sections of the retaining wall system east of Wendell ave., and north of Hwy. 401, competent soil exists at the shallow depths required for footing installation.

If you have any queries after you have reviewed the contents of this report, we shall be pleased to discuss them with you.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

WAT/lt
Encl.

WILLIAM A. TROW AND ASSOCIATES LTD.

DE LEUW CATHER OF CANADA LTD.
1491 YONGE ST., TORONTO, ONT.

FOUNDATION CONDITIONS
WEST APPROACHES TO HWY. 401 - 400 INTERCHANGE

Project: J 1067

William A. Trow & Associates Ltd.

April 19, 1963.

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FOUNDATION CONDITIONS
WEST APPROACHES TO HWY. 401 - 400 INTERCHANGE

This report contains the factual results and foundation recommendations associated with the subsoil investigation for the above-noted phase of the Hwy. 401 - 400 interchange system.

The specific purpose of this study was to determine the competence of the soil, just below frost level, for the support of footings of the various retaining walls incorporated into this approach to the interchange; and to appraise the foundation conditions at slightly greater depths, for the East-bound Collector over the West - North Ramp.

The examination of subsoil conditions comprised a total of 29 borings taken to depths ranging from 20 to 30 feet below the surface. This program supplements the investigation, performed in September 1962, during the early stages of planning for this project.

SITE AND GEOLOGY

The description of the land surrounding this highway development has been given in an earlier report on the subject. The Hwy. 400 - 401 interchange, itself lies in an area of relatively poorly-drained ground near the west limit of the Black Creek drainage system. The flat land at one time formed the bed of a very shallow lagoon extension of glacial Lake Iroquois.

A few hundred feet to the west of Hwy. 400, the ground begins to rise very gradually to a broad ridge of glacial drift which is located in the vicinity of the C.P.R. main line and is about 30 feet higher than the ground surface near the interchange. During the

construction of Hwy. 401, broad ditches were excavated about 4 feet below the ground surface east of Wendell Avenue and considerable soil was removed for borrow purposes along the south side of Hwy. 401 immediately east of the Canadian Pacific Railway. Low swampy ground exists north of Hwy. 401 for a few hundred feet east of Wendell Avenue and near the easterly sections of the W-N ramp. Since some of the borings were made in these areas, the existence of swampy conditions or of previously excavated land is of interest in this project.

SUBSOIL AND LABORATORY WORK

Although considerable mixing of the glacial deposits and of the Lake Iroquois sediments is evident in all borings, it is possible, in very broad terms, to describe the subsoil profile in terms of two distinct soil types, as is indicated in Dwg. 1. The upper levels consist, generally, of laminated or layered lake clay sediments which have been desiccated to a very stiff to hard condition. The thickness of this deposit, very approximately, is in the order of 10 feet.

This clay is underlain by very lean silty clay till which is frequently laminated with silt layers or with silt and fine sand pockets. In several of the borings the strength of this till decreased very abruptly at shallow depth below the clay surface. However, this decrease in strength was quite variable in a lateral direction and closely adjacent holes were found to have quite different strength versus depth profiles.

The results of strength tests in this material had to be viewed with some reservation because of these variations. In general, when field vane tests seemed somewhat high, it was felt that the strength result was affected by gravel particles or silt and sand pockets in the till. Where low laboratory compression test results were obtained, the

underestimation of strength was attributed to disturbance during sampling and handling of the silty materials. The strength versus depth profile shown on Dwg. 1A endeavours to account for these variables.

Because the soil contains numerous silt and fine sand layers or partings within the laminated soil, it is expected that horizontal drainage and, hence consolidation, will occur quickly under areas of isolated concentrated loading. In order to examine this possibility, two consolidation tests were performed on a sample of soft laminated till from Hole 32. In one test the laminations were parallel to the bearing surfaces in the consolidometer, and in the other, the specimen was prepared with the laminations at right angles to these surfaces. The results of this test are given in Dwg. 42a. Unfortunately, it was not possible to obtain an exact duplication of the silt partings or layers, and therefore the rate of drainage, - as expressed by the coefficient of consolidation, - in the inverted specimen, was not significantly greater than for the conventional test. However, it is believed that this lack of difference results in part from model effects and that a more marked difference in drainage rates will occur in the field where the proportion of clay to silt, in a vertical profile, is much greater.

Accordingly, because of horizontal consolidation, it is believed that some increase in strength of the clay can be expected during construction as the weight of the piers and footings is applied. An indication of the order of this increase in strength with consolidation pressure is shown in Dwgs. 40 and 41 for tests on samples of soft clay till from hole 32. The main reason for performing these tests was to determine if the reconsolidation of the samples, under pressures approaching the existing overburden pressure, would produce a more representative measure of in-situ shear strength than was obtained from simple undrained compression tests. As indicated in a previous paragraph, those laboratory specimens which contained the silt pockets and intrusions, were very

sensitive to disturbance and therefore the test results probably err on the low side. On the other hand, the strengths of the consolidated samples are more in accord with the field vane test data, and it is believed, therefore, that some reconsolidation of this material is necessary in order to obtain reliable indications of in-situ strength.

No Atterberg limit measurements were performed in this recent investigation program, since the plasticity characteristics were well documented in the previous investigation of September, 1962. However, numerous moisture content measurements were made. The main purpose of this testing was to demonstrate the variability in subsoil type. Low moisture contents generally were indicative of the lean till, while higher measurements reflected the presence of fatter clay. This variation in subsoil composition is demonstrated by the moisture content profile for samples from holes 6 and 32 shown in Dwg. 43.

FOUNDATIONS

Bridge Structure The foundation requirements for the east bound collector overpass of the West - North Ramp were considered in a preliminary report dated April 9, 1963. In this report, the results of settlement calculations for the piers and abutments of this multi-span bridge were presented. These estimates were based upon approximate indications of pier loadings and minimum permissible foundation depths submitted to us at an early stage of this study. The results of the computations are shown on Dwg. 1A, and the various calculations and assumptions used are presented in Dwgs. 44 to 47. Since this information is self-explanatory, no purpose is served by a detailed discussion of this problem.

It was found that the differential settlement between the various piers and abutments of the bridge will be negligible under a bearing stress of 4500 psf. Even if the compressibility of the subsoil differs somewhat from the values assumed in the calculations, the differential settlement

pattern should remain essentially unchanged. Because of this fact, and in order to obtain some reduction in the size of the very large pier footings, it was suggested that the net bearing stress be increased to 5000 psf. It was also recommended that the pressure used in the design of the split footing at the pier D location be increased to 5500 psf, in order to produce settlements of these smaller footings more equal in magnitude to the movements of the other piers. Although this increase in bearing stress will induce higher shear stresses into the clay, Dwg. 1A indicates that very little overstressing of the soil will occur. Under a pressure of 5000 psf, the maximum shear stress just about touches the average shear strength line for the soil. As the clay consolidates, the ratio of shear strength to shear stress will increase and the danger of overstressing will diminish. It should be appreciated that less settlement of the piers will occur if they are placed higher up on the stiff crust or if the footing size is reduced.

In view of the heavy loading on the bridge abutments, it is anticipated that there will be ample resistance to the horizontal force exerted by the fill approaches to the bridge. Since the abutments will be rigid, the earth force exerted against them will be the "at rest" pressure. For compacted granular backfill, the coefficient of earth pressure at rest will be approximately equal to 0.5. This will be the maximum earth pressure force acting against the abutments. If any tendency for movement takes place, the pressure will reduce to the active condition.

The short term resistance to movement will be provided by the passive resistance generated by the hard clay in front of the toe of the footing and by the undrained shear strength of the clay acting along the base. From the very long term view, the soil will behave as a granular material, and its resistance to movement in this hard clay therefore will be lower than the short term strength. The resistance per lineal foot generated for this long term case will be given conservatively by the expression:

$$R = N \tan \phi + K_p \frac{1}{2} \gamma h^2$$

where	N	is the weight acting on the abutment per foot of length, $\tan \phi$ for this clay is estimated to be equal to 0.5
	$K_p = 4$	is the estimated passive earth pressure coefficient
	$\gamma = 70$	is the submerged weight of the clay, assuming saturated spring time conditions
	n	is the depth of the footing below the surface.

When equating the horizontal force to the resistance available in the above expression, a factor of safety equal to unity should suffice, because of the very conservative assumptions made. As indicated above, any tendency for movement will, in itself, cause the earth pressure to reduce from the "at rest" to the "active" case. In addition, and particularly at the west abutment, the horizontal force will be less at the north and south limits of the fill. The assumption of a completely granular state also is quite severe. Some residual cohesion certainly will be available in this heavily overconsolidated material.

As indicated in the report of October 12, 1962, no embankment stability problem exists at this site. The strength of the clay is more than adequate to support the weight of 17 to 19 feet of fill comprising these approaches.

Retaining Walls The retaining walls supporting both the fill for the east bound collector road and the general fill along the north and south boundary of the highway system, as far west as the Canadian Pacific Railway, can be supported much higher up on the hard clay crust. Therefore a much higher bearing pressure can be utilized, particularly since the maximum footing pressure will apply only at the toe of the abutment footings.

In general, the permissible bearing level will be 4 feet, or just below the level of frost penetration. The only variation from this recommendation is at the north east corner of Wendell Avenue, where a localized fill about 6 to 7 feet was noted.

According to undrained tests made on samples from depths of 5 to 7 feet in representative borings along the route, the shear strength of the clay is in the order of 3000 psf. Although the safe net bearing value for a strip footing under a building would be 6000 psf for a strength of this magnitude, a higher pressure should be permissible for a retaining wall, since the maximum pressure will apply only at the toe. However, it is also extremely desirable to prevent any movement at the top of the wall, since the asphalt road surface will be carried right to this boundary in most locations. Because of this requirement, it is suggested that the maximum net bearing pressure at the toes of the retaining walls be limited to 6000 psf.

Since the prevention of movement at the tops of the retaining walls is so important, it is suggested that they be designed to resist the at rest pressure condition, as outlined for the overpass structure. The same short and long term resistances to sliding also will apply for the retaining walls. Here, again, a low factor of safety can be used in the design, since these assumptions of horizontal forces and resistances are quite conservative.

Wendell Avenue Bridge The foundation requirements for the additions to this bridge were considered in the report of October 12, 1962. In this report a net bearing stress of 4000 psf was recommended for design of abutment footings. It was noted that long term settlements, ranging from 1 to $3\frac{1}{4}$ inches, would occur across the length of the north addition and that somewhat less settlement would take place under the south addition. Since the width of the embankment fill is somewhat greater

than was assumed in that initial investigation, it is probable that somewhat more settlement can occur under the north limits of the bridge extension. It was suggested, however, in the October 1962 report, that the actual movements should be somewhat less than theoretical computations indicate. Since the additions to the bridge will act as separate units, this slight tilting and differential movement, with respect to the existing structure, should not be a matter of serious concern.

WAT/gc
J1067
Apr./63



W. Trow

William A. Trow, P.Eng.




SUMMARY OF STRENGTHS AND RESISTANCES ABOVE 12 FEET

(All depths from present ground surface)




South Side	Penetration Resistance			Shear Strength ksf
	Blows/ft. 350 ft.lbs. energy			
Hole	3 ft.	6 ft.	11 ft.	
34	28	33	36	
33	29	41	15	
32	22	27	20	2.75 @ 8½ ft. 2.16 @ 9½ ft.; 1.45 @ 11 ft.
4	17	30	18	4.6 @ 11 ft.
3	15	18	12	3.8 @ 7 ft., vane > 2.1 @ 9½ ft.; 1.0 @ 12½ ft.
6	27	32	15	3.3 @ 9 ft.; 2.65 @ 9½ ft.; 4.1 @ 11½ ft.
7	32	28	40	3.8 @ 6 ft.
8	24	29	24	
9	27	25	33	
10	19	44	33	
11	22	20	12	3.9 @ 6½ ft.
12	39	28	16	
13	15	17	13	3.7 @ 7 ft.
14	21	18	20	
15	20	57	20	
16	17	15	12	3.6 @ 7 ft.
17	30	39	18	
2	16	18		3.0 @ 7 ft.; 1.45 @ 10 ft.
18	15	18	26	
19	24	22	12	3.0 @ 6 ft.
20	30	24	14	
21	21	23	13	3.4 @ 6 ft.
22	18	7	12	Vane strength > 2.0 @ 7½ ft.
North Side				
1	38	32	18	2.7 @ 8 ft.
23	32	29	23	
24	29	31	11	3.05 @ 6 ft.
25	17	21	15	
26	12 (fill)	8 (fill)	20	2.7 @ 8 ft.
27	35	28	24	3.65 @ 6 ft.
28	21	23	16	
29	14	20	9	3.0 @ 6½ ft.
30	13	40	13	
31	21	21	30	

BOREHOLE No. 6
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 431.3 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

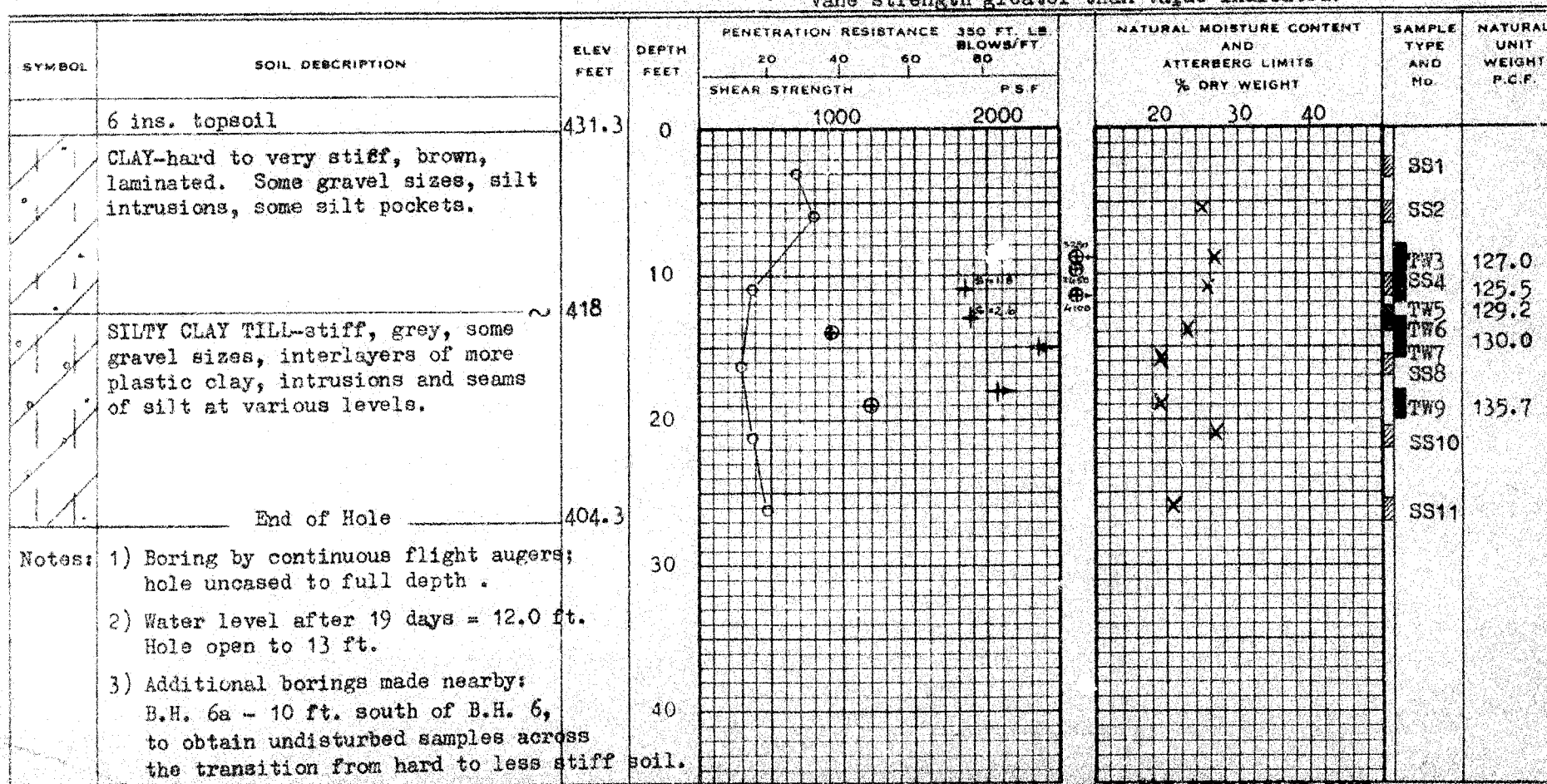
ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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

 Vane strength greater than value indicated.



B.H. 6b - 9 ft. north of B.H. 6, to obtain vane tests at close intervals across the transition zone.

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

LEGEND

DRAWING NO. 3
PROJECT NO. J106

BOREHOLE No. 7
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 432.4 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE —X—X—X—X—
2" DIA. CONE —————

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) +^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

X^{LI}

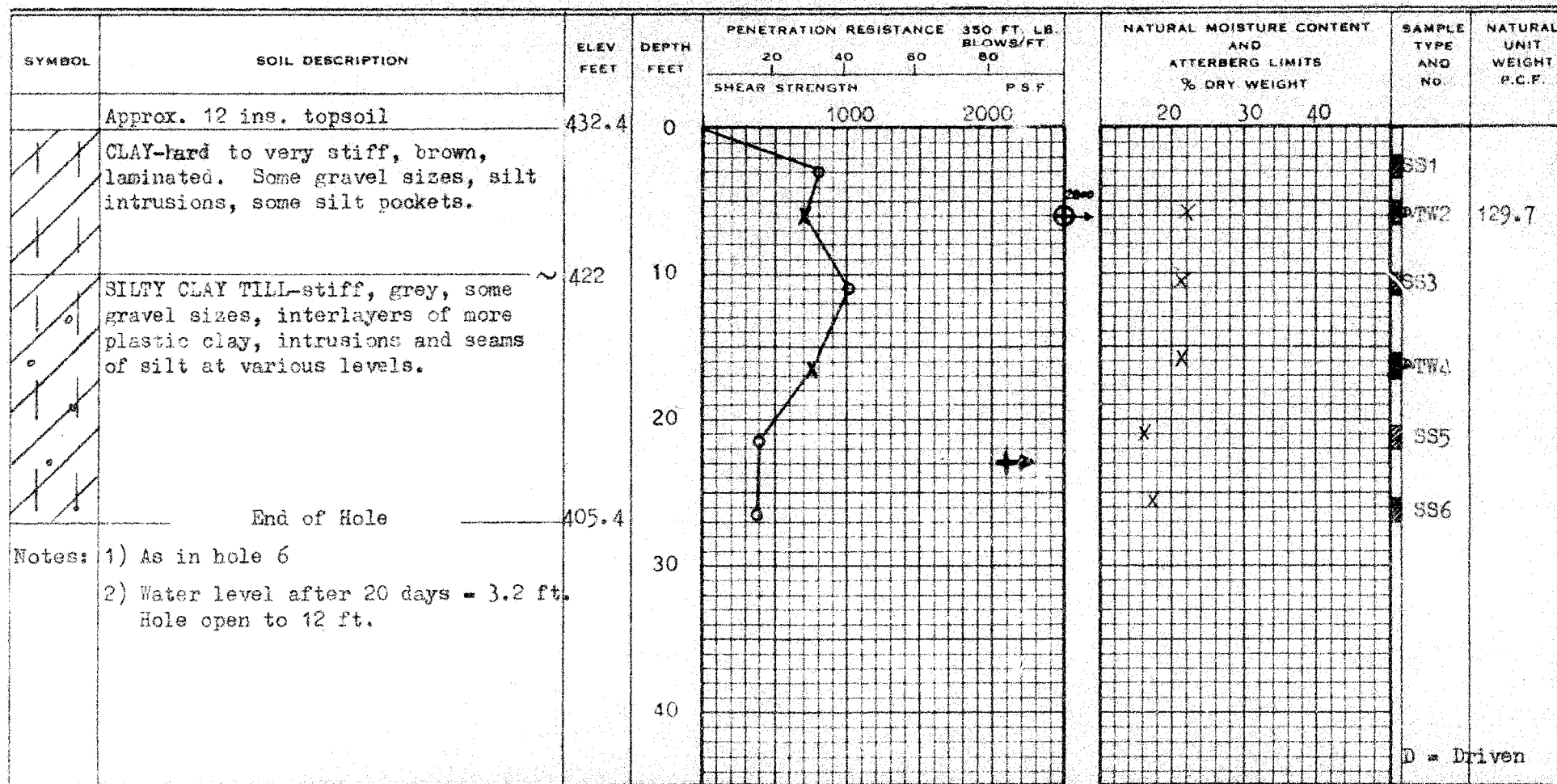
ATTERBERG LIMITS

LIQUID LIMIT —○—

PLASTIC LIMIT ———

SAMPLE TYPE

2" O.D. SPLIT TUBE ■
2" I.D. SHELBY TUBE ■
3" O.D. SHELBY TUBE ■



D = Driven

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

LEGEND




DRAWING No. 4
PROJECT No. J1067

BOREHOLE No. 8
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 433.4 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

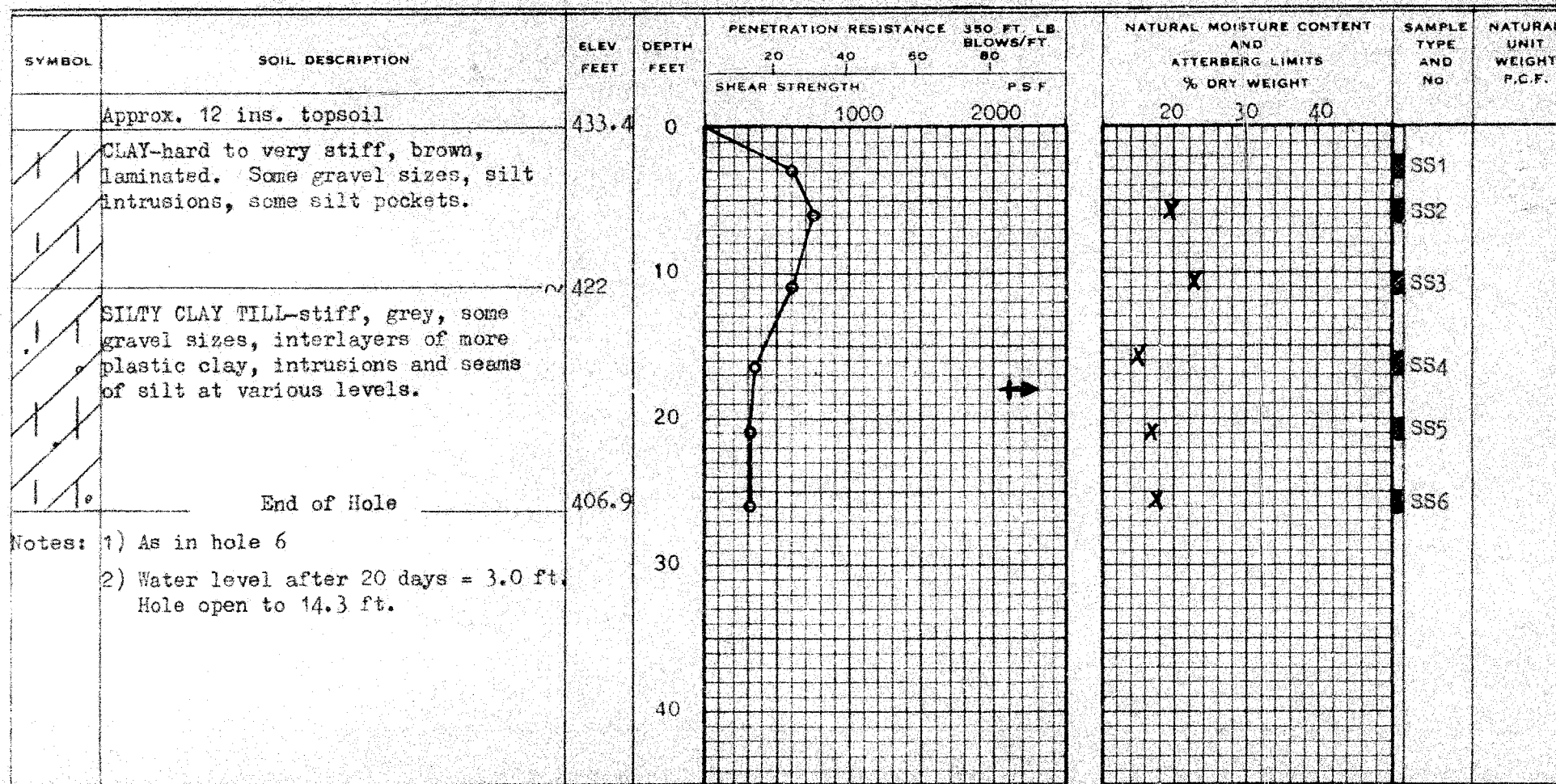
NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX 

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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



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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

LEGEND




DRAWING No. 5
PROJECT No. J1067

BOREHOLE No. 9
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 433.0 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S)  ^S

