

58-F-279C

Hwy. #17

CYPRESS RIVER

BA 833

TROW, SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

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884 WILSON AVE.,
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ST. 8-5921

Project: J260

November 14, 1958

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

Attention: Mr. S. McCombie

Foundation Investigation
Cypress River Crossing
T.C.H. No. 17, District No. 19

Dear Sirs:

Enclosed herewith is our report containing the results of a subsurface exploration program recently completed at the above site. In addition to a presentation of the factual data obtained from field and laboratory work carried out this report includes our comments on the type of footing support required at this site. For your convenience the findings of this report are summarized as follows:

1) The subsoil existing at the site consists of a surface layer of loose to medium dense granular deposit overlying a deep stratum of normally loaded varved silty clay. The upper granular layer exhibits sharp variations in relative density and is not considered suitable for direct footing support.

2) A pile supported foundation for the proposed structure is considered necessary at this river crossing. End bearing piles driven to refusal in the dense sand layer overlying bedrock offered very positive but expensive means of footing support. Pile lengths of the order of 85 to 90 feet will be required.

An analysis has been carried out to determine the capacity and resultant settlement of a group of short (typically 40 feet long) friction piles stopped up within the clay stratum. The analytical results obtained indicate that this type of footing support could be safely designed with a considerable saving in piling costs. This type of design should, however, be subject to the results of site static pile load tests. In this regard it is suggested that the results of a satisfactory static load test on friction piles at this site would not only result in a saving in the foundation costs of the structure but also establish a precedent design for similar structures under-

- 2 -

lain by similar soil conditions. Load test procedure should be in accordance with specifications of the National Building Code of Canada.

3) It is understood that there is to be no major change in the grade elevation and that embankments will be not more than 15 feet high. No embankment instability need be anticipated for fill hights of the order of 15 feet.

We are pleased to have carried out this investigation for you. If we can be of any assistance in clarifying factual data or substantiating the concluding comments contained in this report do not hesitate to contact our office.

Yours very truly,



Lawrence G. Soderman (P. Eng.)

LGS/kb
ENC.

DEPARTMENT OF HIGHWAYS OF ONTARIO
280 DVENPORT ROAD,
TORONTO, ONTARIO

FOUNDATION INVESTIGATION
CYPRESS RIVER CROSSING
T.C.H. No. 17, DISTRICT No. 19

Project J260

Nov. 14, 1958

Trow Soderman and Associates

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Proposed Cypress River Crossing
T.C.H. No. 17 near Nipigon, Ontario

This report covers the soils investigation carried out at the site of the proposed Cypress River crossing of the new Trans-Canada Highway. The proposed crossing is 25 miles east of Nipigon, Ontario on Highway 17. Also included in the report are recommendations for type of foundation most applicable to the site and for safe footing capacities.

Description of Site

The Cypress River Valley at the proposed crossing is wide and shallow with gently sloping banks. The relatively fast river is contained in a system of timber cribs.

It has been proposed that the new bridge be constructed in the same location as the existing structure. The existing bridge is a 60 ft. creosoted timber truss with 15 ft. approach spans and a 24 ft. roadway. Approach fills are of the order of 10 ft. high.

Field Investigation

Four borings were made at the site of the proposed structure. All field work was performed using a standard diamond drilling rig adapted for soil sampling. To facilitate sampling of the soil, 3½ inch and 2 7/8 inch diameter casings were alternately driven and washed out. By this means samples were obtained at nominal five foot intervals of depth.

Sampling of the undisturbed soil below the casing was carried out primarily using a standard 2 inch split spoon. This sampler could readily be pushed into most of the soils encountered. In sands the spoon was driven with a 140 pound hammer dropping 30 inches. The number of blows of this magnitude required to drive the spoon from 6 inches to 18 inches penetration was recorded and is the standard penetration resistance, N, of the soil. On withdrawal the sampler was dismantled and the soil classified. A representative portion was then retained in moisture-proof polyethylene bags. A small number of relatively undisturbed samples were taken in the clay encountered. Two-inch diameter thin walled steel Shelby tubes were pushed into the soil. On withdrawal the ends of the tubes were sealed to prevent change in moisture content prior to testing.

To measure the in-situ shear strength of clay soils, a vane, consisting of two thin steel plates welded in the form of a cross, was inserted into the undisturbed soil below the casing. The torque required to rotate this vane was measured. From the measured torque and known vane dimensions the shear strength of the soil was calculated. By measuring the undisturbed and remoulded strengths, the sensitivity of the soil to disturbance was found.

Drawing 1 shows the location of the boreholes with reference to the existing structure. The depths at which samples were taken, in-place shear test results, laboratory test results and standard penetration test results are presented on the borehole profiles included with this report.

Soil Types Encountered

Since all drilling was performed on the shoulder of the existing roadway the first soil encountered in all holes was the loose to medium dense sand, gravel and boulder fill making up the existing highway grade. This granular layer apparently extends to an average depth of some 22 feet below present road grade. While the fill height appears to be only 10 feet, excavations for timber cribbing would account for much of the greater depth of this deposit. The river bottom is sand and gravel with boulders so it is possible that this top deposit is natural to the area and is in its undisturbed state at depth.

Underlying the coarse granular materials is a layer of light grey essentially fine, clayey sand. Normally 2 to 4 feet in thickness, it was proven to be 9 feet thick in hole 1. Elevation 603 denotes the lower boundary of this light grey sand and the upper horizon of the soft to medium stiff, light grey, varved silty clay.

The varving of the silty clay stratum is generally $\frac{1}{2}$ to 1 inch total thickness with the clay portion making up approximately 40%. At and below elevation 556 fine sand seams varying in magnitude from a few feet to hairline cracks were noted in the clay.

Dense red sand and gravel was encountered below elevations 530 and 533 at 3 out of the 4 locations drilled. This stratum was reached at elevation 537 in hole 2. Bedrock consisting of red and grey metamorphosed sediments was proven at elevations varying from 526 to 529. This rock was drilled for a distance of 10 feet in holes 1 and 2 and 5 feet in the remaining two holes. Drawings 2, 3, 4 and 5 are the detailed logs of the borings.

Lab tests consisting of natural density, natural water content and Atterberg limits were carried out on the Shelby tube samples. Unconsolidated undrained tri-axial tests were performed to help ascertain the shearing resistance of the clay soils. The results of these determinations are shown in the borehole logs. The results of consolidation tests on the silty clay and clay phases of the soils are presented on drawing 6.

In summation, the soil system encountered consists of coarse granular soils - sand, gravel and boulders - extending to depths of 16 to 23 feet below present highway grade, underlain by a thin seam of very fine sand which lies on top of a thick varved clay deposit. The highly compressible medium stiff clay deposit extends to a red sand and gravel layer at elevation 530 to 537 which is found immediately above rock at elevation 526 to 529. Drawing 1 shows projected soils profiles for the area investigated.

The water table in the soils corresponds to the water level in the river which was 14 feet and 1 inch below present highway grade during this investigation. This is based on observations made in holes 2 and 3 several days after their completion.

Foundation Considerations

Spread footings cannot be used in this location because of the following conditions: Although the granular soils near the surface are essentially medium dense, density variations are great and it is necessary to consider the lowest penetration values. The loose state of relative density is the condition limiting allowable footing load in this deposit. The clay stratum is too soft to provide much support for spread footings in the granular soils or in the clay itself. Settlements would be excessive. An additional consideration is the prevention of footing undermining by scour. This would necessitate deep footings close to the clay or some type of permanent scour protection in the form of sheet piling, cribbing or rip-rap.

From the foregoing it appears that a piled foundation is the only type that will provide competent support. Piles driven to develop frictional resistance in the clay or to refusal at the dense sand - bedrock elevation would supply the required capacity.

End bearing piles driven to refusal would have a capacity dependent upon the type of pile, i.e. wood, concrete, steel, and upon the cross-sectional area of the pile.

The capacity of friction piles in varved material is more problematical and does not readily lend itself to a rigorous analysis. It will be noted from the borehole logs that the shear strength determined by the in-situ vane test gives results substantially greater than the tri-axial tests on the same material. This seems reasonable when it is remembered that the area of shear in the vane test is defined during test and is the surface generated by the turning vane. This surface is perpendicular to the varving, this is an average measure of the actual strengths of the pure clay and silty clay portions. Shearing in the tri-axial test takes place on the plane of least resistance. This should be a minimum value. The tri-axial test specimen is also slightly disturbed by sampling.