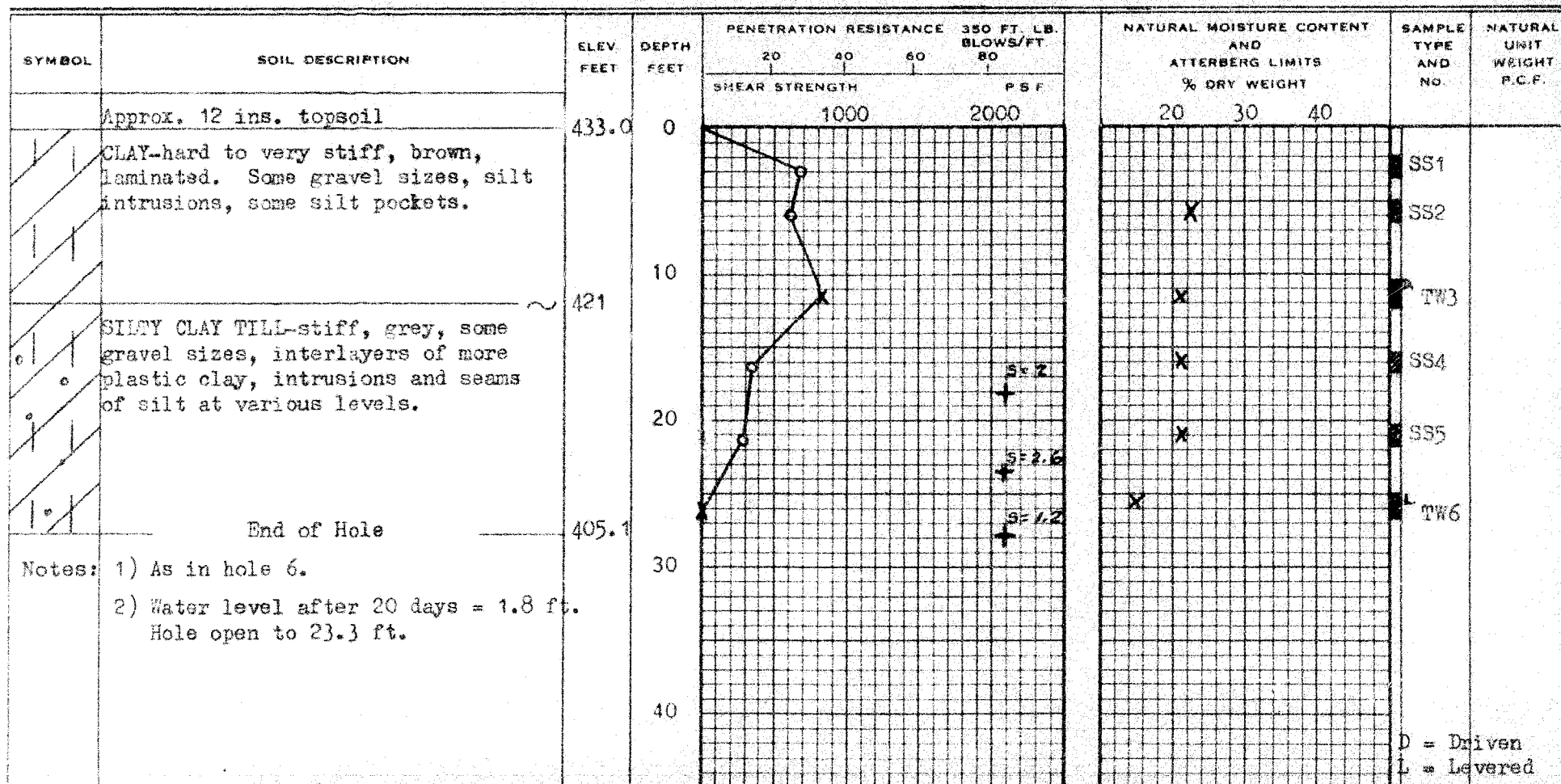
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX 

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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



D = Driven
L = Levered

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

LEGEND

DRAWING No. 6
PROJECT No. J1067

BOREHOLE No. 10
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 434.9 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

2" DIA. CONE

SHEAR STRENGTH

UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE

UNCONFINED COMPRESSION

VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT

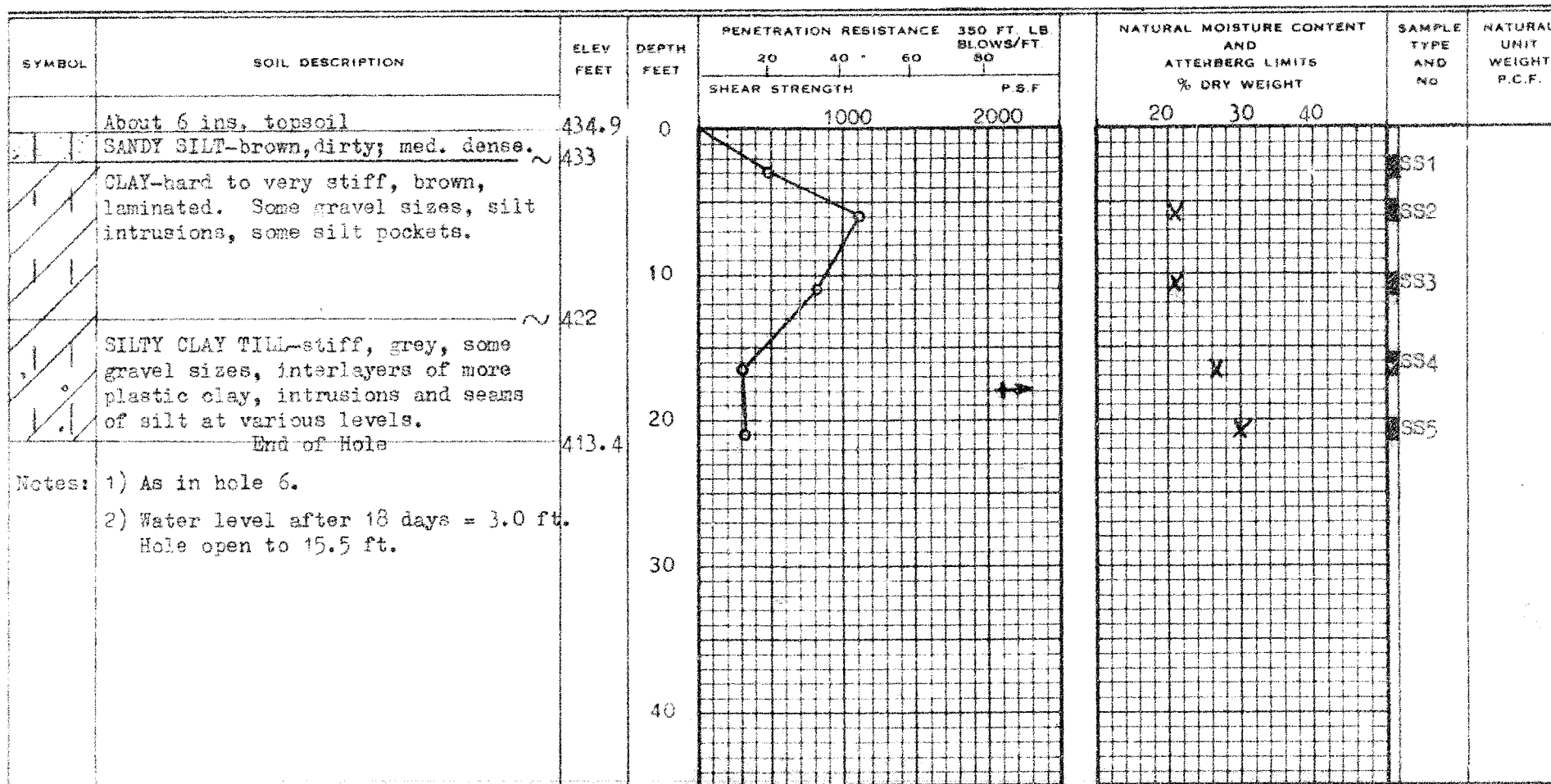
PLASTIC LIMIT

SAMPLE TYPE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

3" O.D. SHELBY TUBE



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SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

LEGEND




DRAWING No. 7
PROJECT No. J1067


BOREHOLE No. 11
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 436.7 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

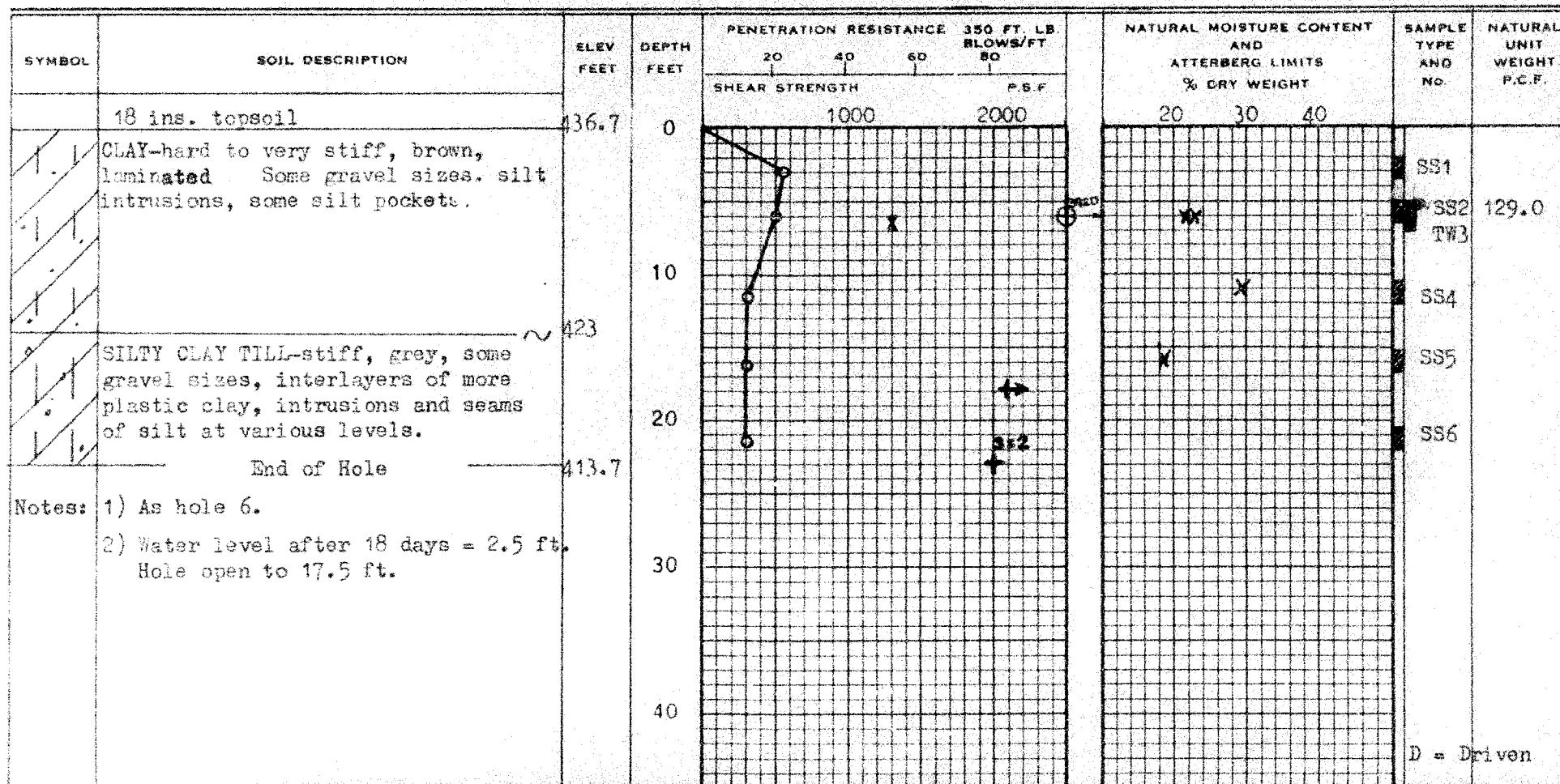
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX 

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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





LEGEND




BOREHOLE NO. 12PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. InterchangeHOLE LOCATION See Dwg. 1.HOLE ELEVATION 436.1 ft.

DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 2" I.D. SHELBY TUBE 2" DIA. CONE 


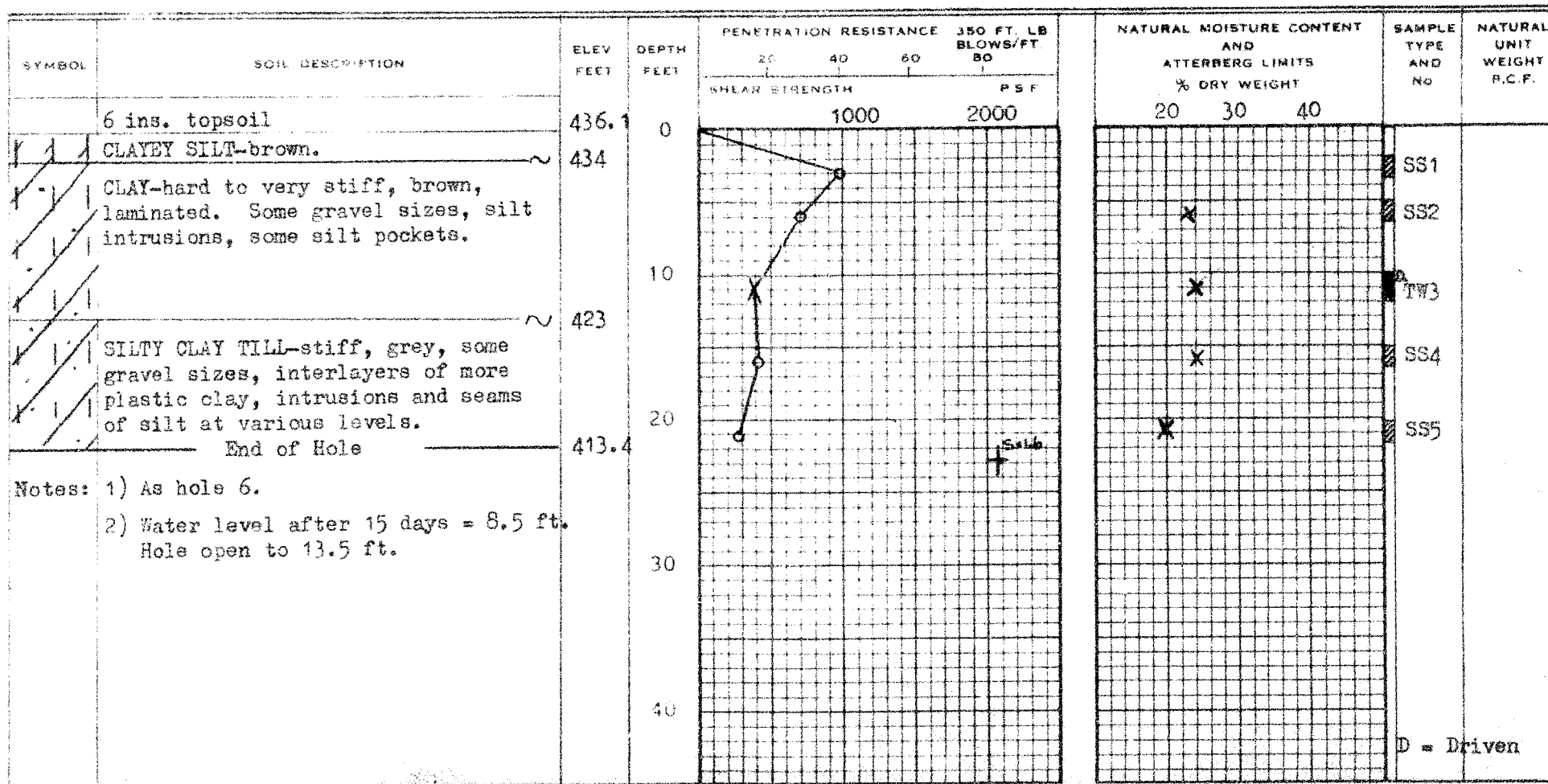

SHEAR STRENGTH

UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE UNCONFINED COMPRESSION VANE TEST AND SENSITIVITY (S) NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX 

ATTERBERG LIMITS

LIQUID LIMIT PLASTIC LIMIT 

SAMPLE TYPE

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

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 9
PROJECT No. J1067




LEGEND

BOREHOLE No. 13
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 436.8 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 


NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX 


ATTERBERG LIMITS


LIQUID LIMIT 

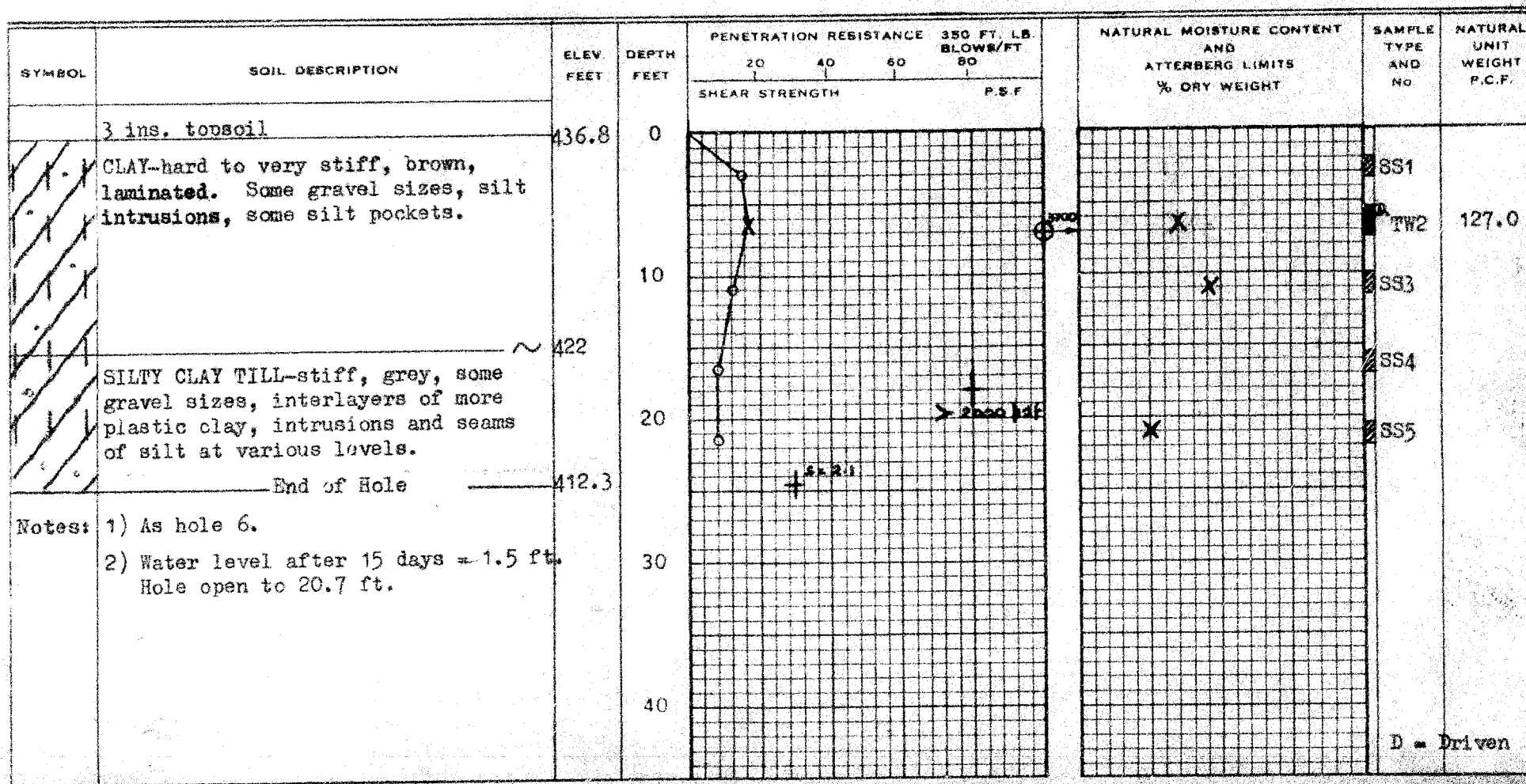
PLASTIC LIMIT 

SAMPLE TYPE

2" O.D. SPLIT TUBE 

2" I.D. SHELBY TUBE 

3" O.D. SHELBY TUBE 



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LEGEND




DRAWING NO. 10
PROJECT NO. J1067

BOREHOLE NO. 14
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 440.5 ft.
DATUM

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

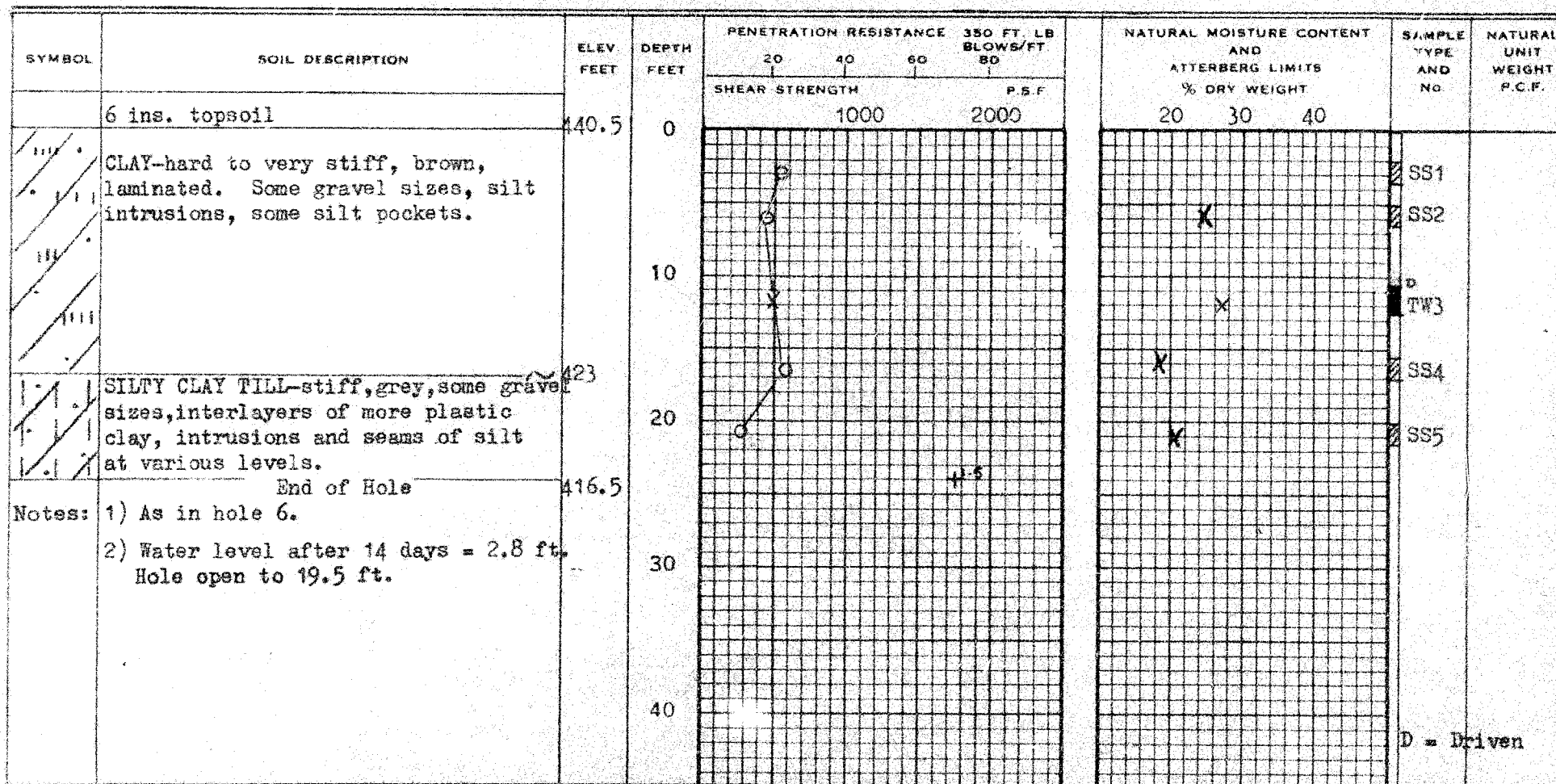
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX 

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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



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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 11
PROJECT No. J1067

LEGEND

BOREHOLE No. 15
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 438.5 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE ○ ○ ○
2" I.D. SHELBY TUBE * * * * *
2" DIA. CONE —————