Since the piles will be placed perpendicular to the varving, it appears reasonable to base capacity predictions on the results of vane tests applying an adequate factor of safety. The net result should be checked against the tri-axial strengths. With regard to safe allowable shear strength for pile capacity determinations, reference is made to work by Tomlinson reported in the Fourth International Conference on Soil Mechanics. From this it appears that the ultimate adhesion of test piles in clays comparable in shear strength is approximately 80% of that shear strength. Applying a factor of safety of two to the net adhesion value results in the safe adhesion value used in design.

Apparent shear strength from vane tests	800 psf
Ultimate adhesion (Tomlinson 1958), 800×0.8	640 psf
Apply factor of safety ≈ 2 , allowable adhesion	300 psf.

The capacity of an 8 inch diameter pile driven 30 feet into the clay can be calculated as follows:

- 4 -

$$Q_{\text{allowable}} = \text{end bearing value} + \text{side friction}$$

$$= \frac{c N_c A_p}{F} + c_a A_s$$

where c = shear strength of the soil at the tip
 N_c = bearing capacity factor = 9
 A_p = cross-sectional area of the pile tip
 F = factor of safety, normally 3
 c_a = allowable adhesion
 A_s = surface area of the pile in the clay

therefore

$$Q_a = \left(\frac{800 \times 9 \times \pi \times 0.33^2}{3} + 300 \times \pi \times 0.67 \times 30 \right) \frac{1}{2000}$$

$$= 10 \text{ tons.}$$

By the same means capacity for piles of various sizes and penetration can be solved. The following table has been prepared on this basis:

Penetration	D i a m e t e r		
	8 in.	10 in.	12 in.
30 ft. tip elev. 573	10 tons	12 tons	15 tons
40 ft. tip elev. 563	13 tons	16 tons	20 tons

A common spacing for wood piles under bridge piers is 3 rows of piles at 3 foot centres. Assuming a 30 foot wide roadway, 30 piles would probably be spaced in three rows.

A pier load of 200 tons appears reasonable for a bridge of this size. This would result in a maximum load of the order of 7 tons per pile. Thus, an 8 inch pile driven 30 ft. into the clay would provide ample capacity. This represents a 40 ft. pile being driven from the present river level.

The amount of settlement to be expected with a pile type foundation must be estimated. Due to the varved nature of the clay, sampling disturbances and uncertain stress distribution from the piles, settlement analysis can only be a rough guide to the actual. The only competent method of estimating settlements under these conditions is by carrying out a pile loading test at the site.

Considering the values of compression index from the consolidation tests carried out on the clay, making allowances for sampling disturbance, relative thickness of the clay and silty clay varves etc., it appears that a reasonable value of compression index, C_c is 0.7. Assuming that the pile loading can be assumed to act at a point $1/3$ of the length of the pile in the clay measured from the bottom, the amount of settlement to be expected with a pile group can be estimated.

An example of this determination is given by the following calculation. Consider 40 foot piles driven into the clay resulting in tips at elevation 573. The total dead load on the group of 30 piles is estimated to be 150 tons. Due to interaction of the piles, the load can be considered to be represented by a rectangular section 30 feet by 6 feet at elevation 583. Assuming that the clay extends to elevation 533, five 10 ft. thick layers of clay can be considered to exist under the equivalent rectangular load. The total settlement is the sum of the settlements calculated from each 10 foot layer. An example calculation for the settlement of the layer between elevation 583 and 573 is as follows:

$$S = \frac{C_c}{1 + e_o} \times T \times \log_{10} \left(\frac{P_o + P}{P_o} \right)$$

- where C_c = 0.7 as explained above
 e_o = initial void ratio, assume 1.1
 T = thickness of layer considered
 P_o = existing intergranular pressure at the mid-height of T
 P = additional load at mid. T assuming a 1 : 2 distribution of load from point of application.

Substitution in the above expression gives:

$$S = \frac{0.7}{1 + 1.1} \times 120 \times \log_{10} \frac{2390 + 770}{2390}$$

$$= 5 \text{ inches.}$$

By successive calculations the settlements due to consolidation are found for each layer and totaled to equal 8.5 inches. Assuming an elastic compression of 20%, the net settlement would equal 10 inches.

Similarly piles 50 feet in length penetrating 40 feet into the clay to elevation 563 could be expected to settle 8 inches.

Only a rough estimate of time versus settlement can be made for a pile group due to the varved nature of the soils and the presence of sand seams below elevation 558 or so. An average coefficient of consolidation, C_v , from the tests carried out in this investigation appears to be 0.11 square feet per day for the range of loading that will occur in the soil. Assume that time of settlement will be governed

by a 50 foot clay layer from elevation 603 to 553 which represents boundary conditions formed by the top of the deposit and the presence of sand drainage paths below elevation 558. The time to 90% consolidation will be of the order of 13 years with $\frac{1}{2}$ of the settlement occurring in 3 years.

Recommendations

The soils encountered at this site consist of loose to medium dense granular deposits overlying a deep bed of clay. The only type of foundation that appears feasible is a piled foundation.

Long end bearing piles driven to refusal in the dense sand overlying bedrock or to bedrock itself will provide support dependent on the type and size of pile used. The length of pile required will be of the order of 85 feet if driven from present river level.

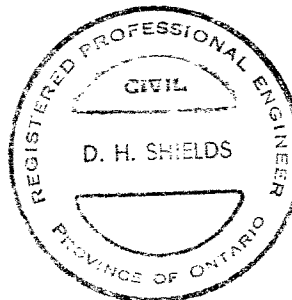
Piles can be stopped in the clay and would develop support by friction along their surfaces. Numerous short piles could be used to develop the total reaction required. Estimates of load capacity of various pile sizes are given in the body of the report.

Settlements of a pile foundation in the clay are difficult to estimate precisely but could conceivably be of the order of 1 foot. The time to 90% settlement is estimated to be 13 years with one-half (6 inches) of the settlement occurring in the first three years.

The great depth to bedrock appears to warrant further investigation into the capacity and settlement characteristics of short (i.e. piles 40 to 50 feet below river level) piles by means of load tests. It is recommended that both piles driven to elevation 573 and 563 be tested to ultimate failure. The test should conform to the procedures outlined in the National Building Code.

DHS/kb

for *L. G. Soderman*
D.H. Shields (P. Eng.)