SHEAR STRENGTH

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

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

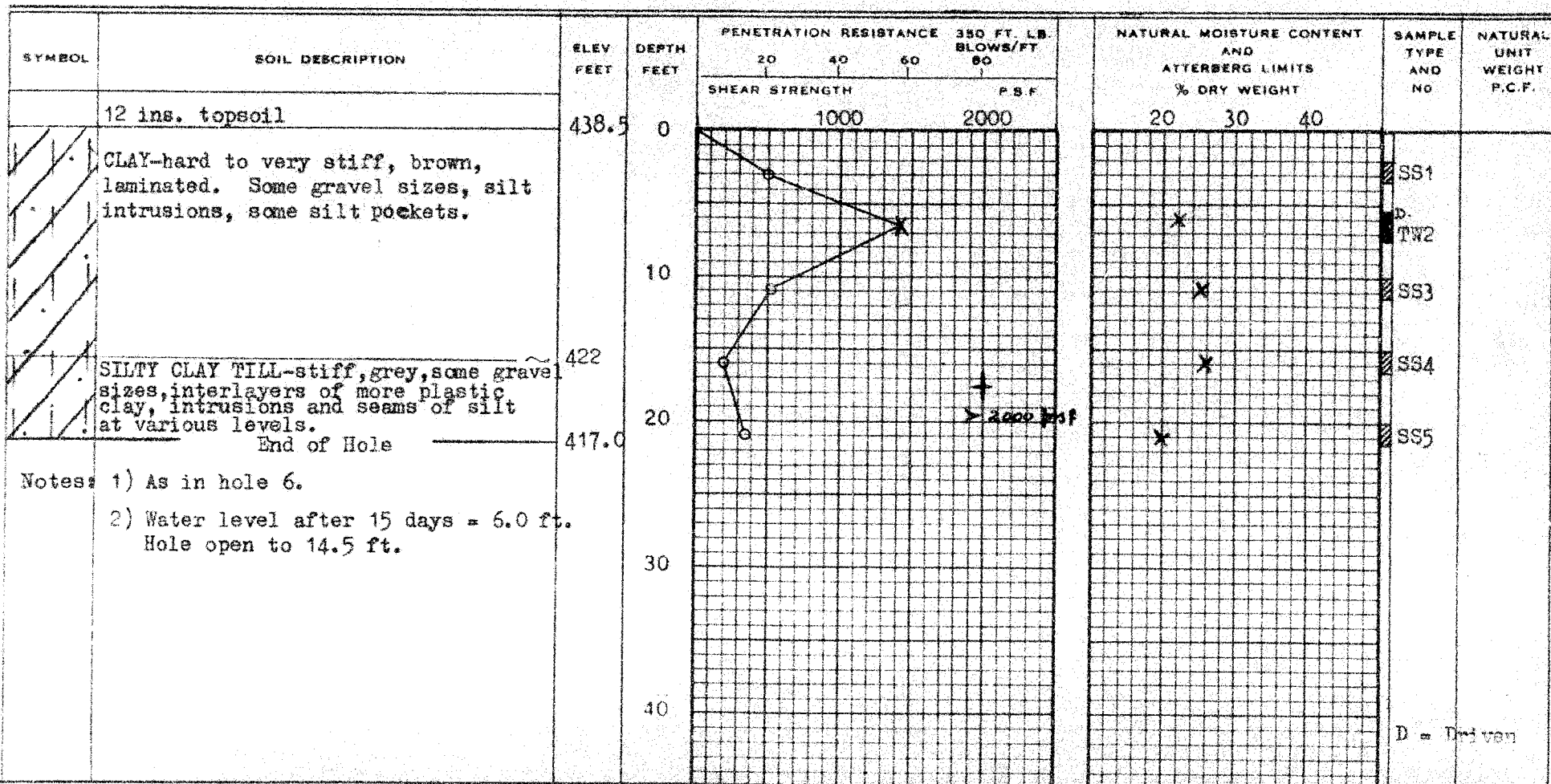
LI
X

ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT ———

SAMPLE TYPE

2" O.D. SPLIT TUBE ■
2" I.D. SHELBY TUBE ■
3" O.D. SHELBY TUBE ■



SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

DRAWING No. 12
PROJECT No. J1062

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE * * * * *
2" DIA. CONE —————

SHEAR STRENGTH

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

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

X^L

ATTERBERG LIMITS

LIQUID LIMIT _____
PLASTIC LIMIT _____

SAMPLE TYPE

2" O.D. SPLIT TUBE _____
2" I.D. SHELBY TUBE _____
3" O.D. SHELBY TUBE _____

DATUM _____

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	PENETRATION RESISTANCE		350 FT. LB. BLOWS/FT.	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS			SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.		
				20	40		60	80	% DRY WEIGHT				
				SHEAR STRENGTH				P.S.F.					
	6 ins. topsoil	439.1	0		1000	2000	20	30	40				
	CLAY-hard to very stiff, brown, laminated. Some gravel sizes, silt intrusions, some silt pockets.									SS1			
			10							TW2	127.0		
										SS3			
		424								SS4			
	SILTY CLAY TILL-stiff, grey, some gravel sizes, interlayers of more plastic clay, intrusions and seams of silt at various levels.		20							SS5			
	End of Hole	416.1											
Notes: 1) As in hole 6.													
2) Water level after 14 days, 3.0 ft. Hole open to 16.0 ft.													
			30										
			40										

D = Driven

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 13
PROJECT No. J1067




LEGEND

BOREHOLE No. 17
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 443.8 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH

UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 




NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX

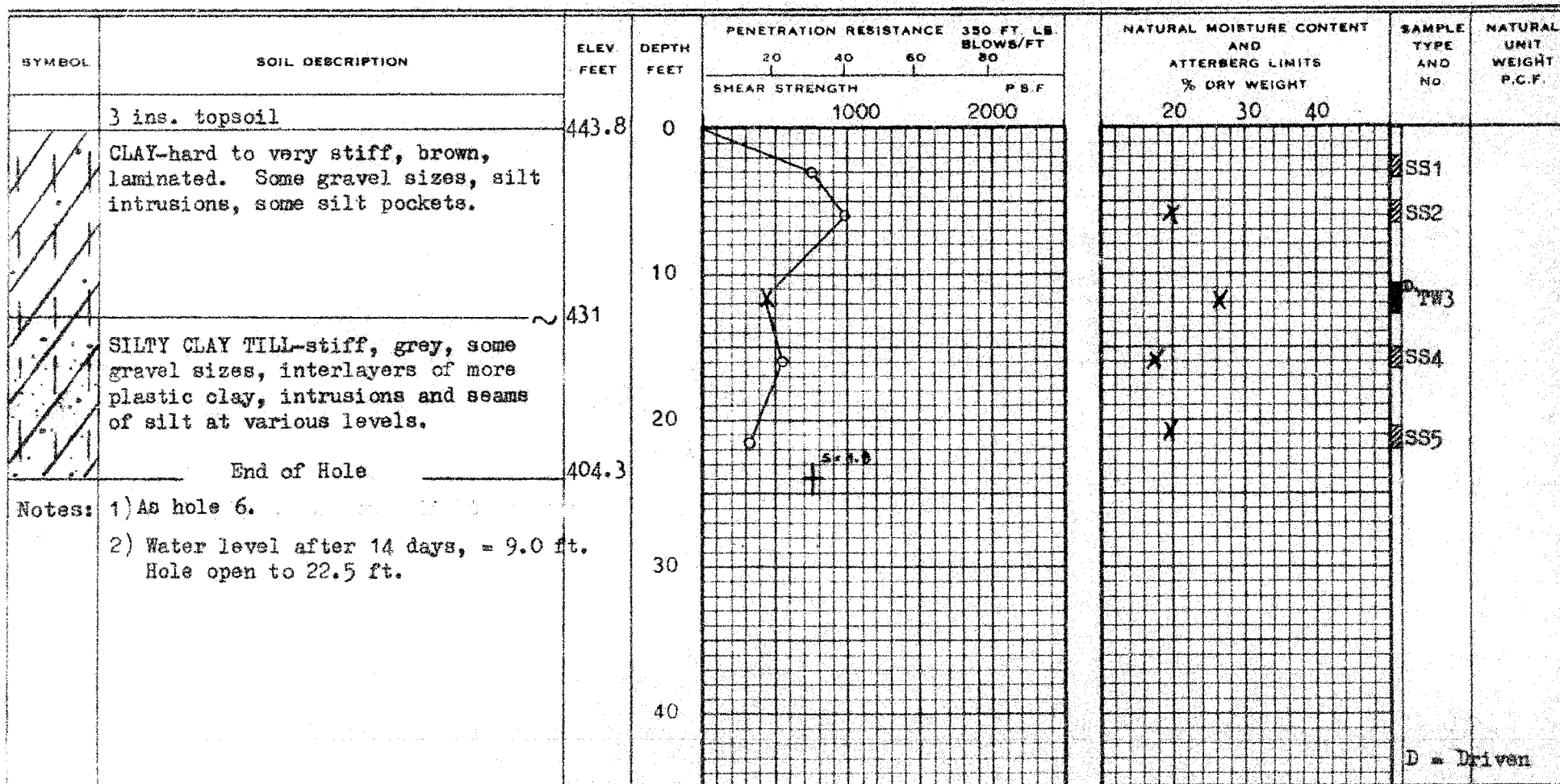
LI
X

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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



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SITE INVESTIGATIONS : SOIL MECHANICS CONSULTATION

DRAWING No. 14
PROJECT No. J1067




BOREHOLE No. 18
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 444.9 ft.
DATE


LEGEND

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

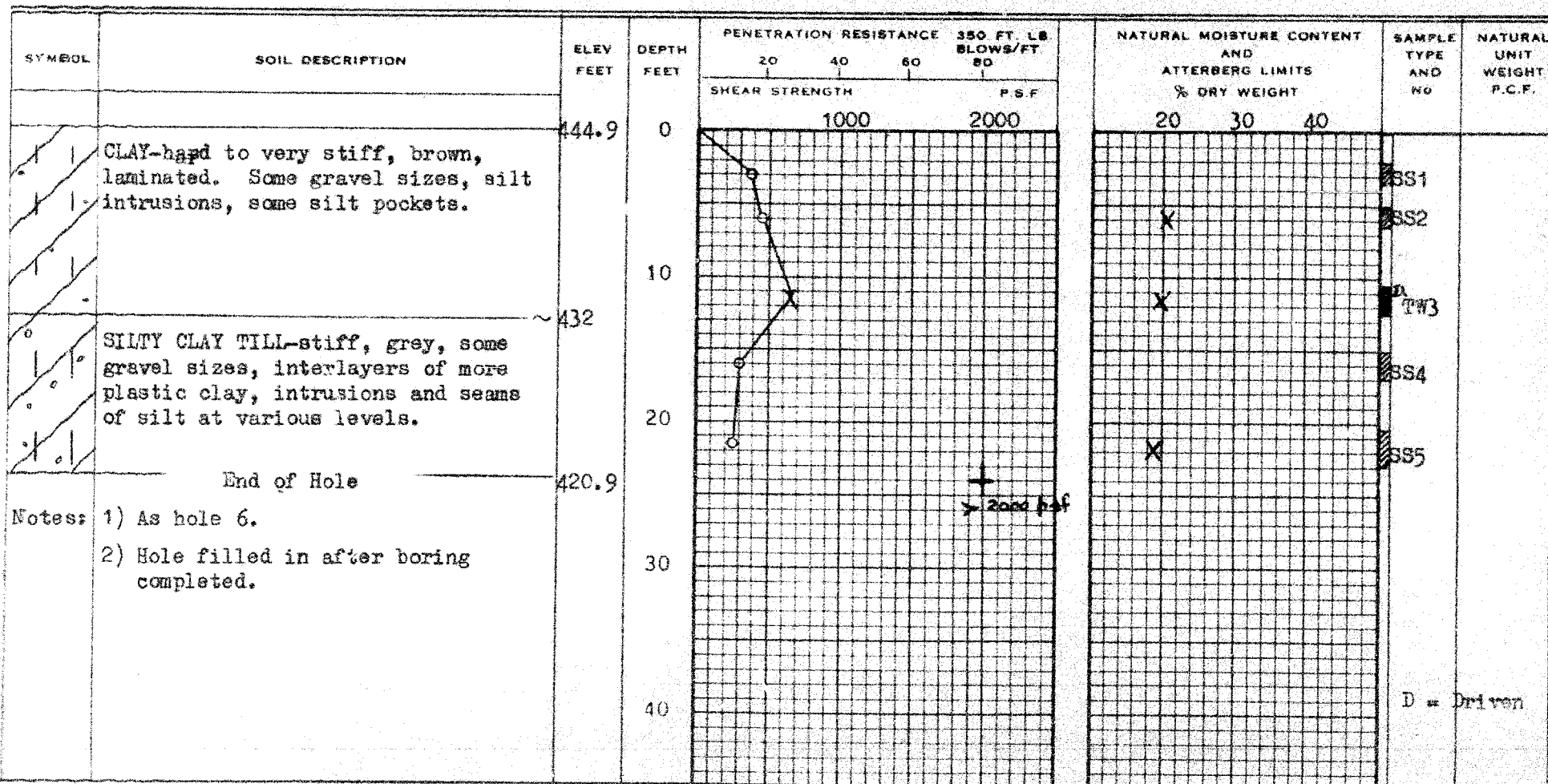
NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX 

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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



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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 15
PROJECT No. J1067

LEGEND

BOREHOLE No. 19
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 447.7 ft.
DATUM

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE ○ ○ ○
2" I.D. SHELBY TUBE * * * * *
2" DIA. CONE ————

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) †

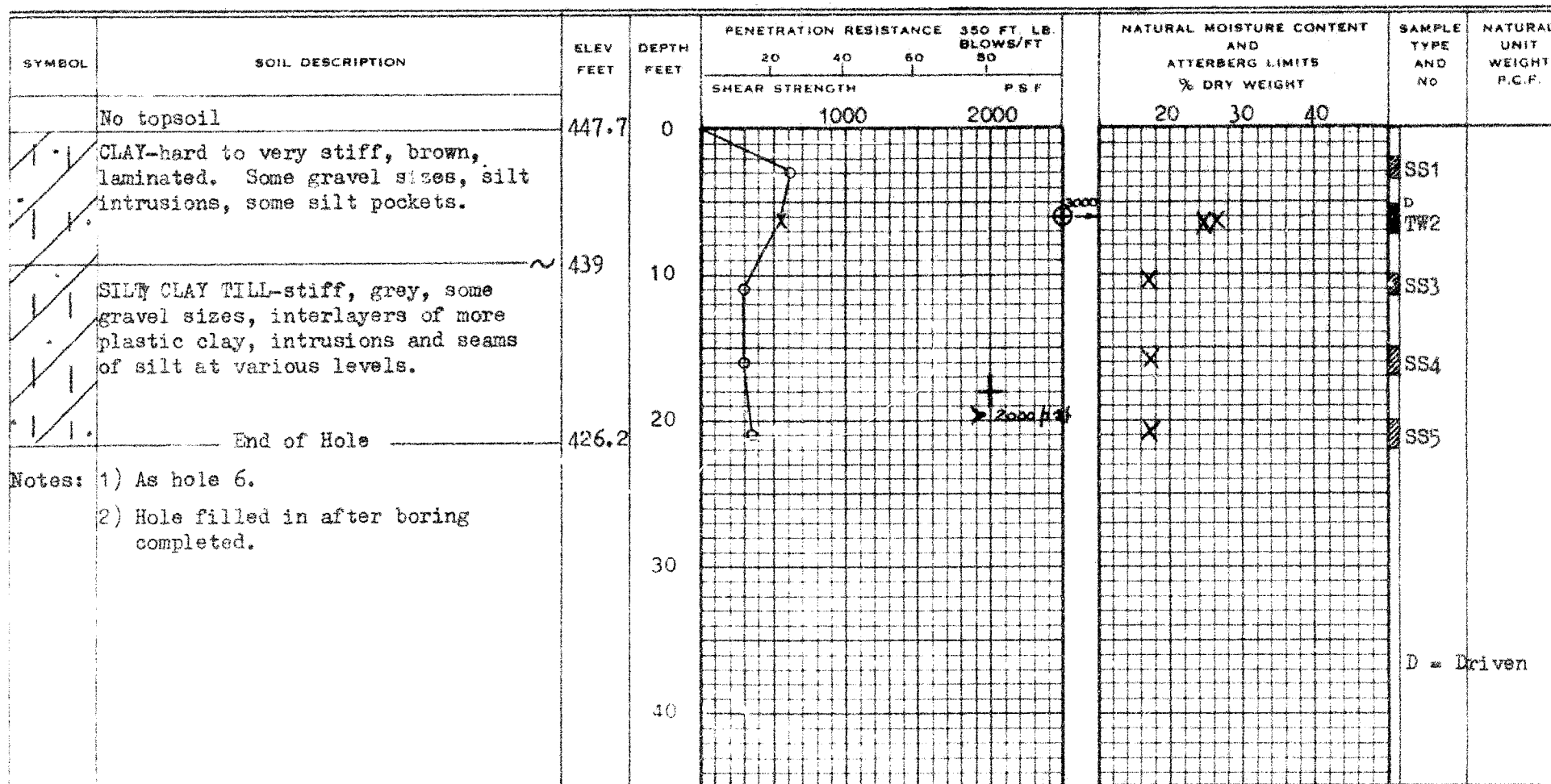
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT —|—

SAMPLE TYPE

2" O.D. SPLIT TUBE ⊠
2" I.D. SHELBY TUBE ⊡
3" O.D. SHELBY TUBE ⊢




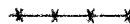

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION




DRAWING NO. 16
PROJECT NO. J1067

LEGEND

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX 

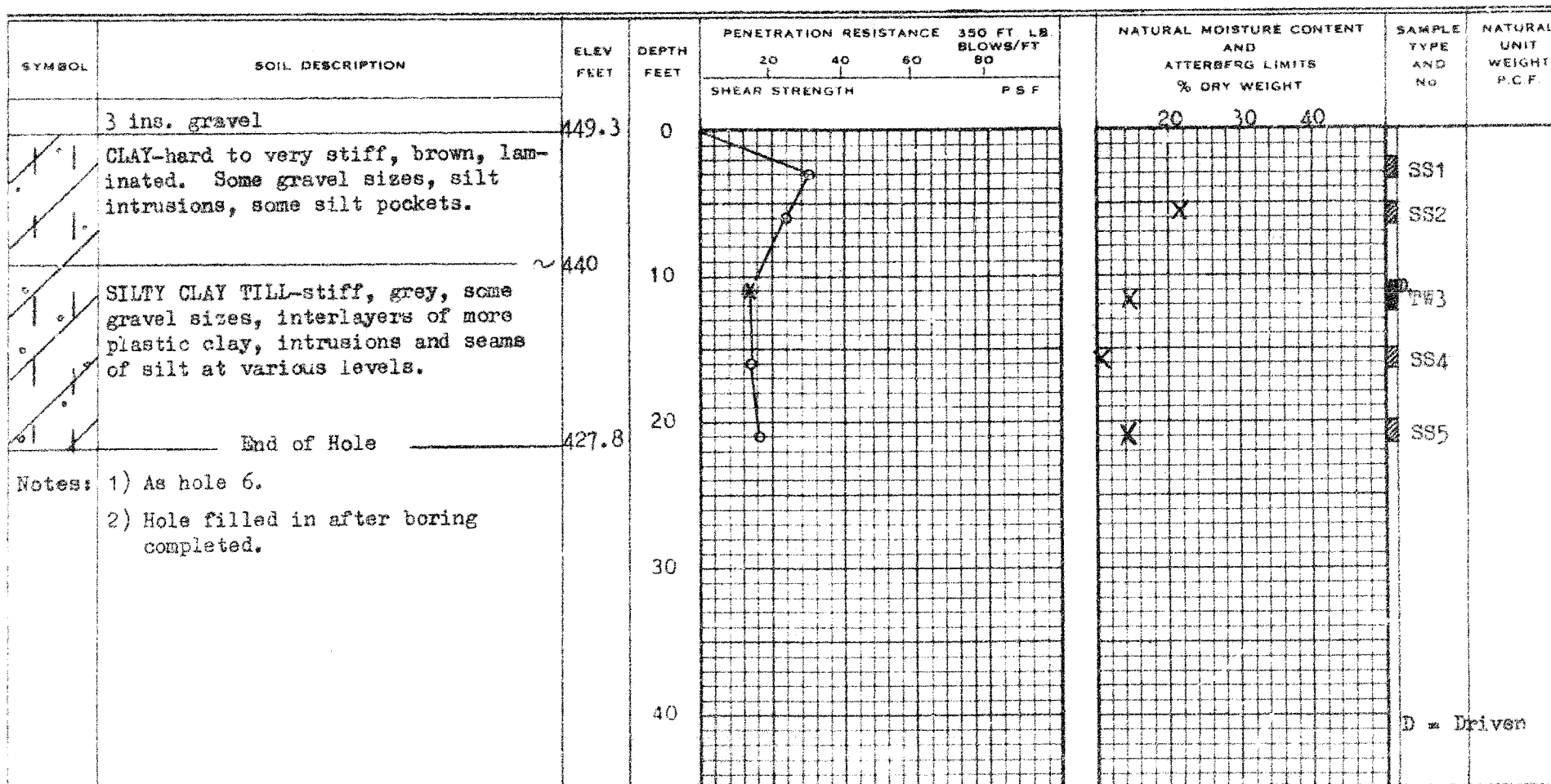
ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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

BOREHOLE NO. 20
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above.
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 449.3 ft.
DATUM



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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

LEGEND

DRAWING No. 17
PROJECT No. J1067

BOREHOLE No. 21
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 451.1 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE —×—×—×—
2" DIA. CONE ————

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) ⊕^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

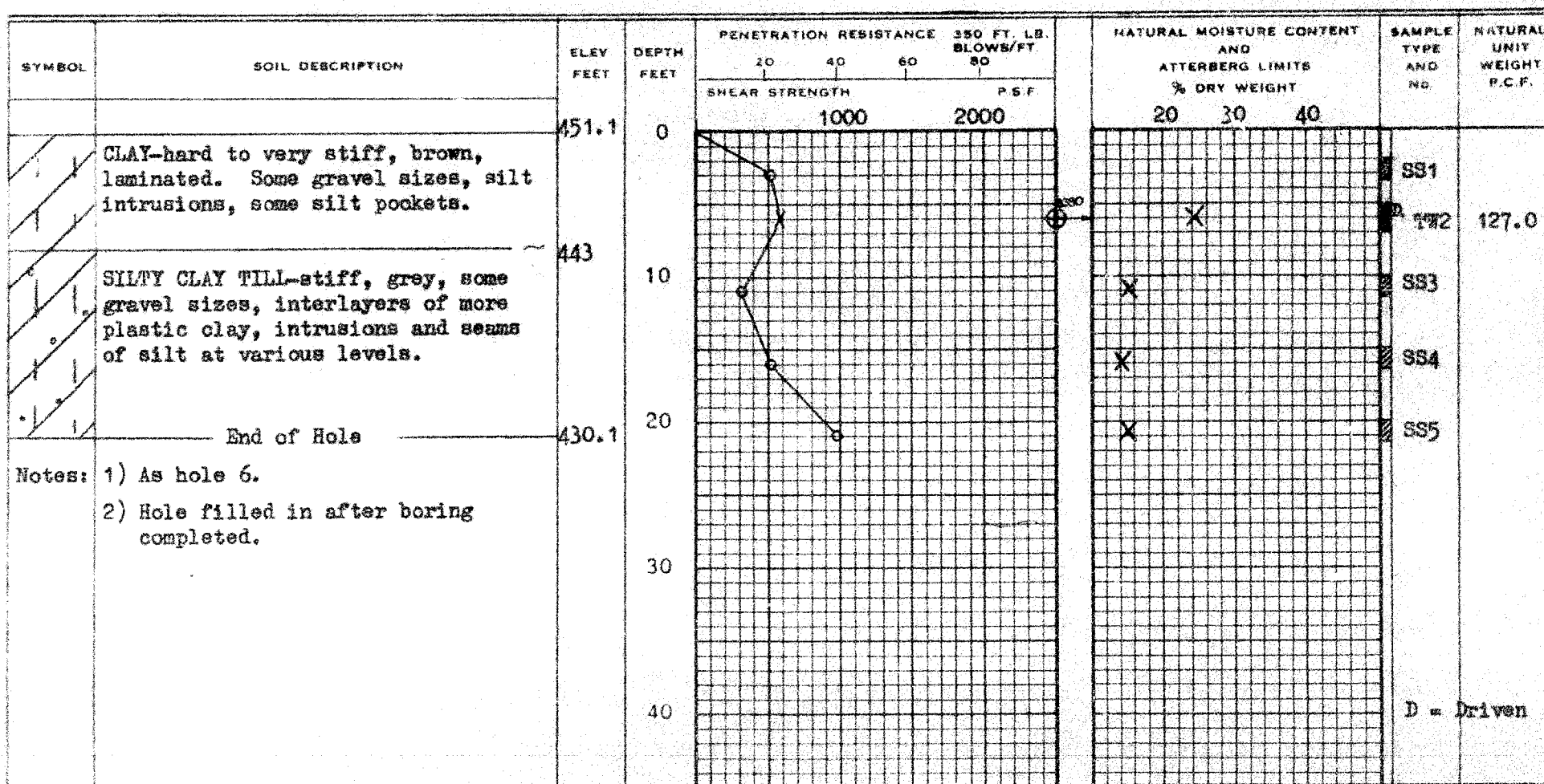
LI
X

ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT ———

SAMPLE TYPE

2" O.D. SPLIT TUBE ■
2" I.D. SHELBY TUBE ■
3" O.D. SHELBY TUBE ■






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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION




DRAWING NO. 18
PROJECT J1067


LEGEND

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX 

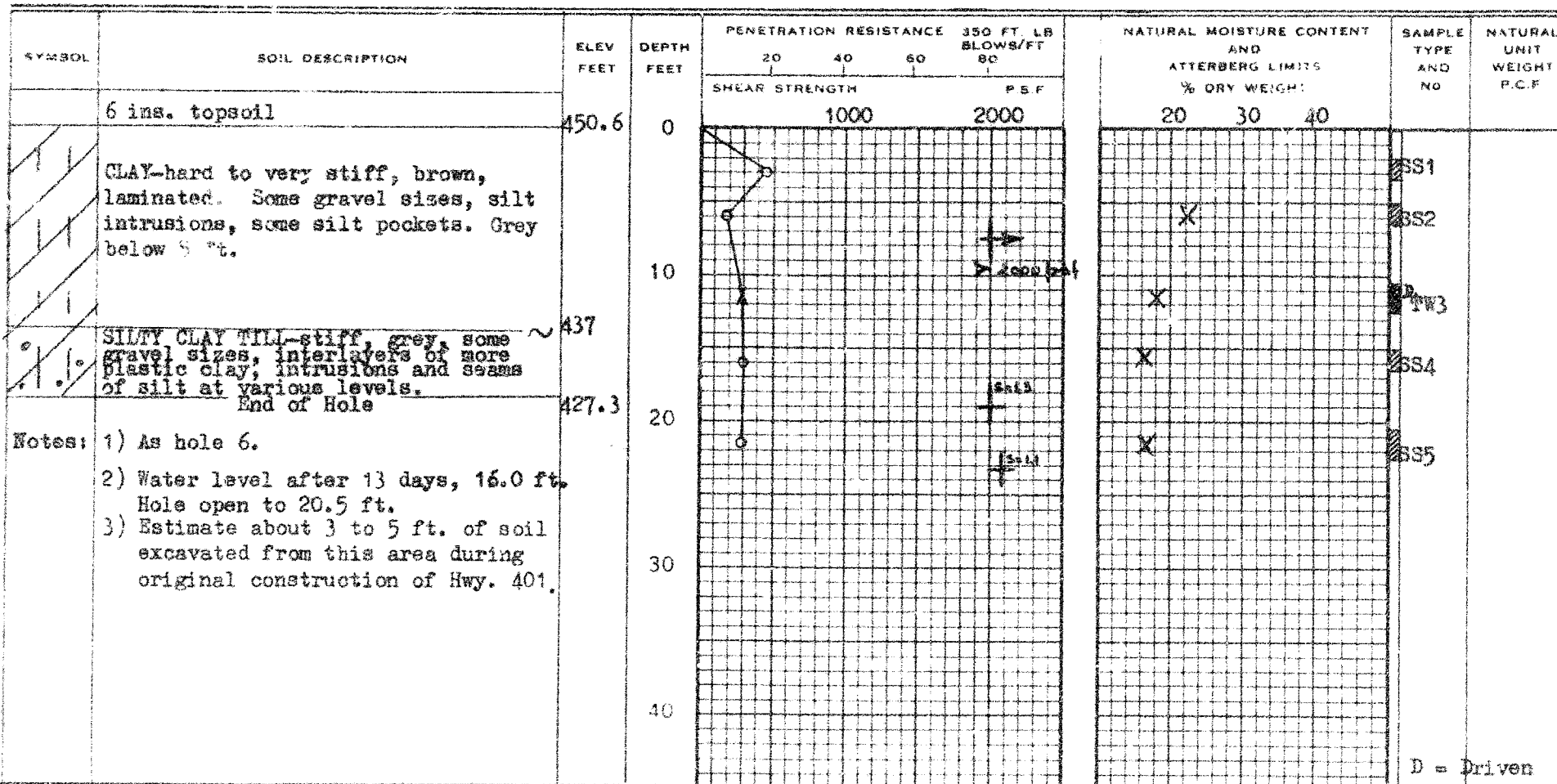
ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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

BOREHOLE NO. 22
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 450.6 ft.
DATUM



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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

LEGEND




DRAWING No. 19
PROJECT No. J1067


BOREHOLE No. 23
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 446.5 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL
AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

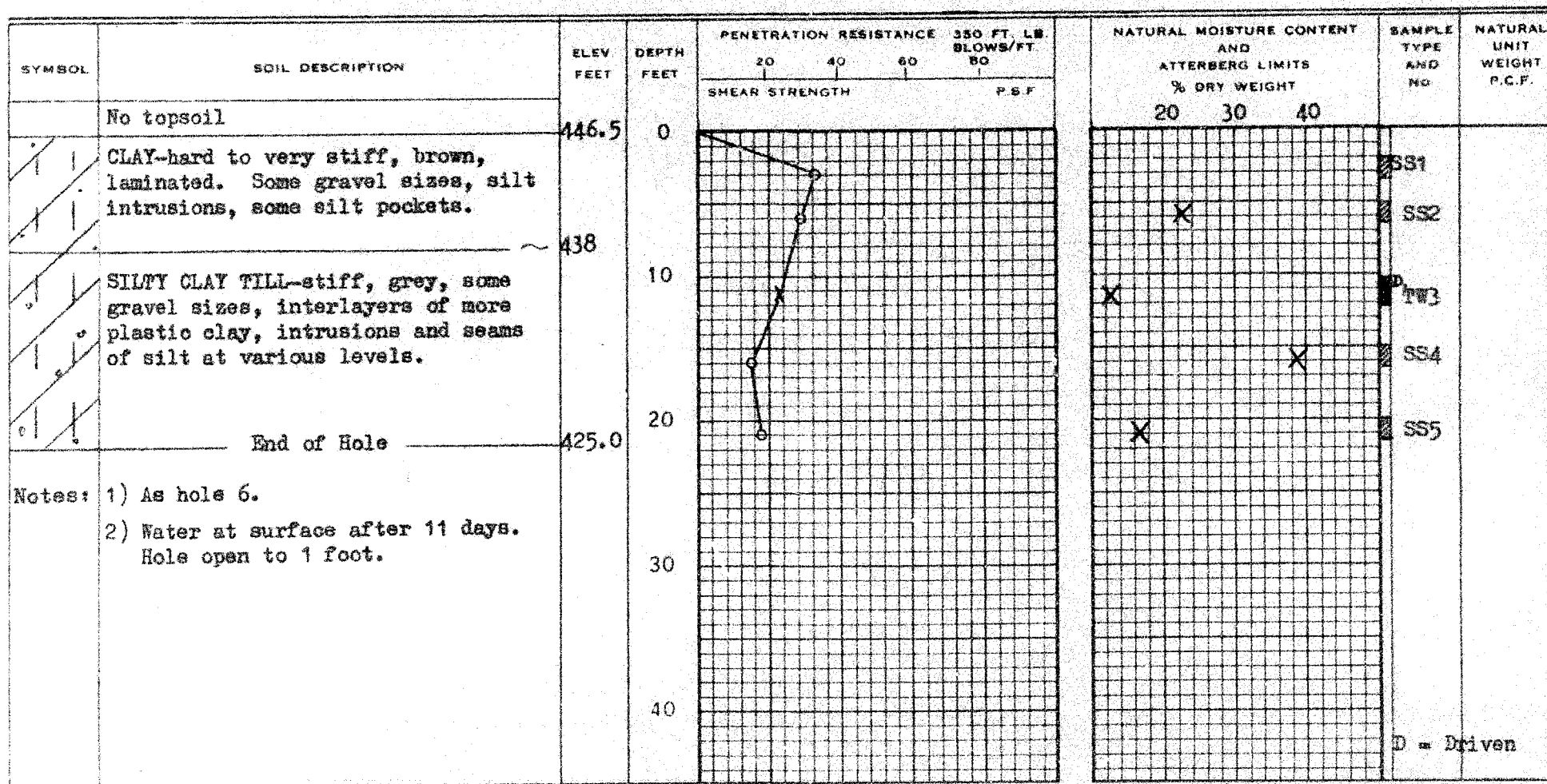
NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX 

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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



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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 20
PROJECT NO. J1067




LEGEND

BOREHOLE NO. 24
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 444.7 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S)  ⁵




NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

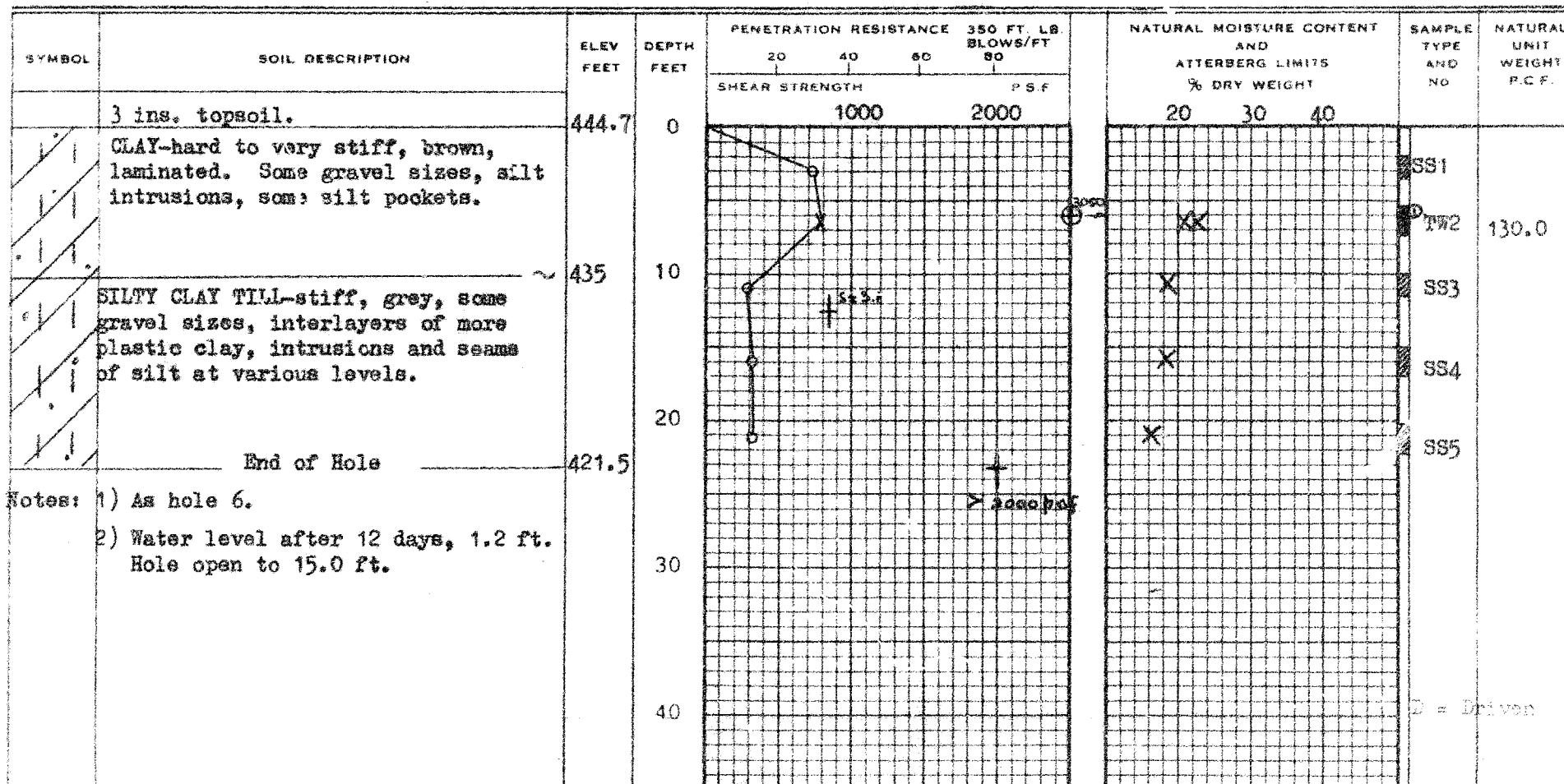
LI
X

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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



WILLIAM A. TROW & ASSOCIATES LTD.




SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

DRAWING NO. 21
PROJECT NO. J1067




LEGEND

BOREHOLE NO. 25
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION AS above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 443.6 ft.
DATUM

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

LI
X

ATTERBERG LIMITS

LIQUID LIMIT

—○—

PLASTIC LIMIT

—|—

SAMPLE TYPE

2" O.D. SPLIT TUBE

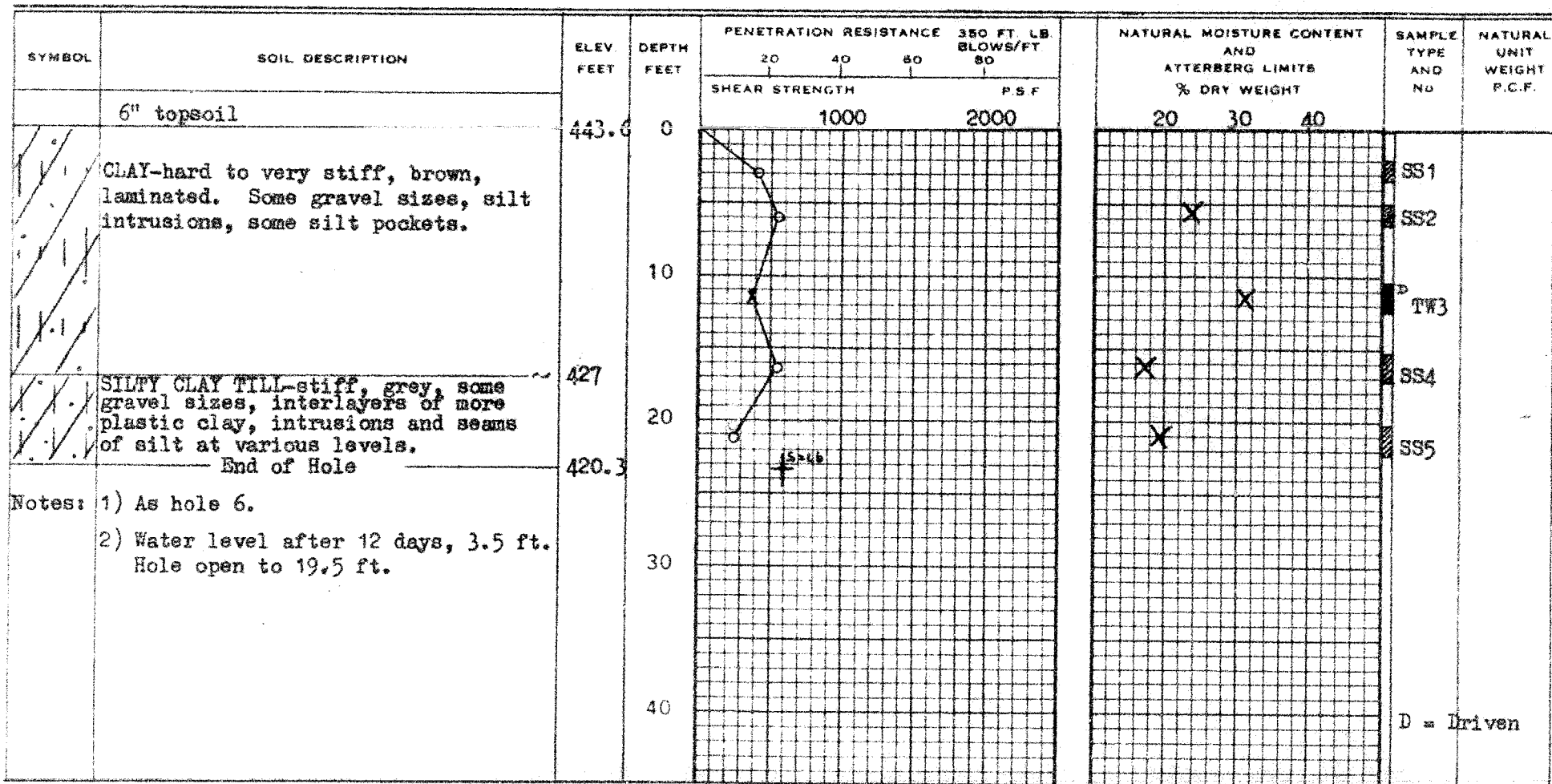


2" I.D. SHELBY TUBE






3" O.D. SHELBY TUBE








BOREHOLE NO. 26
 PROJECT Hwy. 401 - West Approaches to Hwy. 400
 LOCATION As above. Interchange
 HOLE LOCATION See Dwg. 1.
 HOLE ELEVATION 437.8 ft.
 DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 2" DIA. CONE 

SHEAR STRENGTH

UNDRAINED TRIAXIAL
 AT OVERBURDEN PRESSURE 
 UNCONFINED COMPRESSION 
 VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE CONTENT
 AND LIQUIDITY INDEX 

ATTERBERG LIMITS

LIQUID LIMIT 

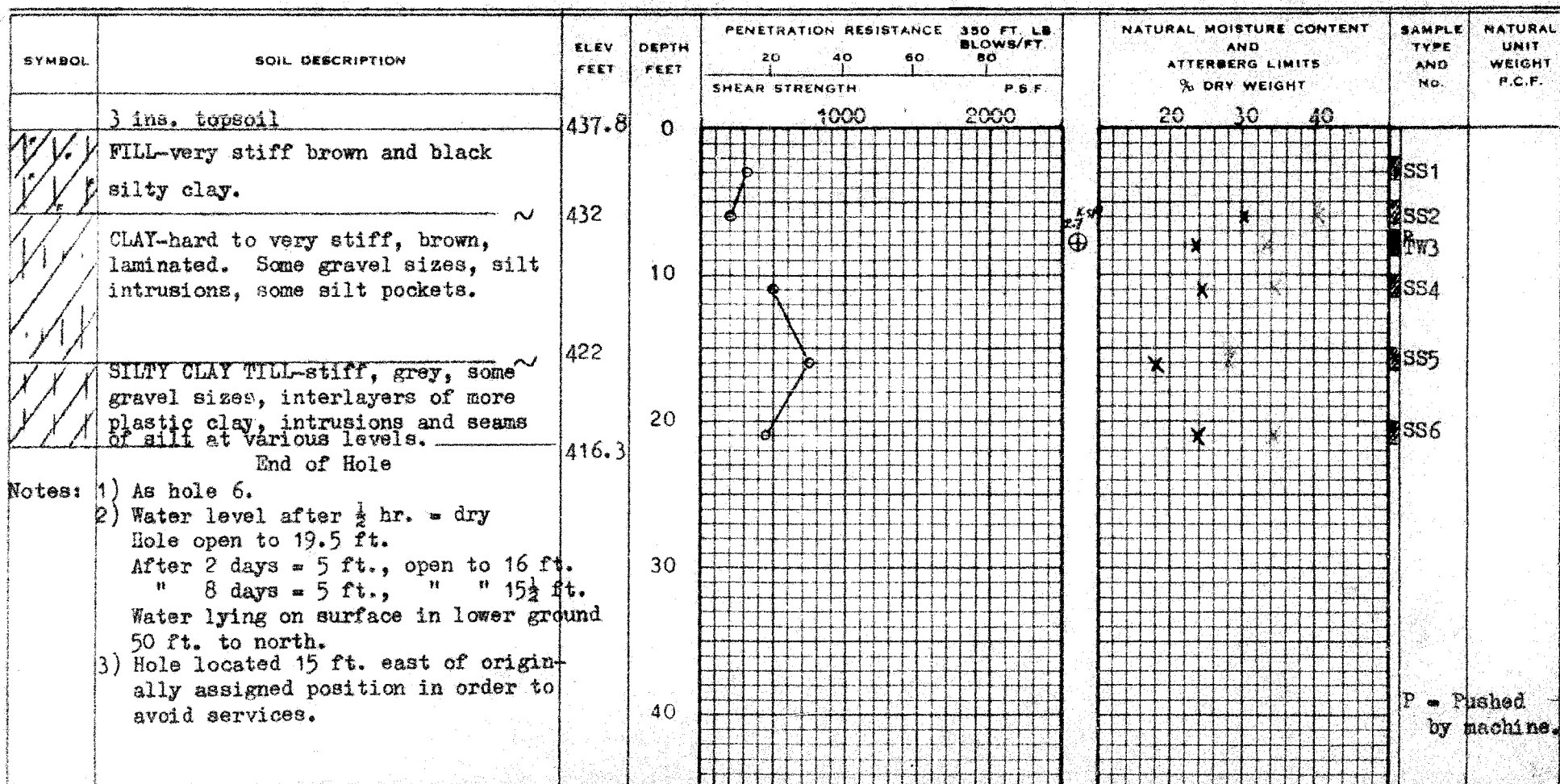
PLASTIC LIMIT 

SAMPLE TYPE

2" O.D. SPLIT TUBE 

2" I.D. SHELBY TUBE 

3" O.D. SHELBY TUBE 



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SITE INVESTIGATIONS - SOIL MECHANICS CONSULTATION

DRAWING NO. 23
PROJECT NO. J1067




LEGEND

BOREHOLE NO. 27
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. **Interchange**
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 434.4 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 




NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

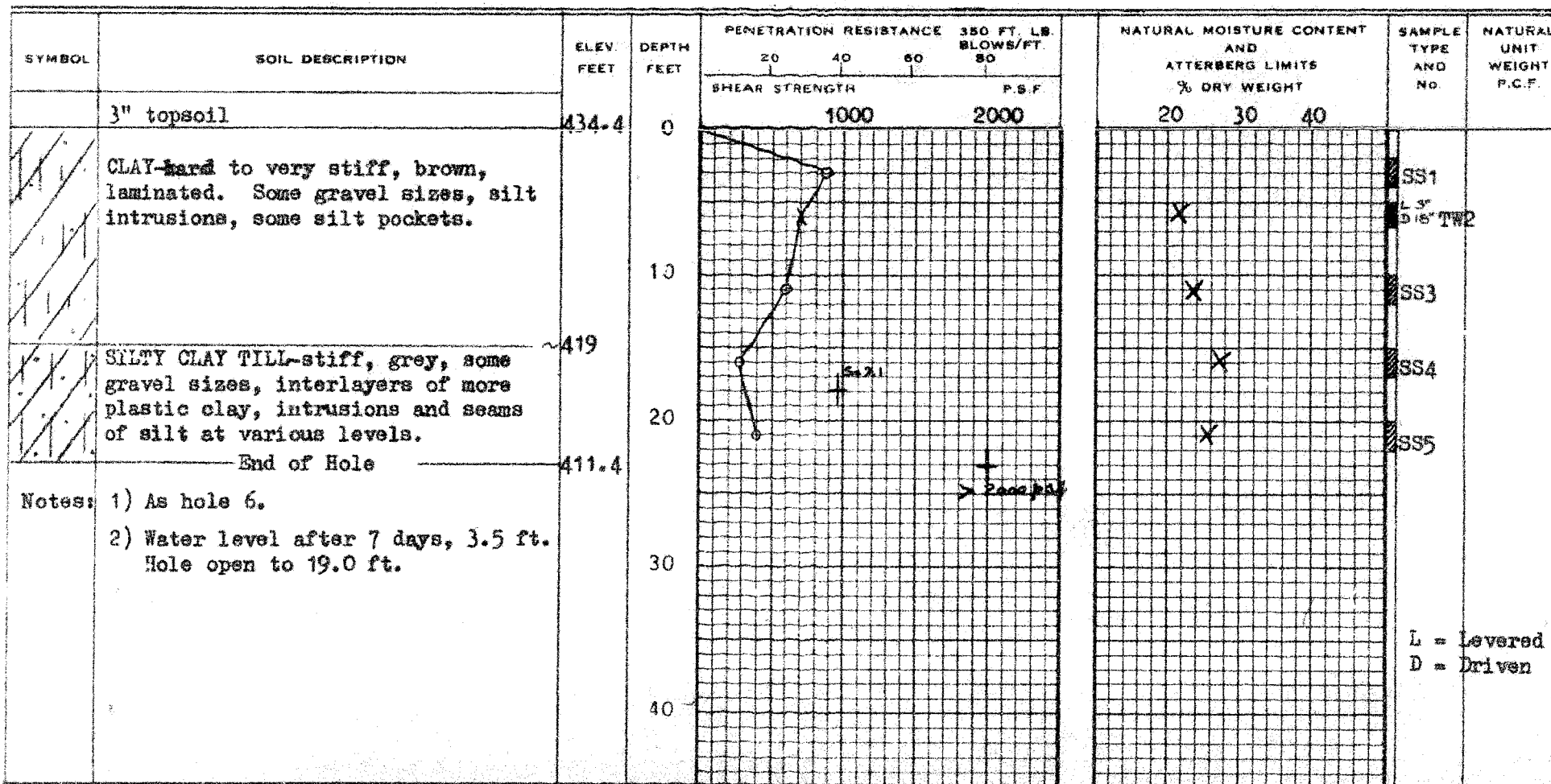
LI 
X

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE




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






LEGEND


BOREHOLE No. 28
 PROJECT Hwy. 401 - West Approaches to Hwy. 400
 LOCATION As above. Interchange
 HOLE LOCATION See Dwg. 1.
 HOLE ELEVATION 426.3 ft.
 DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
 UNCONFINED COMPRESSION 
 VANE TEST AND SENSITIVITY (S) 