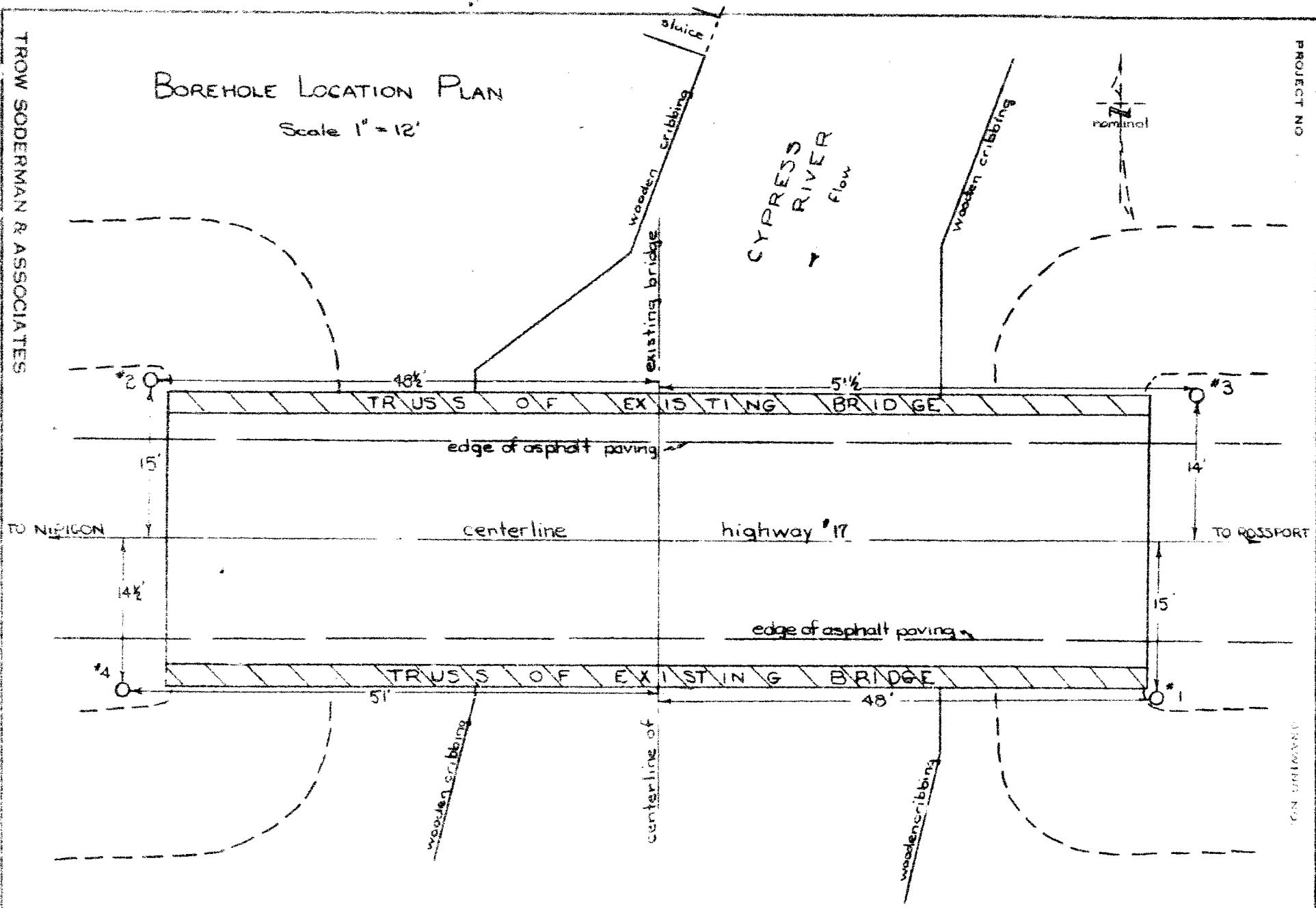


BOREHOLE LOCATION PLAN

Scale 1" = 12'

PROJECT NO.

DRAWING NO.



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Cypress River Crossing T.C.H. No. 17

LOCATION 24 miles east of Nipigon, Ont.