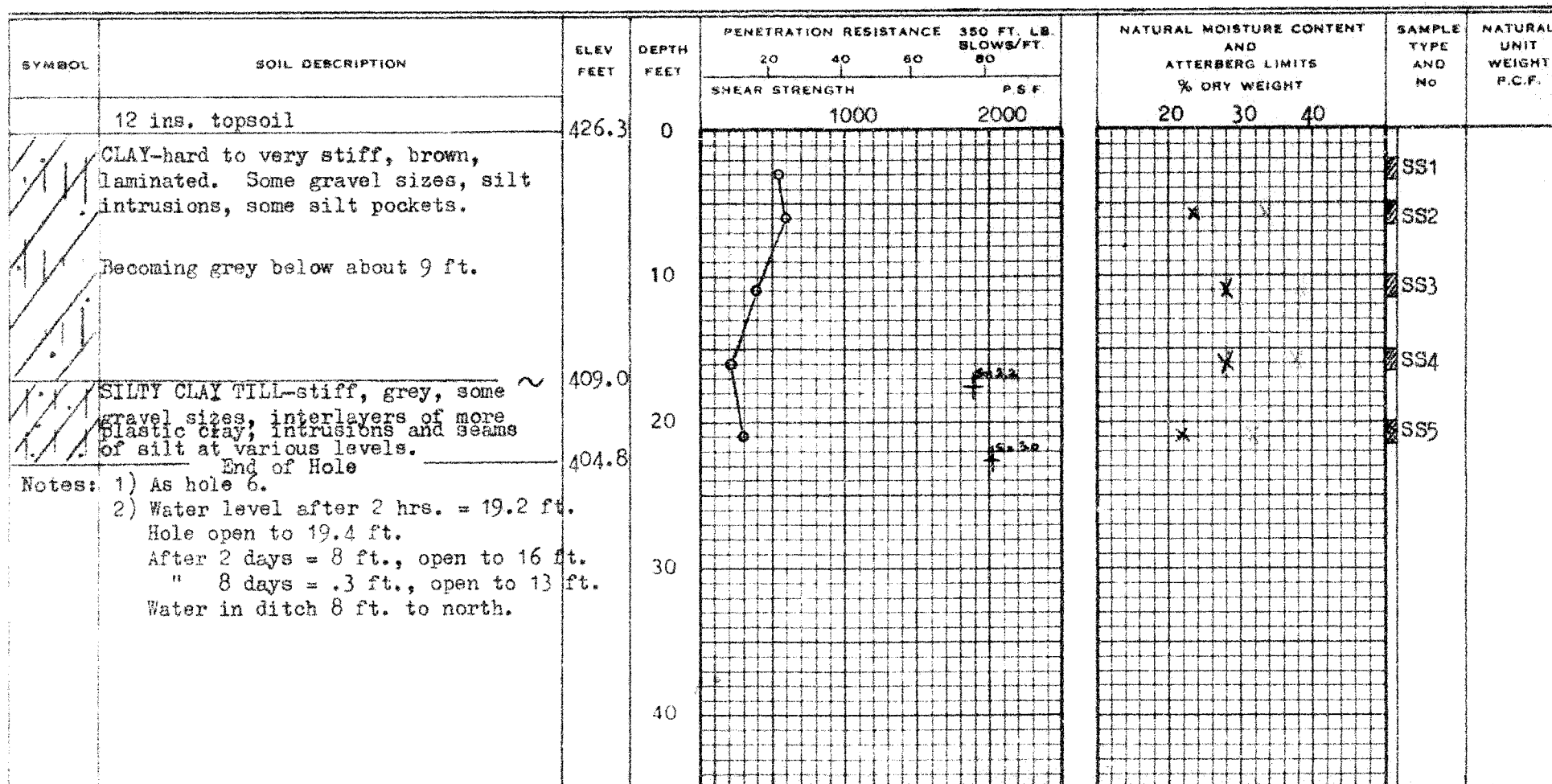
NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX 

ATTERBERG LIMITS

LIQUID LIMIT 
 PLASTIC LIMIT 

SAMPLE TYPE

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



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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

LEGEND

DRAWING NO. 25
PROJECT NO. J1067

BOREHOLE No. 29
PROJECT Hwy. 401 - West Approaches to Hwy. 400 Interchange
LOCATION As above.
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 428.3 ft.
DATUM

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE *-*-*-*
2" DIA. CONE ————

SHEAR STRENGTH

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

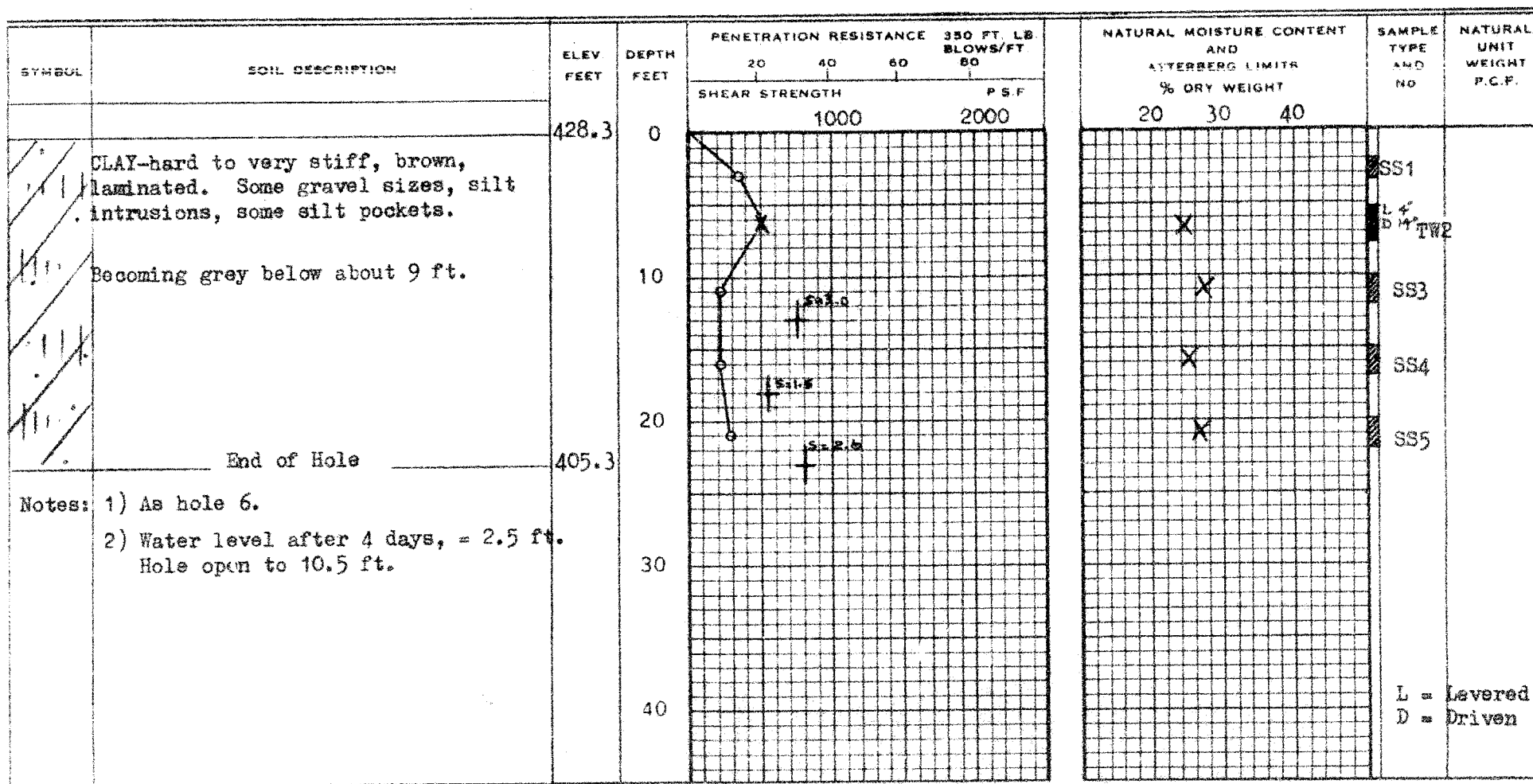
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT ———

SAMPLE TYPE

2" O.D. SPLIT TUBE ⊠
2" I.D. SHELBY TUBE ⊡
3" O.D. SHELBY TUBE ⊢



L = Levered
D = Driven

WILLIAM A. TROW & ASSOCIATES LTD.




SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 26
PROJECT NO. J1067




LEGEND

BOREHOLE NO. 30
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 431.5 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

SAMPLE TYPE

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

LI
X

—
—

—
—
—

SYMBOL SOIL DESCRIPTION ELEV FEET DEPTH FEET

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

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

SS1
P15
D5
TW2
SS3
SS4
SS5

P = Pushed by machine.
D = Driven.

9 ins. topsoil

431.5

0

CLAY-hard to very stiff, brown, laminated. Some gravel sizes, silt intrusions, some silt pockets.

Becomes grey below about 13 ft.

10

End of Hole

409.0

20

Notes: 1) As hole 6.
2) Water level after 6 hrs. = 19.7 ft.
Hole open to 19.8 ft.
After 2 days = 16.8 ft., open to 17.5 ft.
" 8 days = 9 ft., open to 13.5 ft.

30

40

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 27
PROJECT NO. J1067

LEGEND

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE *—*—*—*—
2" DIA. CONE ————

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) +^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

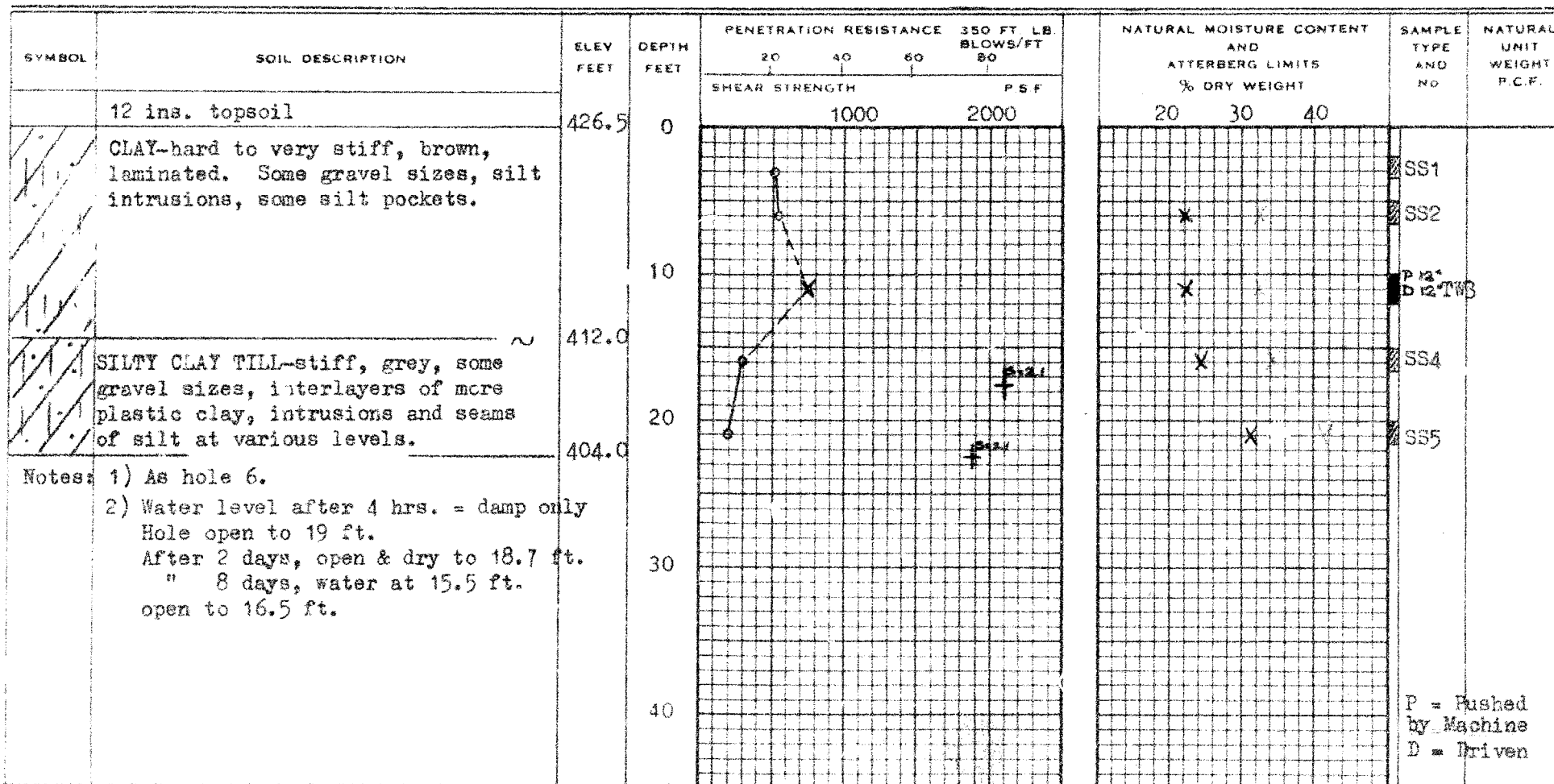
ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT ———

SAMPLE TYPE







2" O.D. SPLIT TUBE ———
2" I.D. SHELBY TUBE —■—
3" O.D. SHELBY TUBE —■—







BOREHOLE No. 31
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 426.5 ft.
DATUM _____

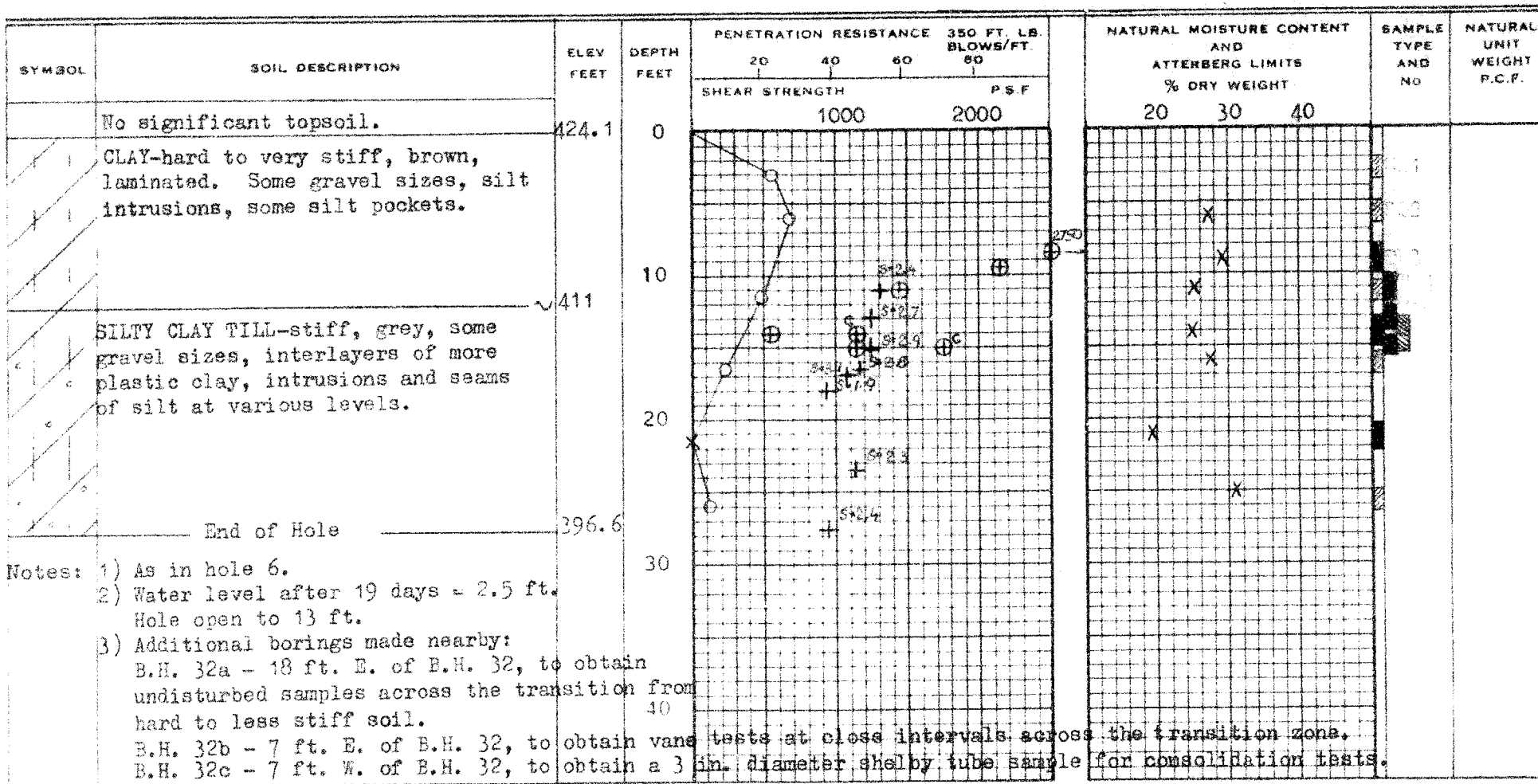


LEGEND

BOREHOLE No. 32
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 424.1 ft.
DATUM _____

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX 
ATTERBERG LIMITS
LIQUID LIMIT 
PLASTIC LIMIT 
SAMPLE TYPE
2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
3" O.D. SHELBY TUBE 



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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 29
PROJECT No. J1067

LEGEND

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE * * * * *
2" DIA. CONE ————

SHEAR STRENGTH

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

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

X^{LI}

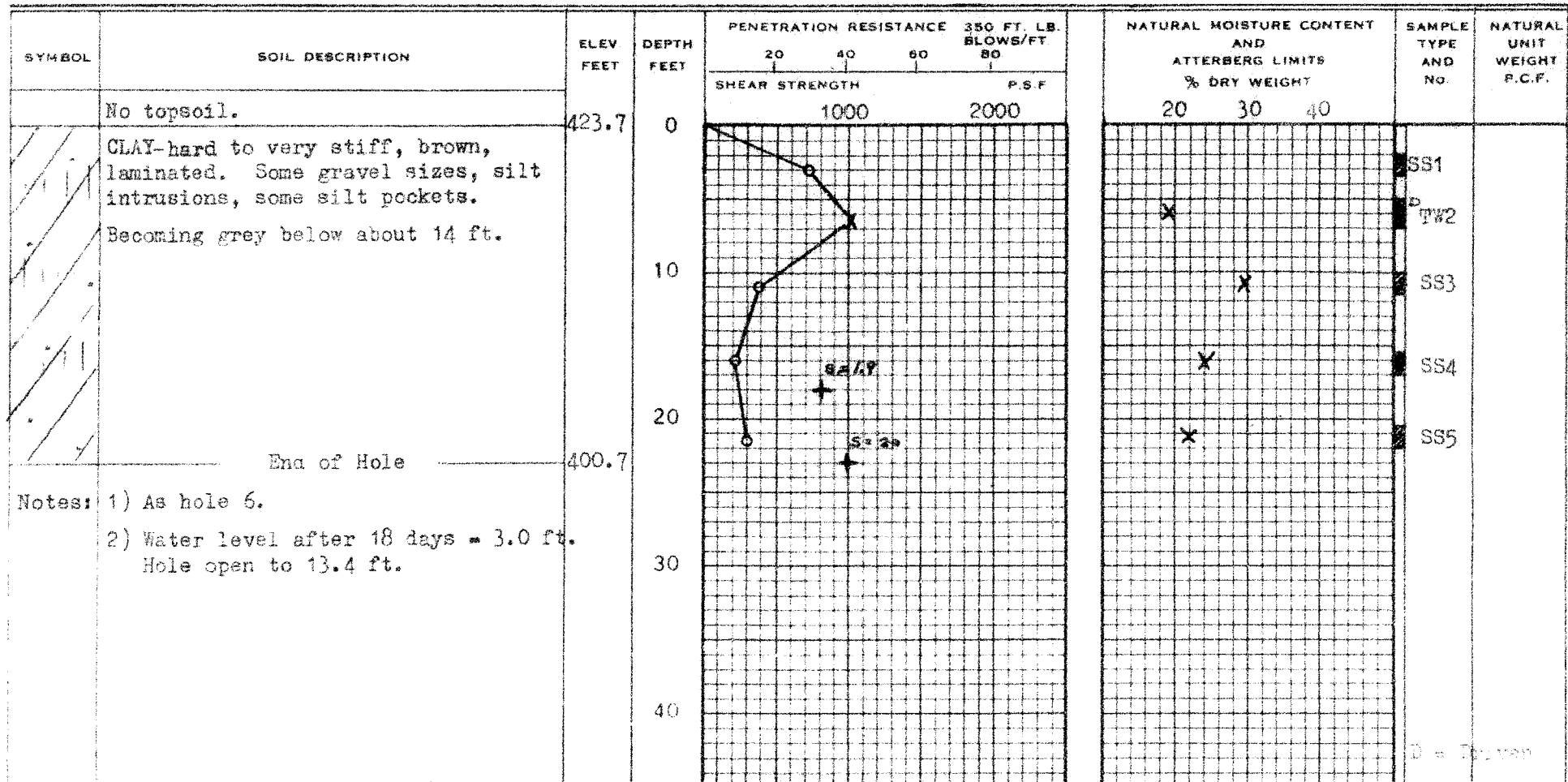
ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT ———

SAMPLE TYPE

2" O.D. SPLIT TUBE —■—
2" I.D. SHELBY TUBE —■—
3" O.D. SHELBY TUBE —■—








BOREHOLE No. 33
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 423.7 ft.
DATUM _____








LEGEND

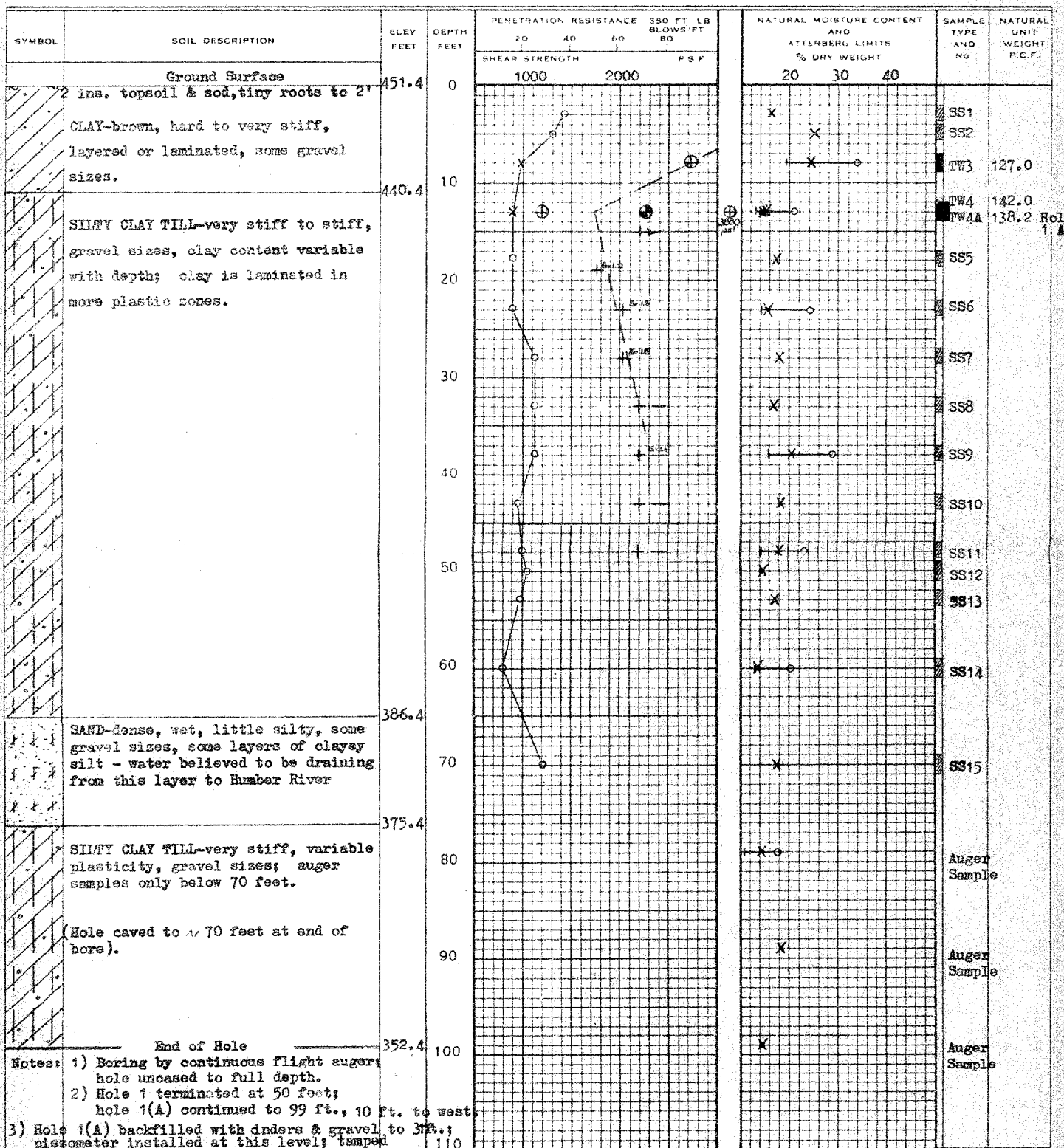
BOREHOLE No. 1
 PROJECT Proposed Expansion of Hwy. 401. W.P. 233-60
 LOCATION C.P.R. West of Wendell Avenue
 HOLE LOCATION See Dwg. 1.
 HOLE ELEVATION 451.4 ft.
 DATUM B.M. No. T201 Weston Road Bridge over Hw. 401
= El 431.255

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 2" DIA. CONE 
 SHEAR STRENGTH 
 UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
 UNCONFINED COMPRESSION 
 VANE TEST AND SENSITIVITY 
 Consolidated cyclic undrained triaxial test

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS
 LIQUID LIMIT 
 PLASTIC LIMIT 
 SAMPLE TYPE
 2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 3" O.D. SHELBY TUBE 



Notes: 1) Boring by continuous flight augers; hole uncased to full depth.
 2) Hole 1 terminated at 50 feet; hole 1(A) continued to 99 ft., 10 ft. to west.
 3) Hole 1(A) backfilled with cinders & gravel to 375.4; piezometer installed at this level; tamped bentonite seal from 21 1/2 to 27 1/2 ft. Water level to 15 ft. when piezometer installed.
 For water level see Table 1.

+ -- = in excess of 2200 psf

LEGEND

BOREHOLE NO. 2
 PROJECT Proposed Expansion of Hwy. 101. W.P. 233-60
 LOCATION C.P.R. West of Wendell Avenue
 HOLE LOCATION See Dig. 1.
 HOLE ELEVATION 444.4 ft.
 DATUM See Hole 1.

PENETRATION RESISTANCE

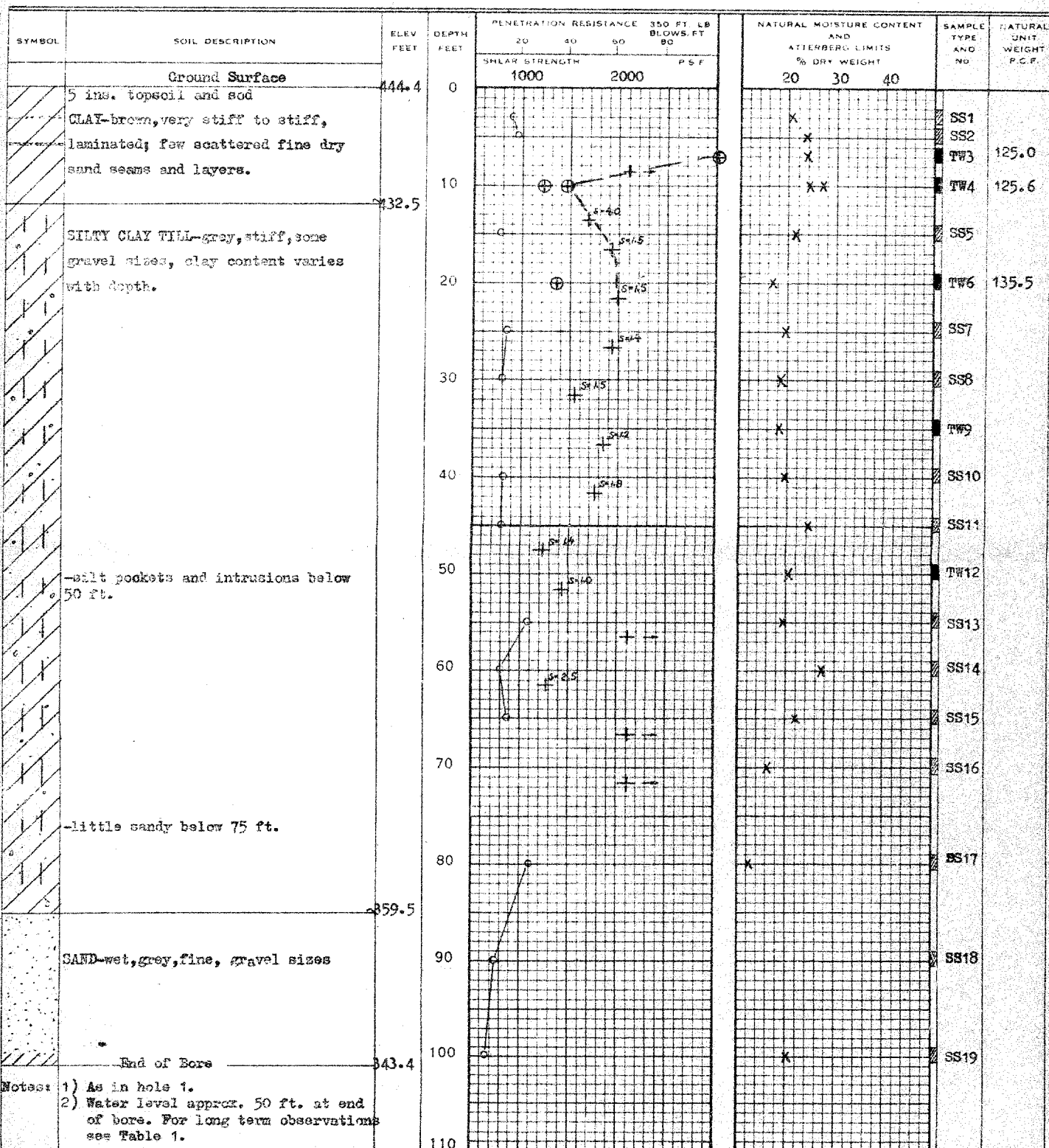
2 O.D. SPLIT TUBE \bigcirc
 2 I.D. SHELBY TUBE \times
 2 DIA. CONE \rightarrow
 SHEAR STRENGTH
 UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE \oplus
 UNCONFINED COMPRESSION \otimes
 VANE TEST AND SENSITIVITY (S, \dagger)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX \times LI

ATTERBERG LIMITS

LIQUID LIMIT \bigcirc PLASTIC LIMIT \rightarrow

SAMPLE TYPE

2" O.D. SPLIT TUBE \bigcirc 2" I.D. SHELBY TUBE \times 3" O.D. SHELBY TUBE \otimes 

BOREHOLE NO. 3
 PROJECT Proposed Expansion of Hwy. 401. W.P. 233-60
 LOCATION C.P.R. West of Wendell Avenue, Bridge 125
 HOLE LOCATION See Dwg. 1.
 HOLE ELEVATION 429.9 ft.
 DATUM See Hole 1.

PENETRATION RESISTANCE

1. 0.0 SPLIT TUBE
 2. 1.0 SHELBY TUBE
 3. DIA. CONE

SHEAR STRENGTH

- UNDRAINED TRIAXIAL
 AT OVERBURDEN PRESSURE
 UNCONFINED COMPRESSION
 VANE TEST AND SENSITIVITY $(s_u) + s$

Consolidated, cyclic,
 undrained triaxial test.

NATURAL MOISTURE CONTENT

AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT

PLASTIC LIMIT

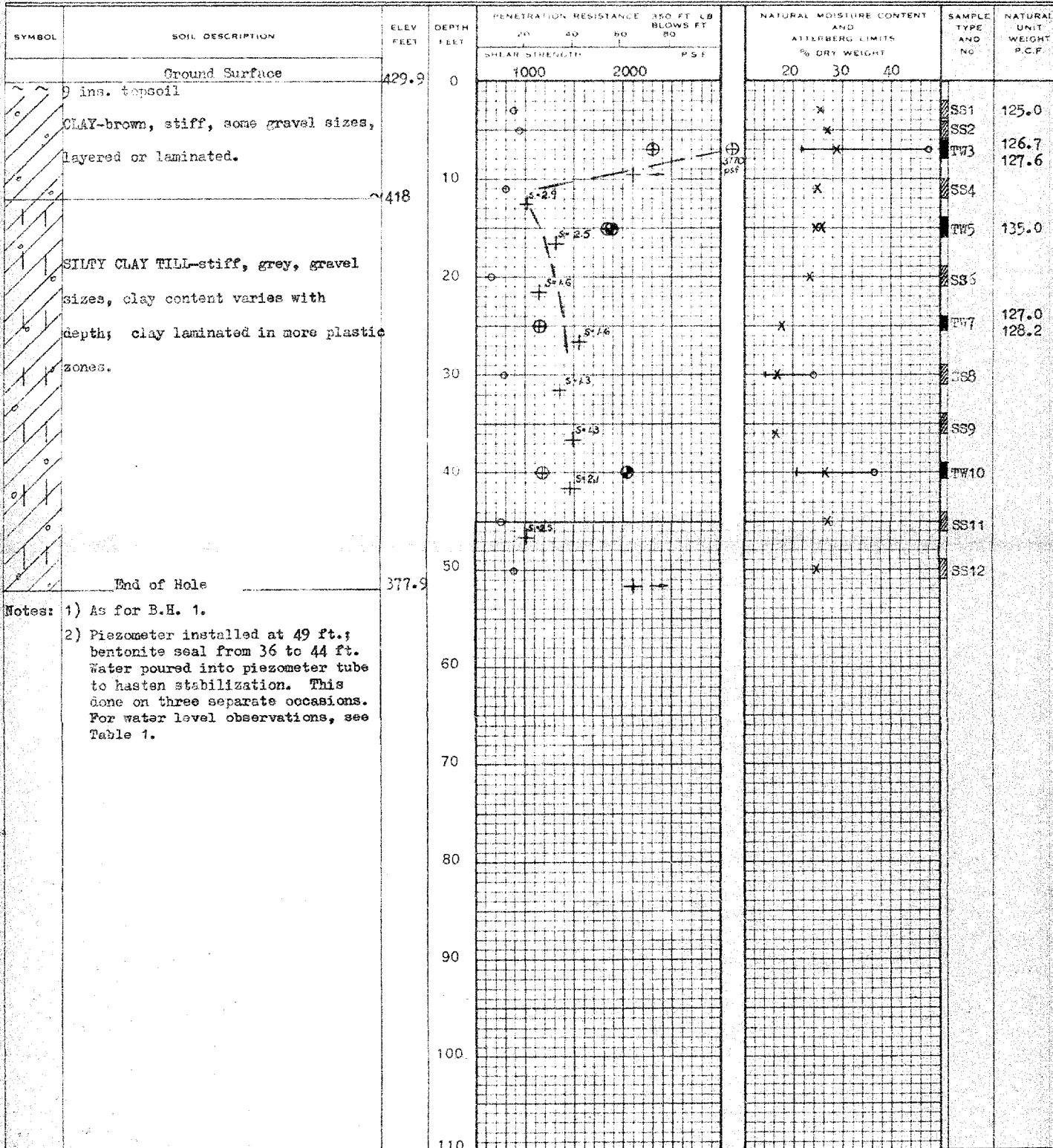
SAMPLE TYPE

2. 0.0 SPLIT TUBE

2. 1.0 SHELBY TUBE




1. 0.0 SHELBY TUBE

X LI






BOREHOLE NO. 4
 PROJECT Proposed Expansion of Hwy. 401, W.P. 233-60
 LOCATION C.P.R. West of Wendell Avenue
 HOLE LOCATION See Dwg. 1.
 HOLE ELEVATION 429.0 ft.
 DATUM See Hole 1.

PENETRATION RESISTANCE

2 O.D. SPLIT TUBE 
 2 I.D. SHELBY TUBE 
 2 DIA. CONE 

SHEAR STRENGTH




UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
 UNCONFINED COMPRESSION 
 VANE TEST AND SENSITIVITY (S, τ) 

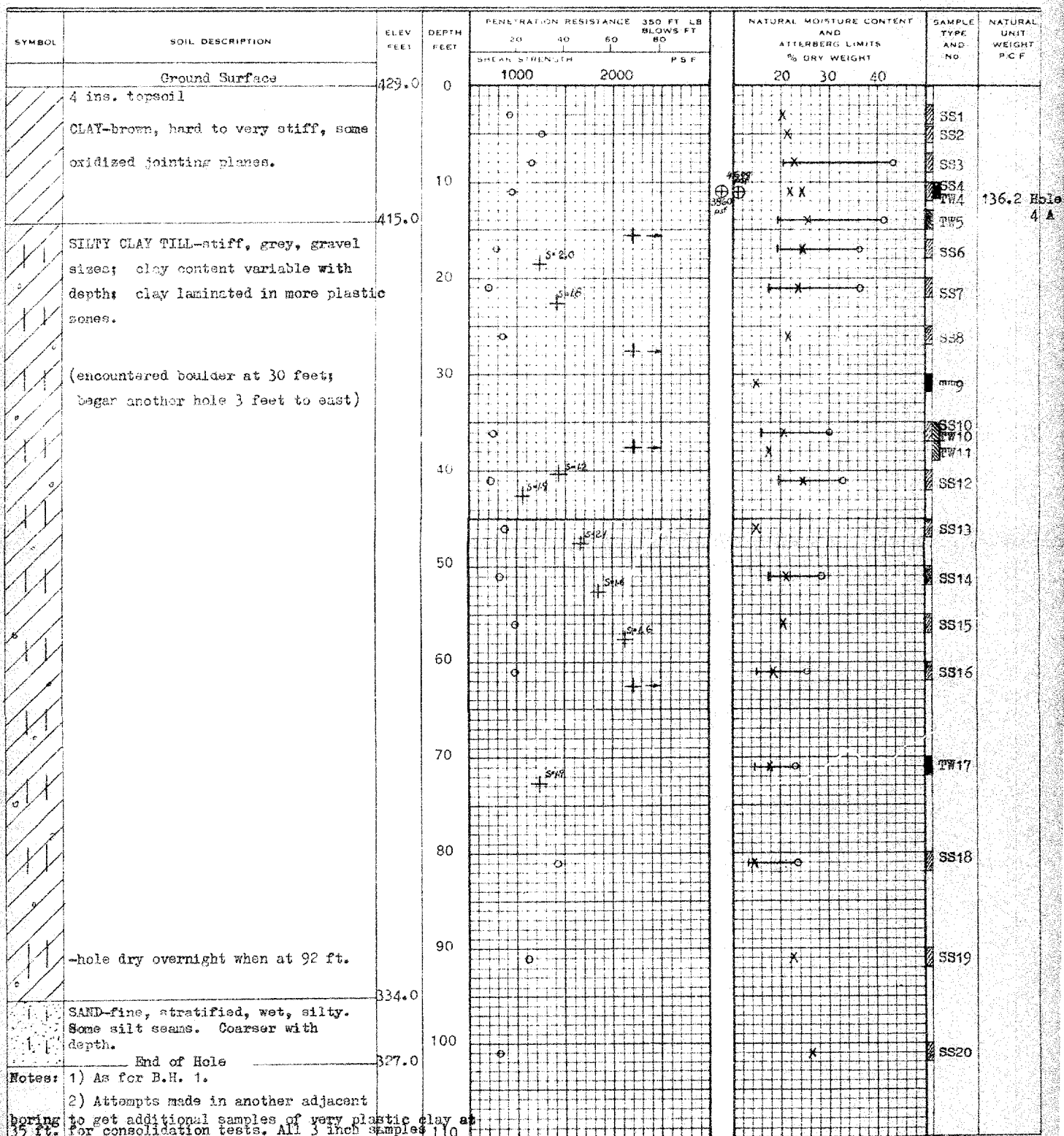
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT 
 PLASTIC LIMIT 

SAMPLE TYPE

2 O.D. SPLIT TUBE 
 2 I.D. SHELBY TUBE 
 2 O.D. SHELBY TUBE 



WILLIAM A. TROW & ASSOCIATES LTD.


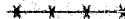

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

LEGEND




DRAWING No. 30
PROJECT No. J1067

BOREHOLE No. 34
PROJECT Hwy. 401 - West Approaches to Hwy. 400
LOCATION As above. Interchange
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 422.1 ft.
DATUM _____

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
UNCONFINED COMPRESSION 
VANE TEST AND SENSITIVITY (S) 




NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

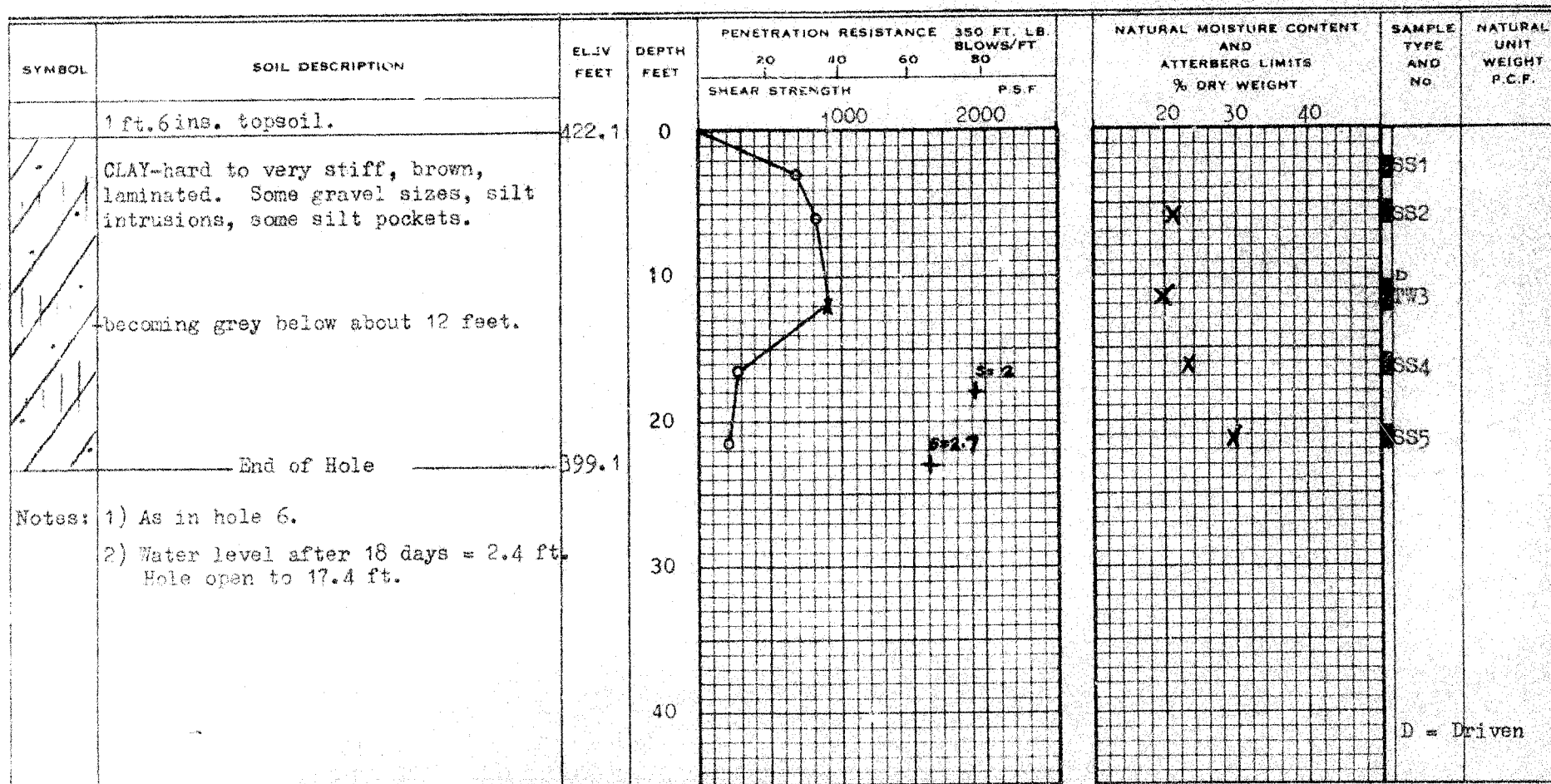
X^{LI}

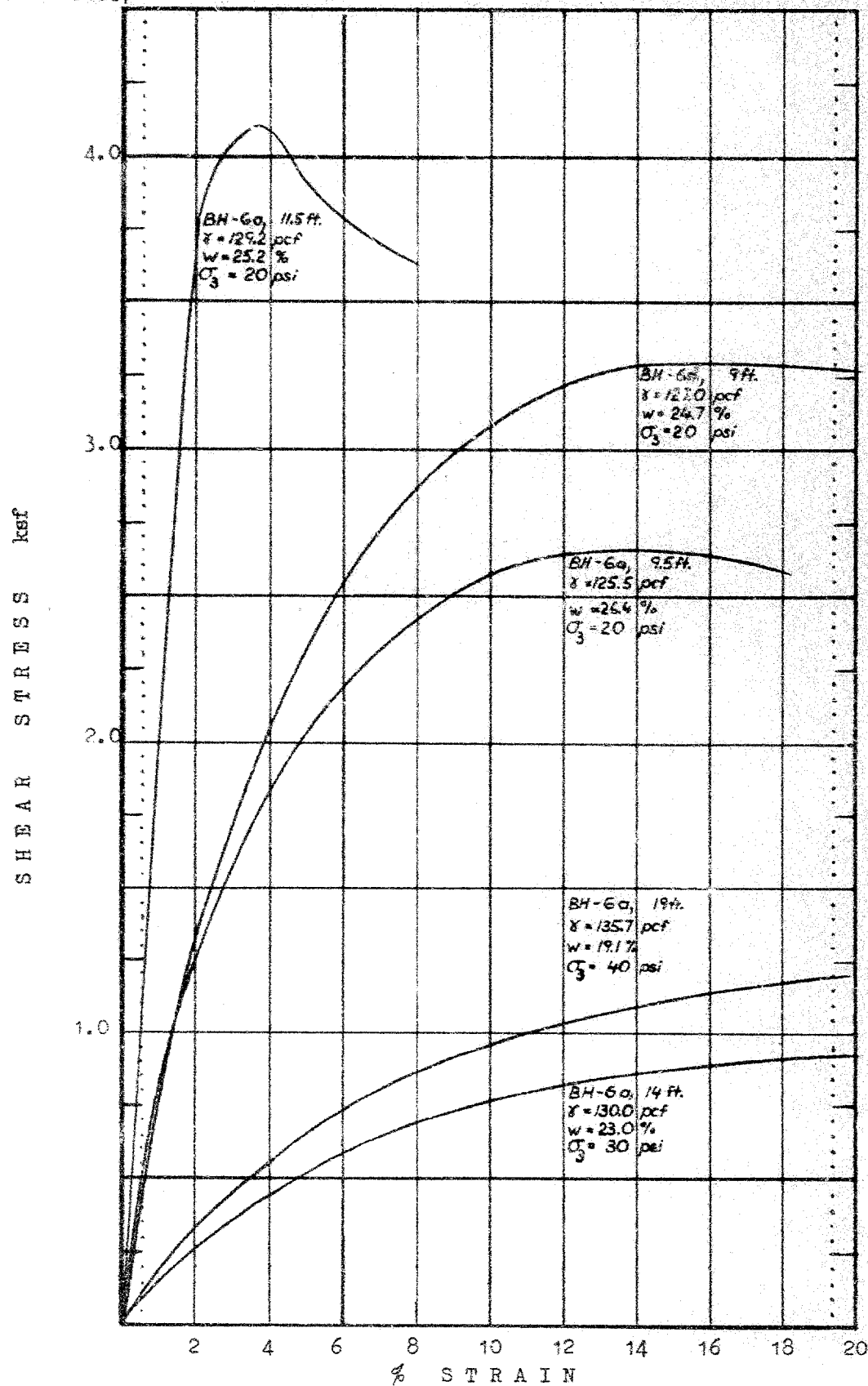
ATTERBERG LIMITS

LIQUID LIMIT 
PLASTIC LIMIT 

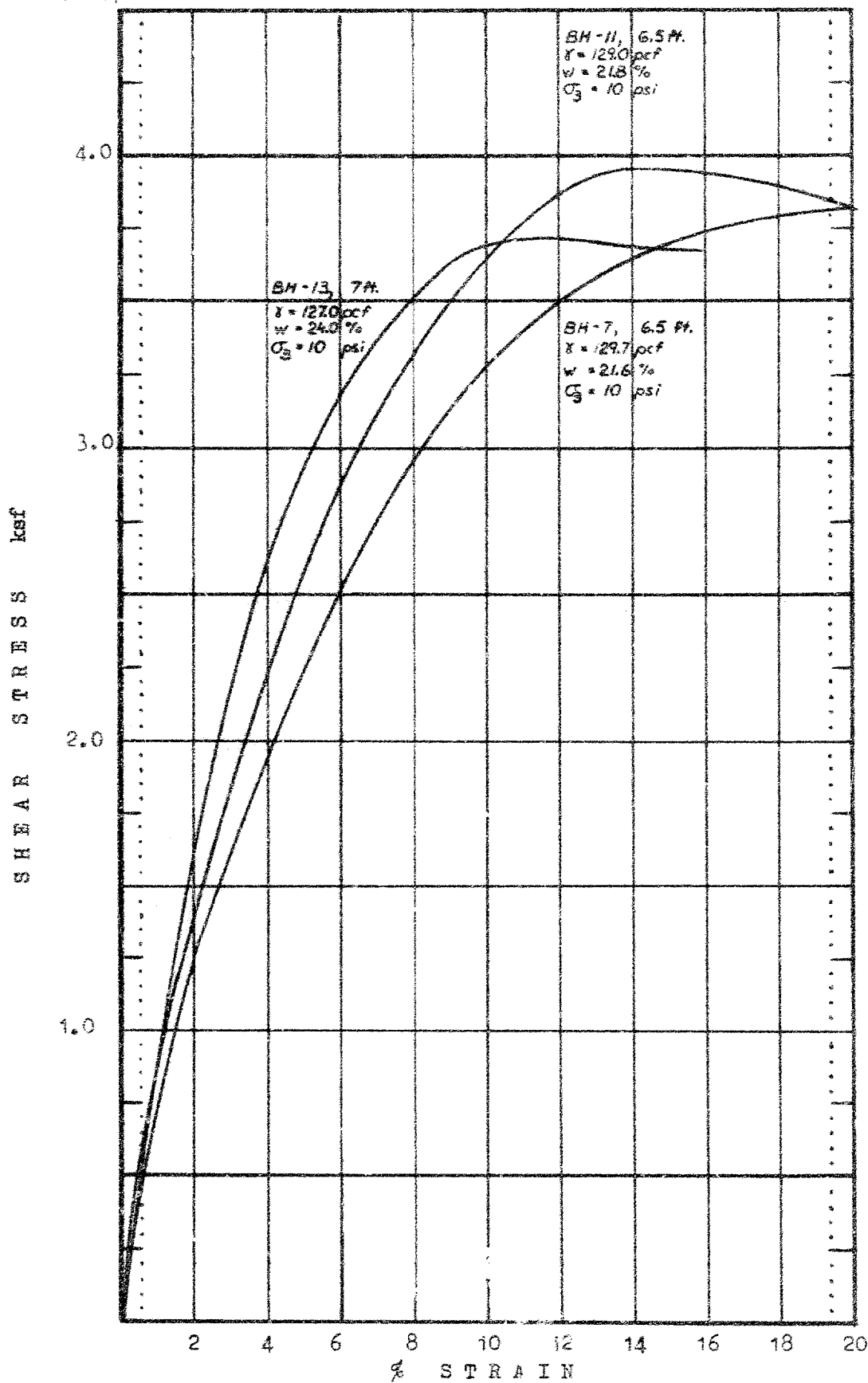
SAMPLE TYPE

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

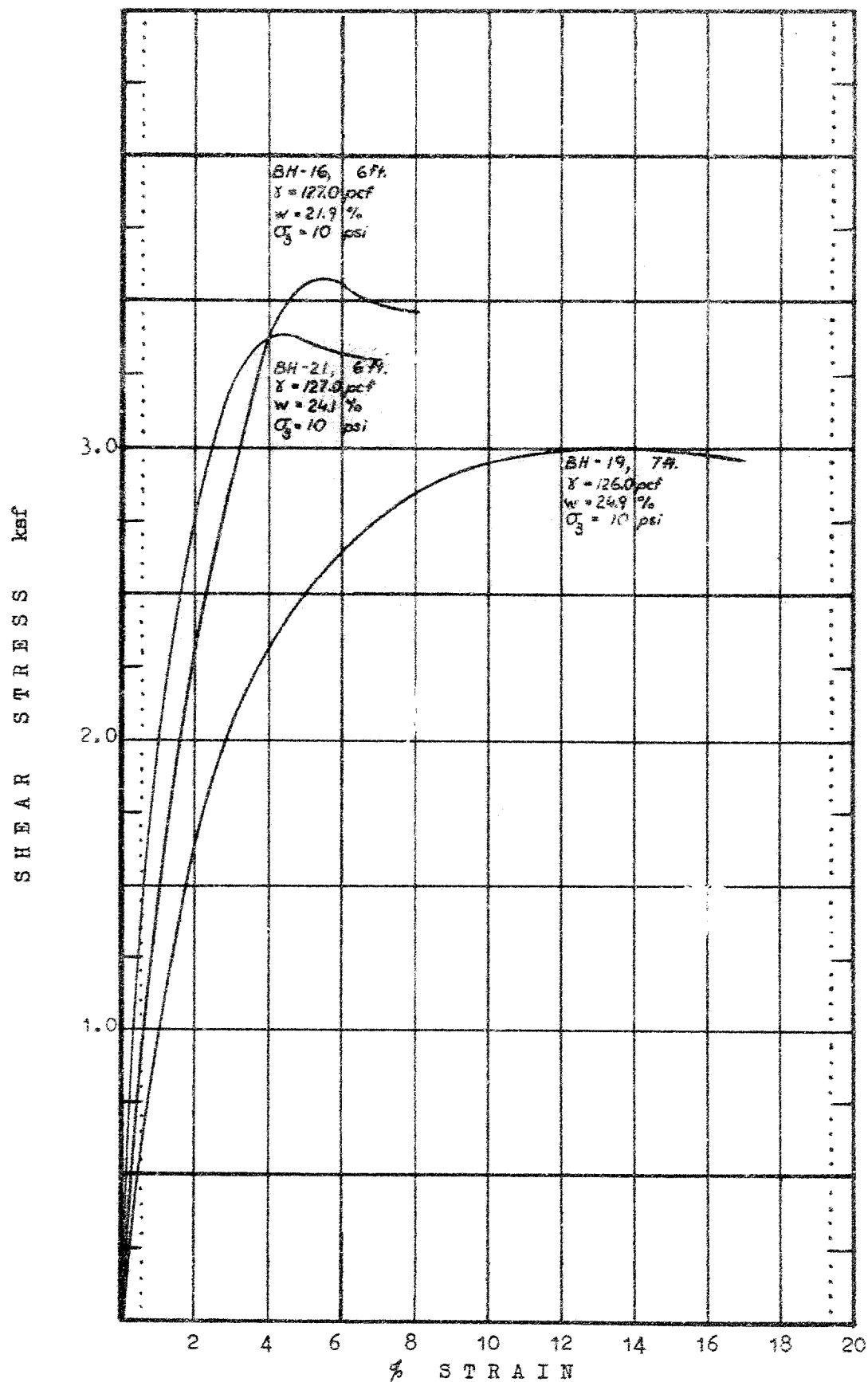




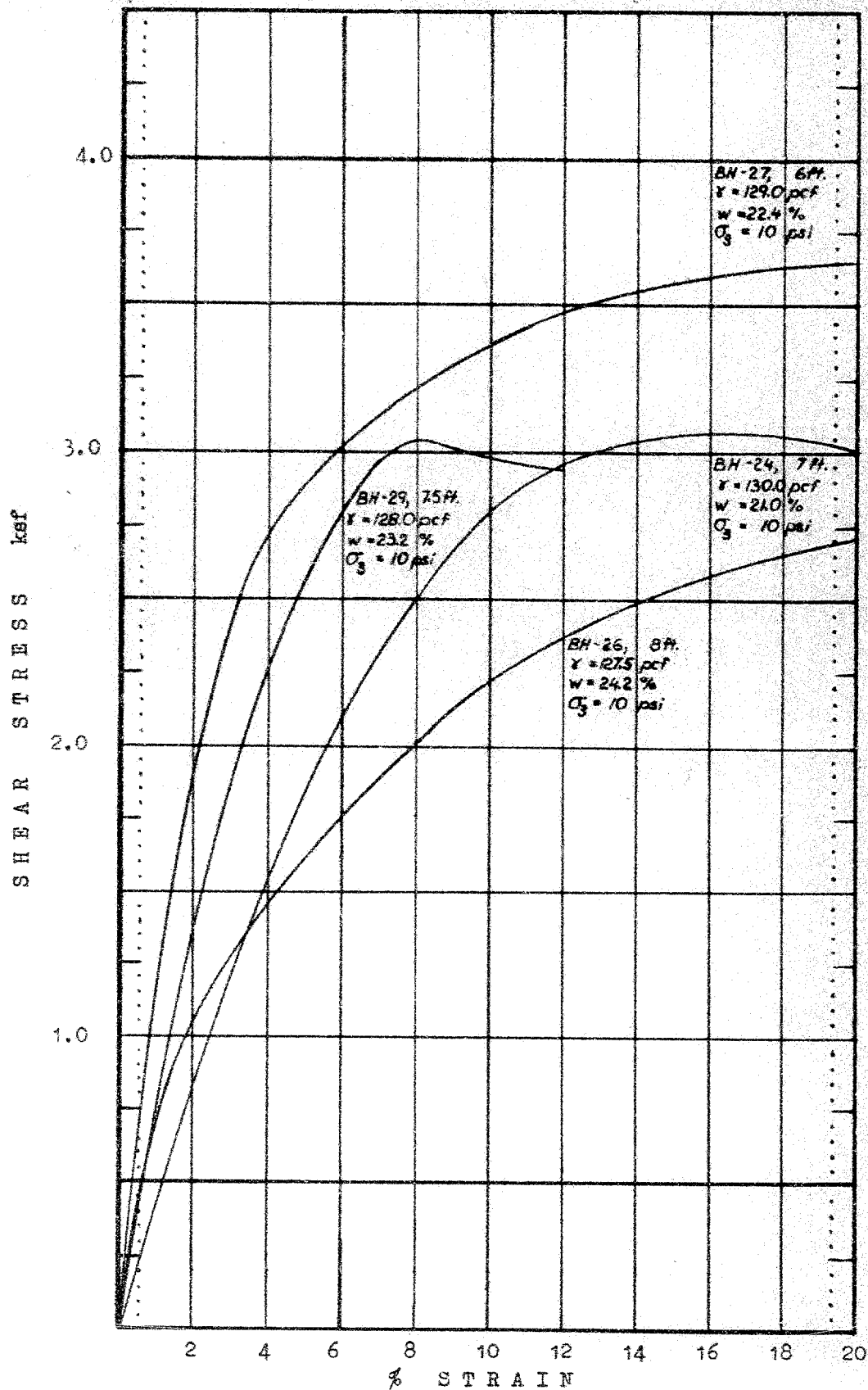
UNDRAINED TRIAXIAL TEST RESULTS



UNDRAINED TRIAXIAL TEST RESULTS

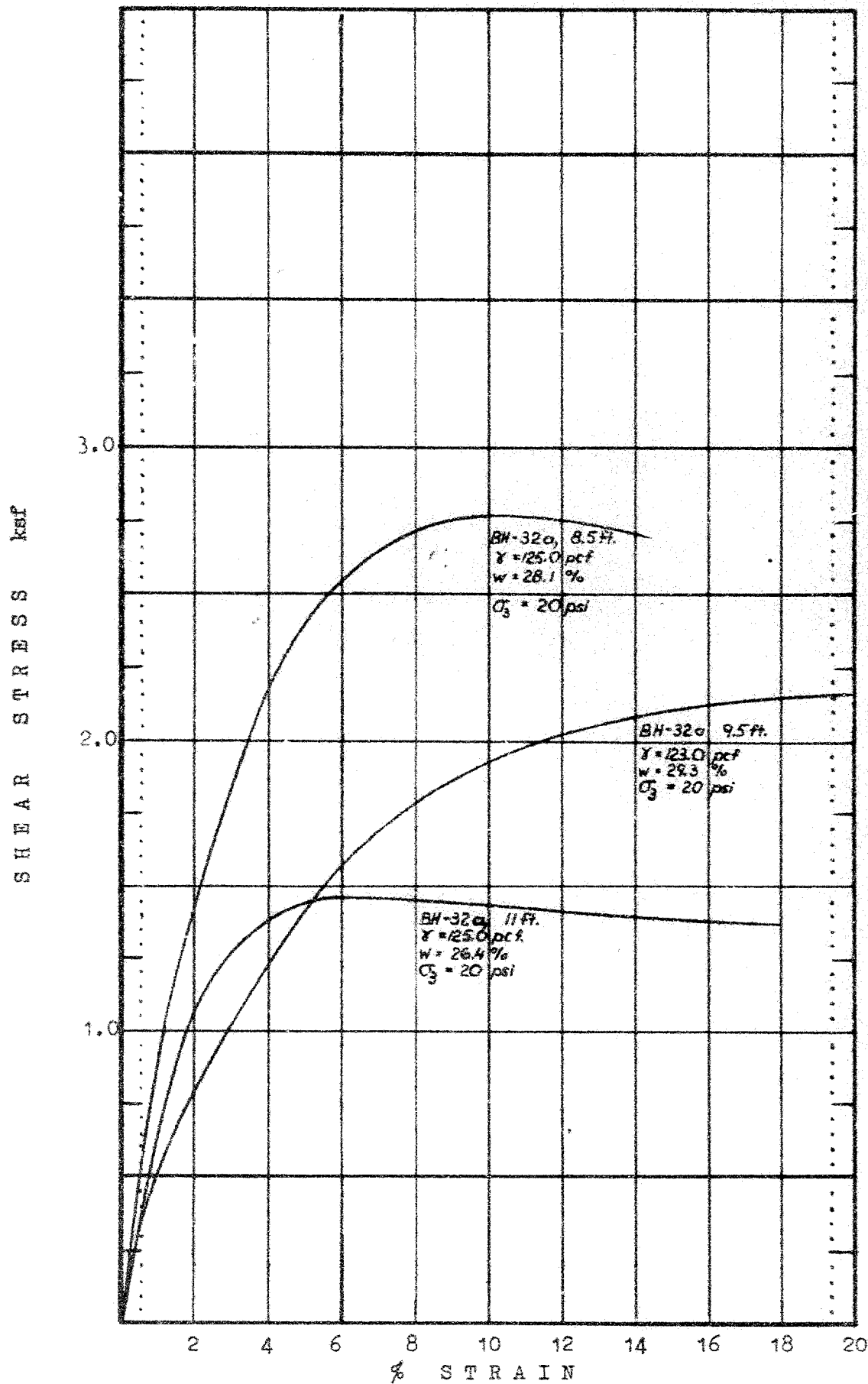


UNDRAINED TRIAXIAL TEST RESULTS



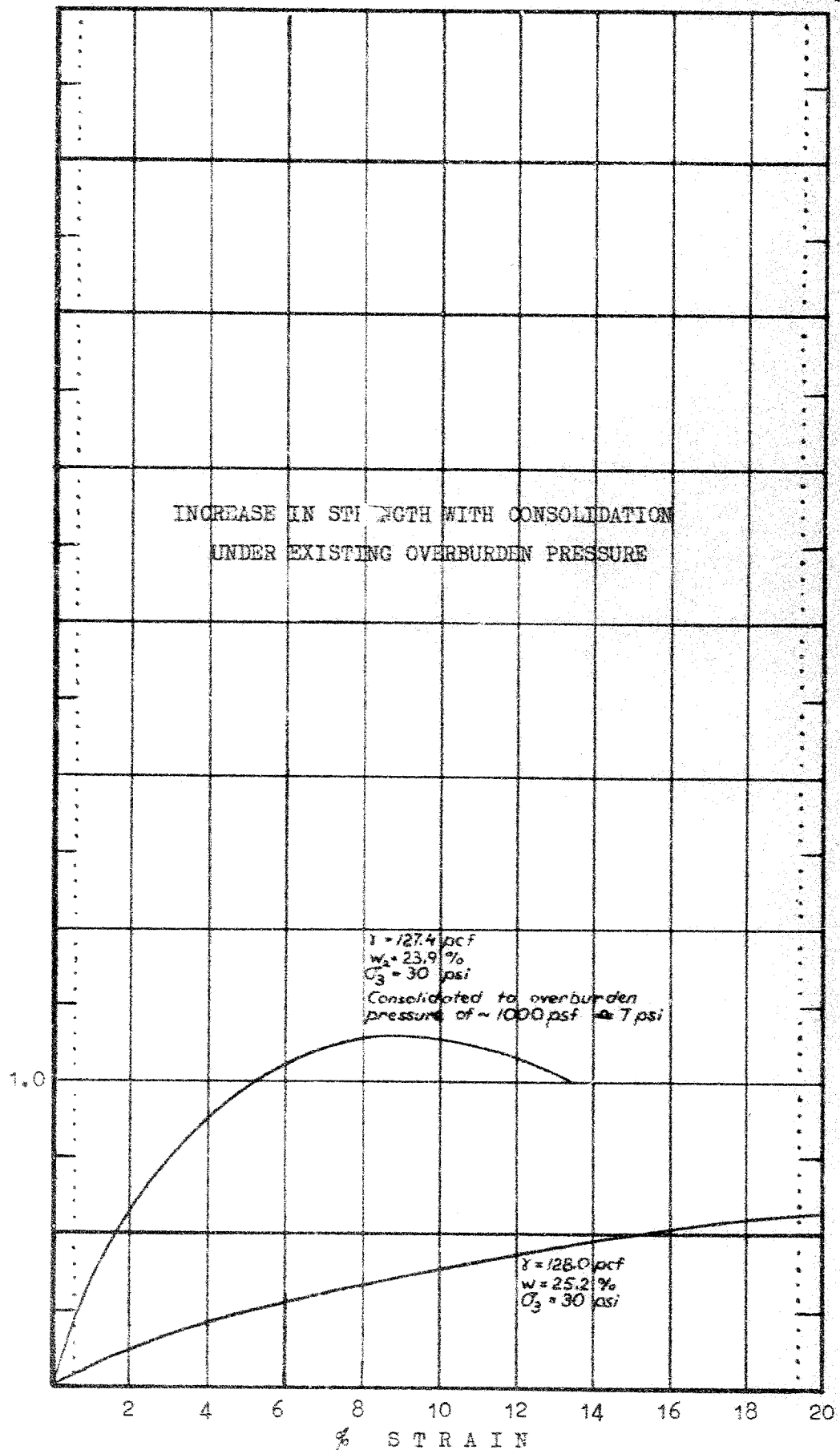
UNDRAINED TRIAXIAL TEST RESULTS

WILLIAM A. TROW AND ASSOCIATES



UNDRAINED TRIAXIAL TEST RESULTS

S H E A R S T R E S S k s f



UNDRAINED TRIAXIAL TEST RESULTS
BOREHOLE 32a - 14 FT.

WILLIAM A. TROW AND ASSOCIATES

(w_2 = Moisture Content
after consolidation)

SHEAR STRESS ksf

2.0

1.0

INCREASE IN STRENGTH WITH CONSOLIDATION
UNDER EXISTING OVERBURDEN PRESSURE $\gamma = 126.7 \text{ pcf}$
 $w_L = 23.6 \%$
 $\sigma_3 = 30 \text{ psi}$ Consolidated to overburden
pressure of $\sim 1000 \text{ psf} \approx 7 \text{ psi}$ $\gamma = 126.0 \text{ pcf}$
 $w_L = 24.7 \%$
 $\sigma_3 = 30 \text{ psi}$

2 4 6 8 10 12 14 16 18 20

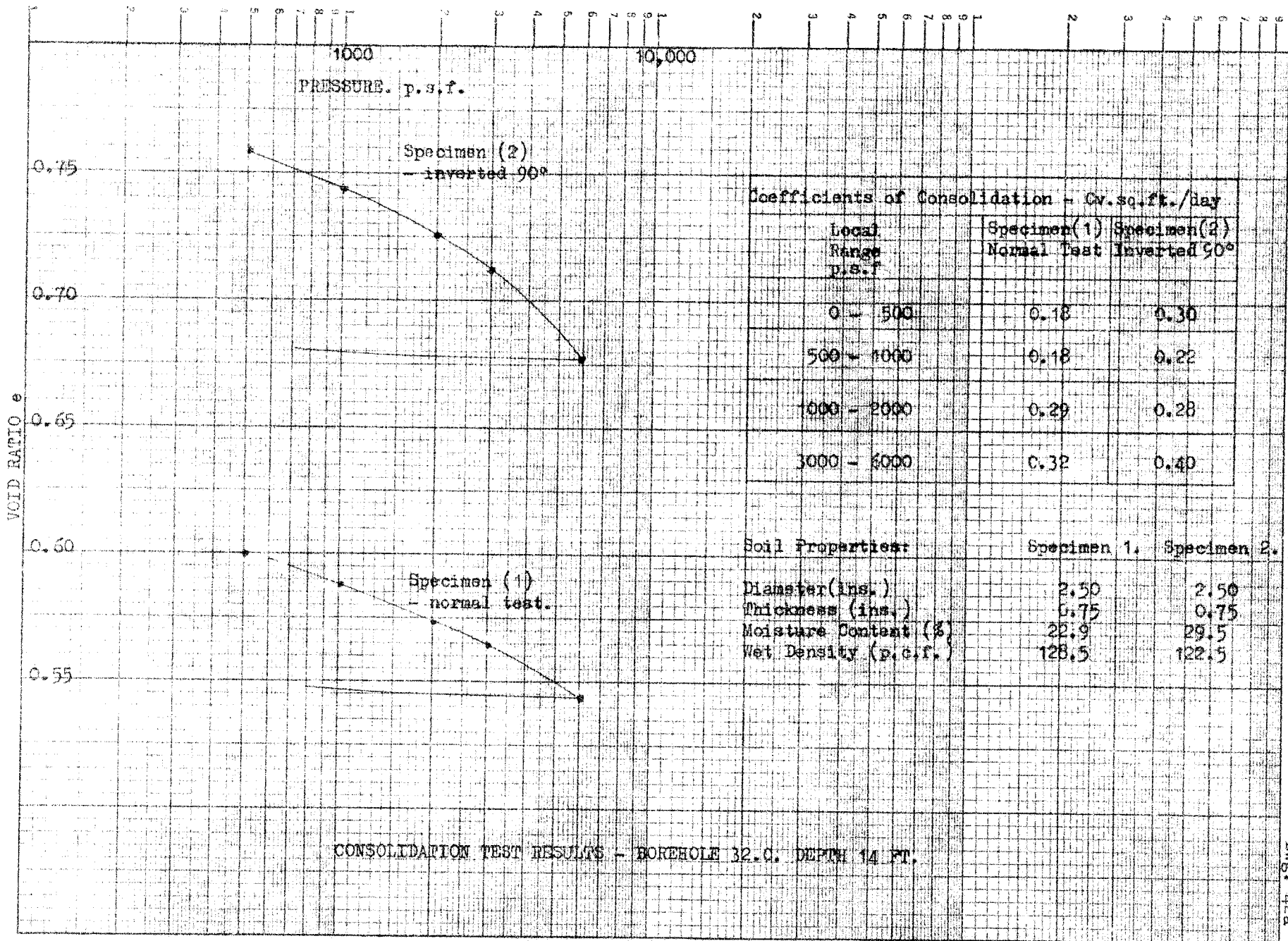
% STRAIN

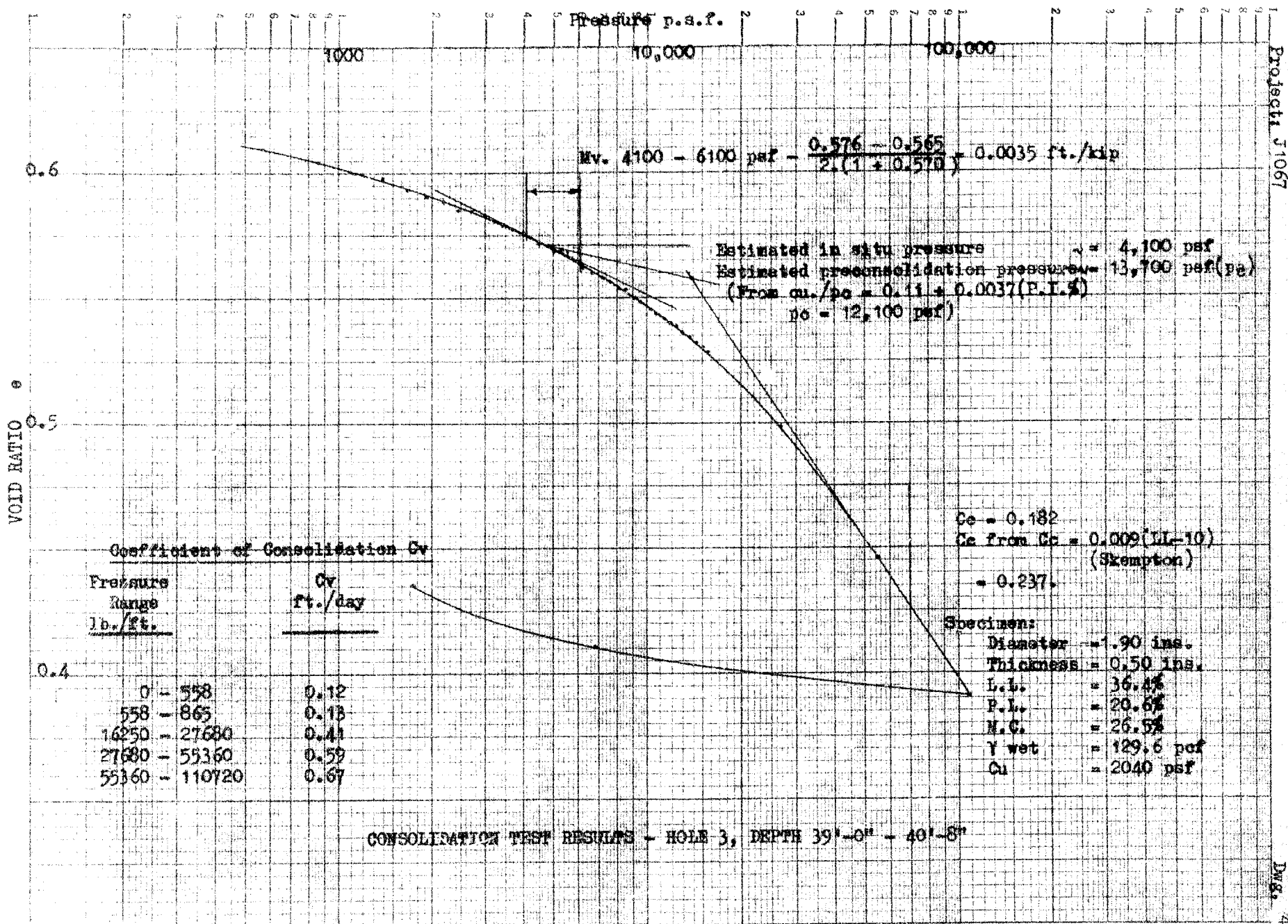
UNDRAINED TRIAXIAL TEST RESULTS

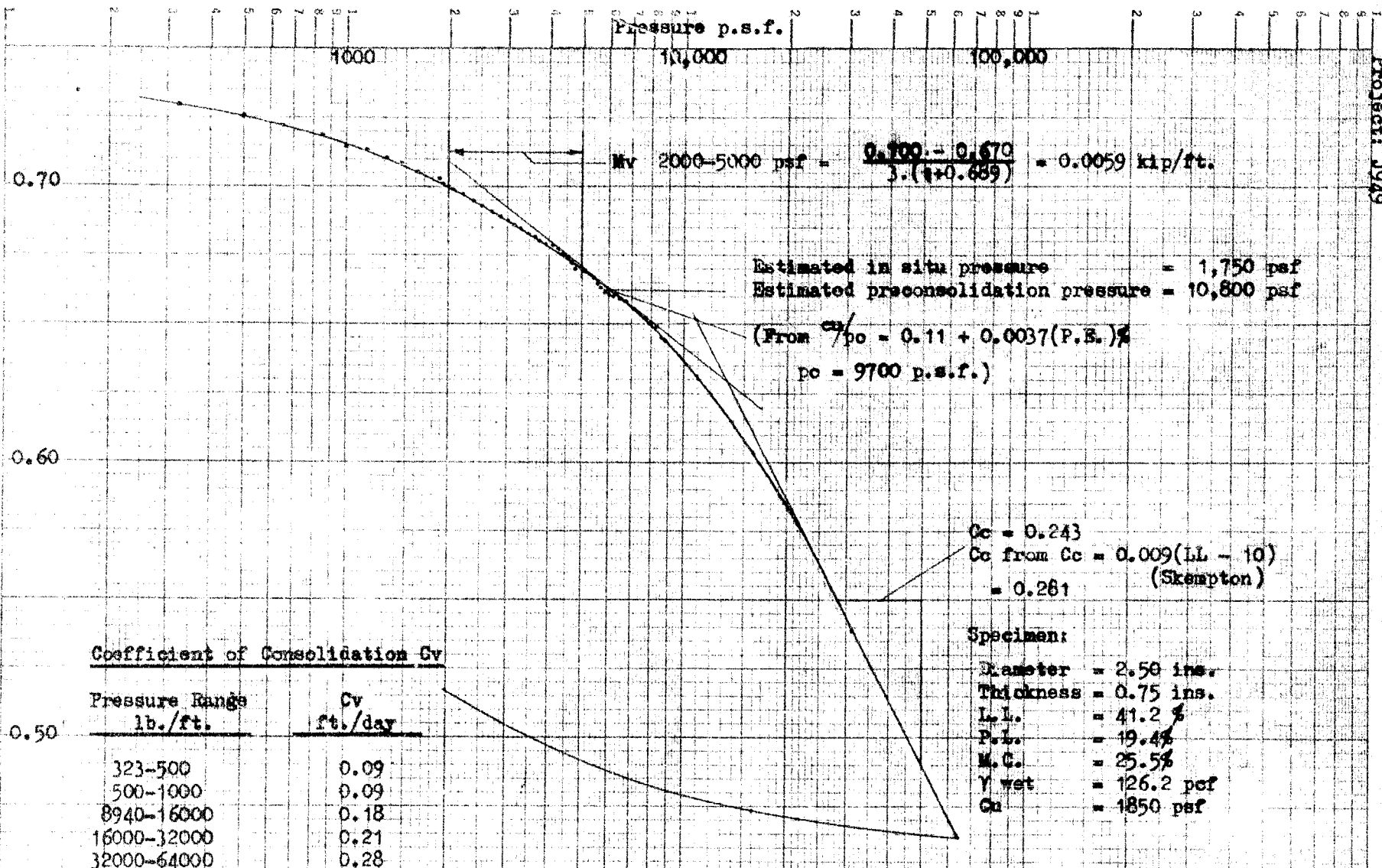
BOREHOLE 32a - 15 FT.

(W2 = Moisture Content after consolidation)

WILLIAM A. TROW AND ASSOCIATES

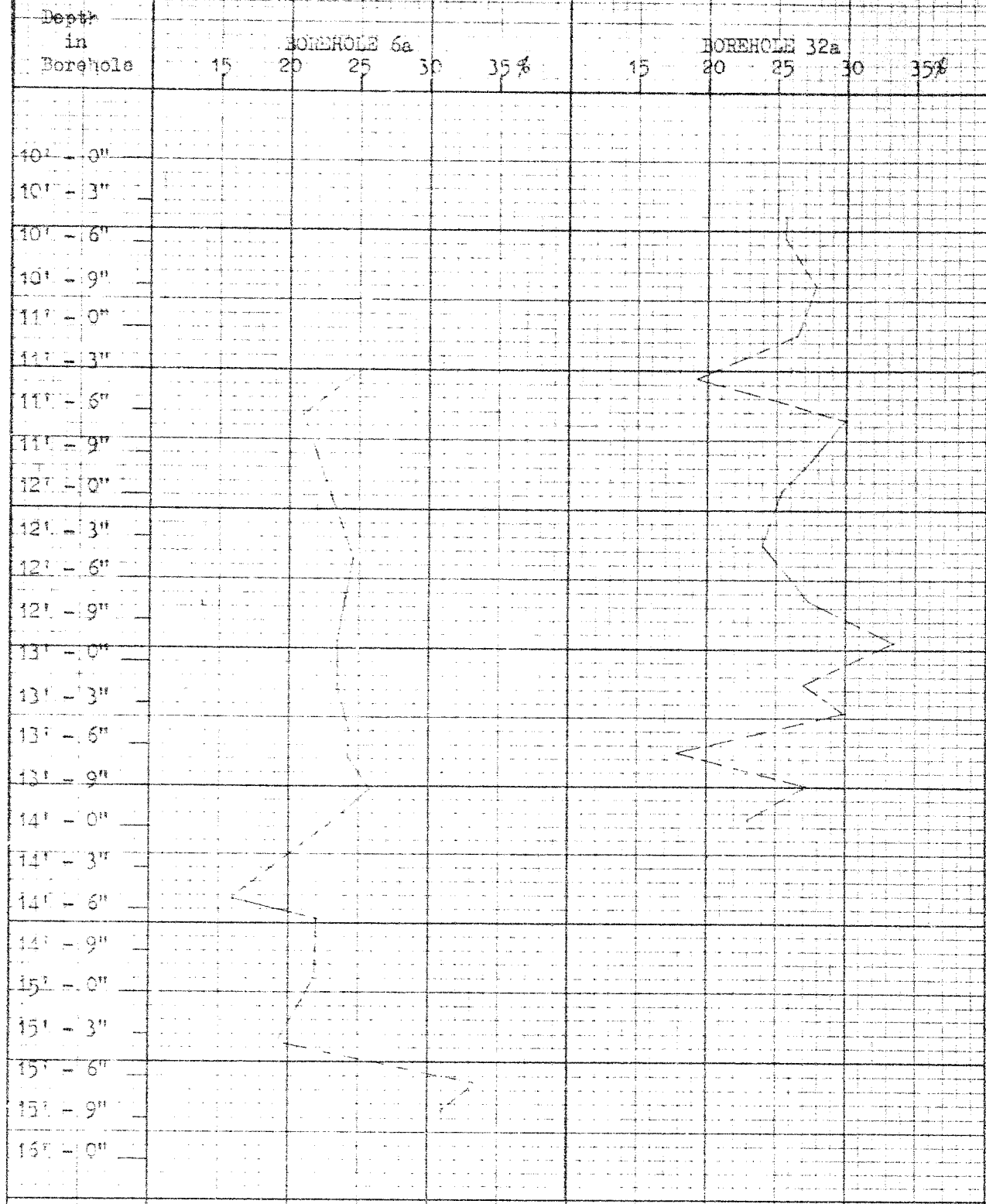






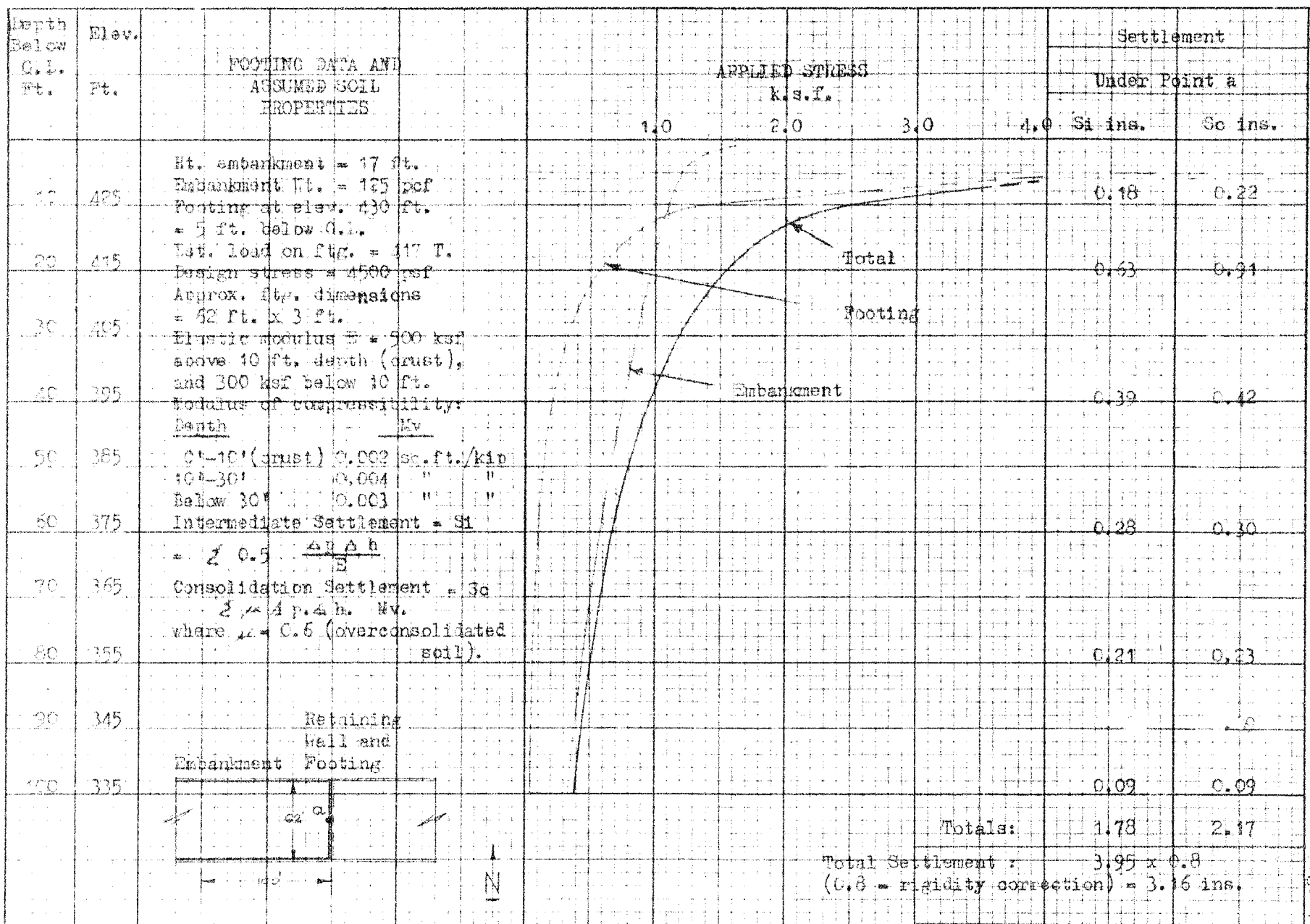
CONSOLIDATION TEST RESULT - HOLE 4, DEPTH 13 - 15 FT.

MOISTURE CONTENT PROFILE



DETAILED MOISTURE CONTENT PROFILES - BOREHOLES 6a, 32a
 TO ILLUSTRATE VARIABLE SOIL NATURE

K&E 10x10 to the inch 350-500 KUDTLEBENDER CO. MADISON, WIS.



Project: J1067

DWG. 44

IMMEDIATE AND CONSOLIDATION SETTLEMENT CALCULATIONS - WEST EMBANKMENT
AND FOOTINGS FOR EAST BOUND COLLECTOR OVER W-11 RAMP

Depth Below Footings Ft.	FOOTING DATA AND ASSUMED SOIL PROPERTIES	APPLIED STRESS UNDER CENTRE OF FOOTING k.s.f.					Settlement Under Centre Of Footings	
		1.0	2.0	3.0	4.0	5.0	Si. ins.	Sc. ins.
	Applied load per footing ≈ 100 T. Design stress = 4500 psf Approx. ftg. dimensions = 20 ft. x 25 ft. Footings located at average depth of 7 ft., hence crust depth below footings ≈ 3 ft. Elastic Modulus, $E = 500$ ksf in crust ($\approx 0-3$ ft. below ftg.), and 300 ksf below crust Modulus of compressibility Depth below ftg. M_v						0.15	0.19
10							1.10	1.58
20								
30							0.33	0.35
40								
	0'-3' (crust) 0.002 sq. ft./ksf							
	3'-23' 0.004 " "							
50	below 23' 0.003 " "						0.15	0.16
	Immediate Settlement = $\frac{q_i}{E}$ = $\frac{4500}{500} = 9$ in.							
60								
	Consolidation Settlement = S_c = $\sum \mu \Delta p \Delta h / E_v$ where $\mu = 0.6$ (overconsolidated soil)							
70							0.09	0.10
80								
90								
						Totals:	1.83	2.38
						Total Settlement : 4.21×0.8 (0.8 = rigidity correction)		3.37 ins.

IMMEDIATE AND CONSOLIDATION SETTLEMENT CALCULATIONS
FOOTINGS AT LOCATIONS "B", "C" AND "E" FOR EAST BOUND COLLECTOR OVER W-N RAMP

Project: J1067

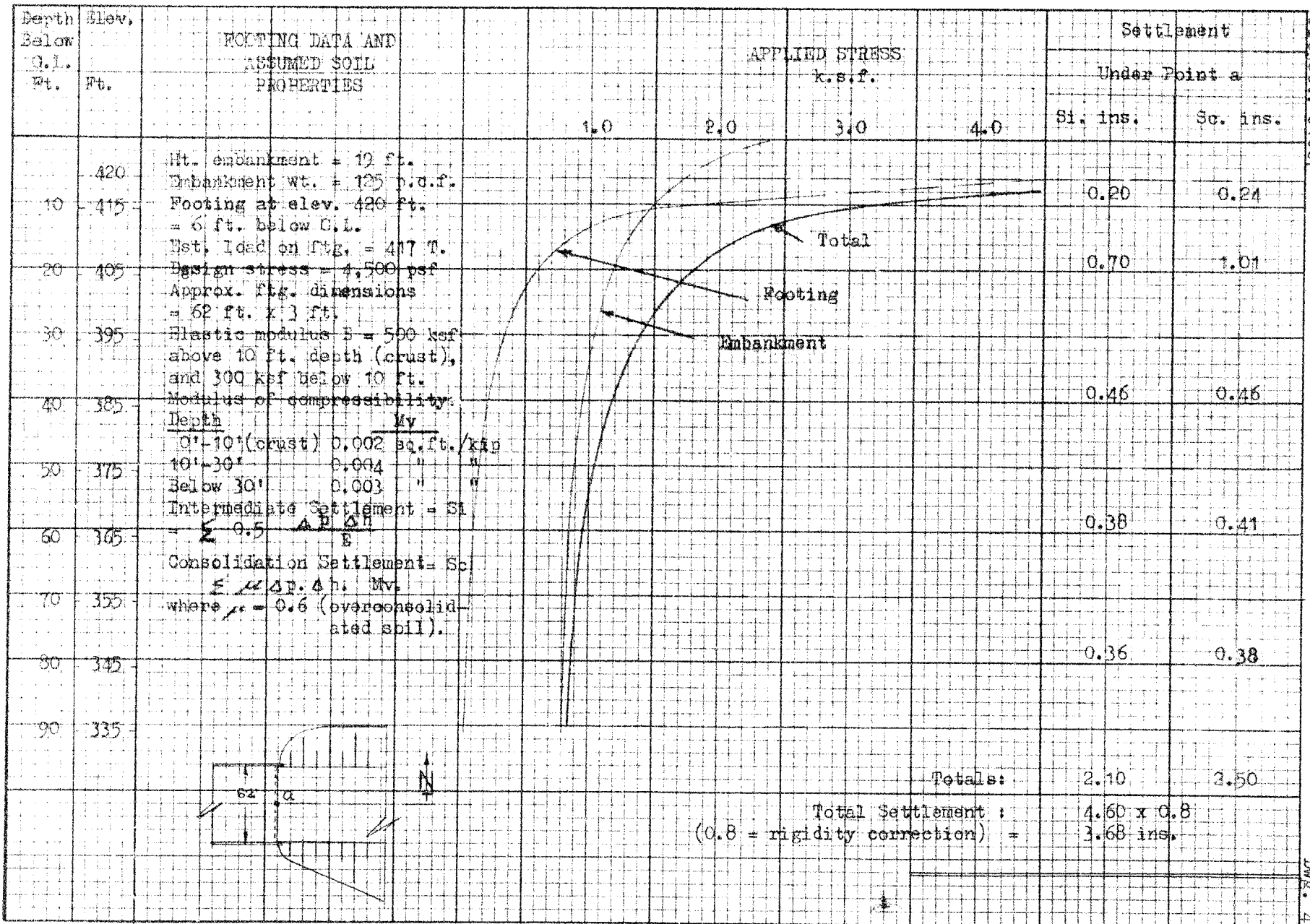
Dwg. 45

Depth Below Footings Ft.	FOOTING DATA AND ASSUMED SOIL PROPERTIES	APPLIED STRESS UNDER CENTRE OF FOOTING K.s.f.					Settlement Under Centre of Footing	
		1.0	2.0	3.0	4.0	5.0	Si. ins.	Sc. ins.
10	Applied load per footing = 550 T. Design stress = 4,500 psf Approx. ftg. dimensions = 20 ft. x 12.5 ft. Footings located at approx. Bl 422 ft. hence crust depth below ftg. ~ 1 ft. Elastic modulus, E = 500 ksf in crust (0 - 1 ft. below ftg.), and 100 ksf below crust Modulus of Compressibility:						0.05	0.06
20	Depth below ftg. 0' - 1' (crust)						0.94	1.35
30	1' - 2'						0.21	0.22
40	Below 2'							
50	Immediate Settlement = S_i = $E \cdot 0.5 \cdot \frac{\Delta p}{E_p} \cdot \Delta h$ Consolidation Settlement = S_c = $E_{vp} \cdot \Delta h \cdot M_v$ where $\Delta p = 0.5$ (overconsolidated soil)						0.08	0.09
60								
70							0.04	0.05
80								
						Totals:	1.32	1.77
						Total Settlement :	3.09 x 0.8 (0.8 = rigidity correction) = 2.47 ins.	

IMMEDIATE AND CONSOLIDATION SETTLEMENT CALCULATIONS - DIVIDED FOOTING
AT LOCATION "D" FOR EAST BOUND COLLECTOR OVER W-N RAMP

Project: J1067

DWG. 45

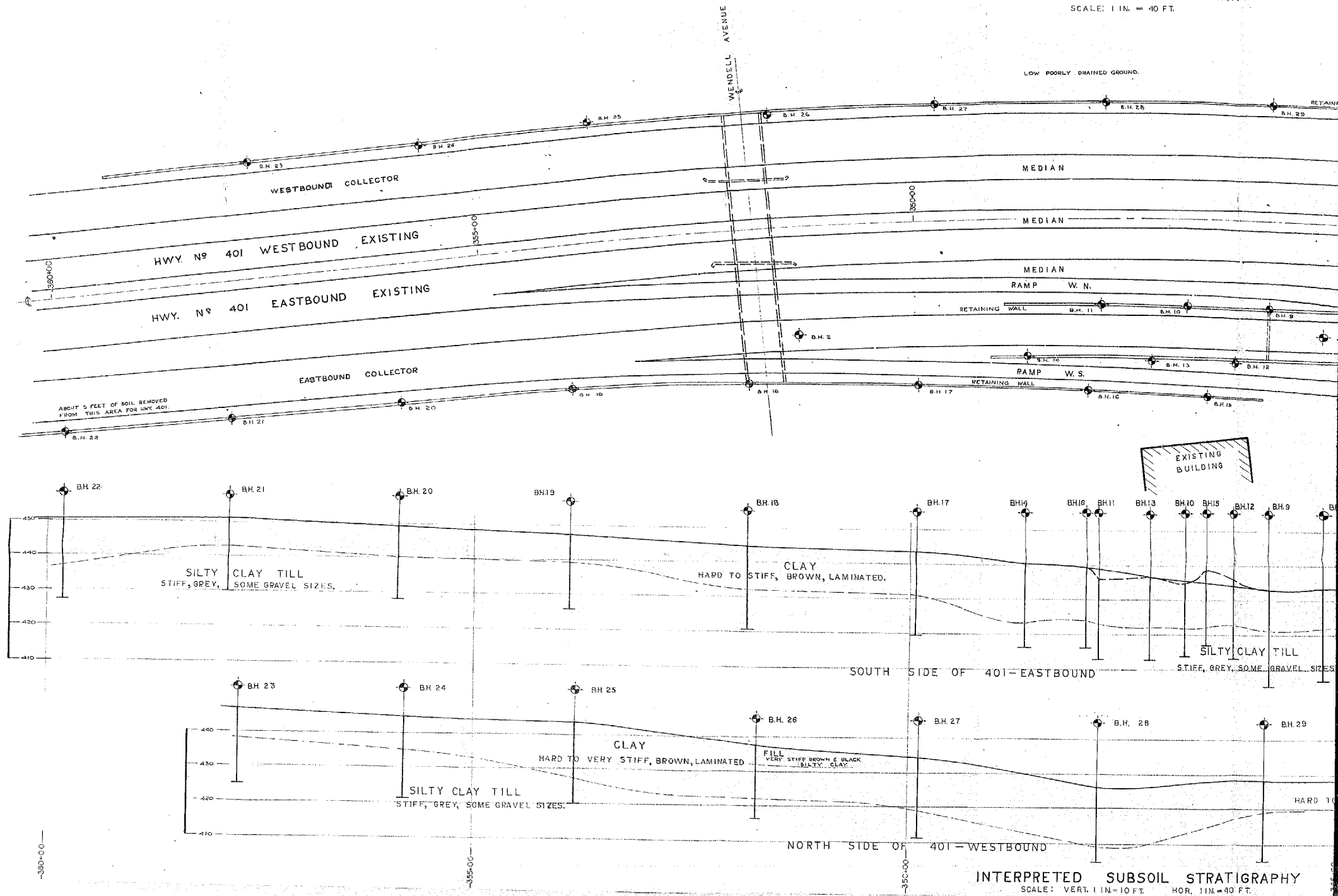


IMMEDIATE AND CONSOLIDATION SETTLEMENT CALCULATIONS - EAST EMBANKMENT
AND FOOTING FOR EAST BOUND COLLECTOR OVER W-N

63-F-220C
W.P. 233-60
WEST APPROACHES
To Hwy. 401-400
INTERCHANGE

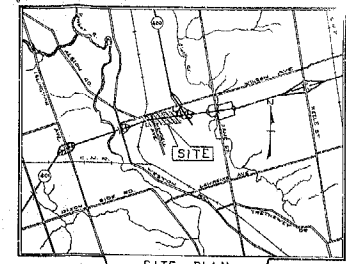
BOREHOLE LOCATION PLAN

SCALE: 1 IN. = 40 FT.



BOREHOLE LOCATION PLAN

SCALE: 1 IN. = 40 FT.



SITE PLAN
SCALE: 1 IN. = 0.5 MI.

LOW POORLY DRAINED GROUND.

RETAINING WALL

DITCH AREA ABOUT 4 FEET LOWER.

RAMP N.W.

MEDIAN

WESTBOUND COLLECTOR

MEDIAN

HWY. NO 401 WESTBOUND EXISTING

MEDIAN

HWY. NO 401 EASTBOUND EXISTING

RAMP W.N.

RETAINING WALL

DITCH AREA ABOUT 4 FEET LOWER.

CONCRETE RETAINING WALL

EASTBOUND COLLECTOR

RAMP W.S.

RETAINING WALL

RETAINING WALL

EXISTING BUILDING

SILTY CLAY TILL

STIFF, GREY, SOME GRAVEL SIZES.

CLAY

HARD TO VERY STIFF, BROWN LAMINATED.

BECOMING GREY BELOW THIS LINE

TH SIDE OF 401-EASTBOUND

OF 401-WESTBOUND

INTERPRETED SUBSOIL STRATIGRAPHY

SCALE: VERT. 1 IN. = 10 FT. HOR. 1 IN. = 40 FT.

LEGEND

- BOREHOLE.
- BOREHOLE & VANE TEST.
- VANE TEST.

W.A. TROW & ASSOC. LTD.
FOUNDATION INVESTIGATION

HWY. 401-WEST APPROACHES TO
HWY. 400 - INTERCHANGE
NORTH YORK TORONTO

PROJ. 1067 DATE APR. 1963 DWG NO 1