HOLE LOCATION See dwg. No. 1

HOLE ELEVATION AND DATUM 628.2

C.L. of bridge C.L. roadway = 627.9

BOREHOLE NO. 1

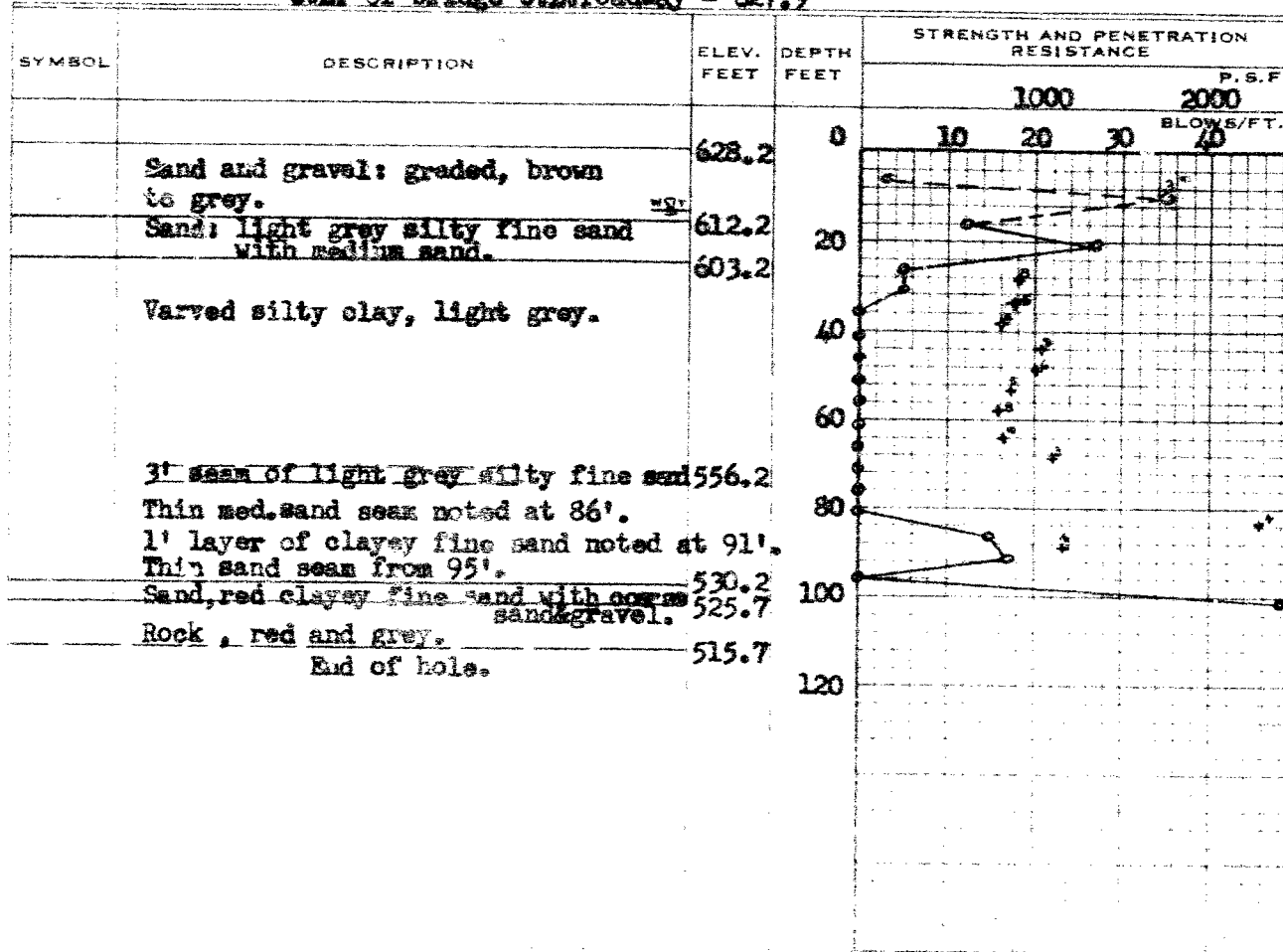
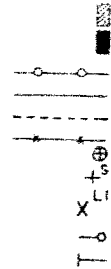
FIELD SUPERVISOR D.S.

DRILLER E.S.

PREP. D.S.

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



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	
	SS7	
	SS8	
	SS9	
	SS10	
	SS11	
	SS12	
	SS13	
	SS14	
	SS15	
	SS16	
	SS17	
	SS18	
	SS19	
	SS20	

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Cypress River Crossing T.C.H. No. 17

LOCATION 24 miles east of Nipigon, Ont.

HOLE LOCATION See dwg. No. 1

HOLE ELEVATION AND DATUM 627.6

C.L. Bridge C.L. Roadway = 627.9

BOREHOLE NO. 2

FIELD SUPERVISOR, D.S.

DRILLER H.J.

PREP. D.S.

LEGEND

- 2 1/2 DIA. SPLIT TUBE
2 1/2 SHELBY TUBE
2 1/2 SPLIT TUBE
2 1/2 DIA. CONC.
CASING
2 1/2 SHELBY
1 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	
				1000	2000
				P. S. F. BLOWS/F.T.	
	Sand gravel and boulders, graded, brown to grey.	627.6	0		
	Sand - light grey fine to med. sand	606.6	20		
	with coarse sand	602.6			
	Varved silty clay - light grey.		40		
	6" clayey fine sand seam noted at 70'		60		
	then sand seam noted at 75'.		75		
	Silt, clay and sand layers from 80'.		80		
	Sand - fine to med. red, clayey sand	537.1	100		
	with fine to coarse gravel.	528.1			
	Rock - red and grey.	517.8	120		
	End of hole.				

[illegible]

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Cypress River Crossing T.C.H. No. 17
 LOCATION 24 miles east of Nipigon, Ont.
 HOLE LOCATION See dwg. No. 1
 HOLE ELEVATION AND DATUM 628.0
 C.L. Bridge C.L. Roadway = 627.9

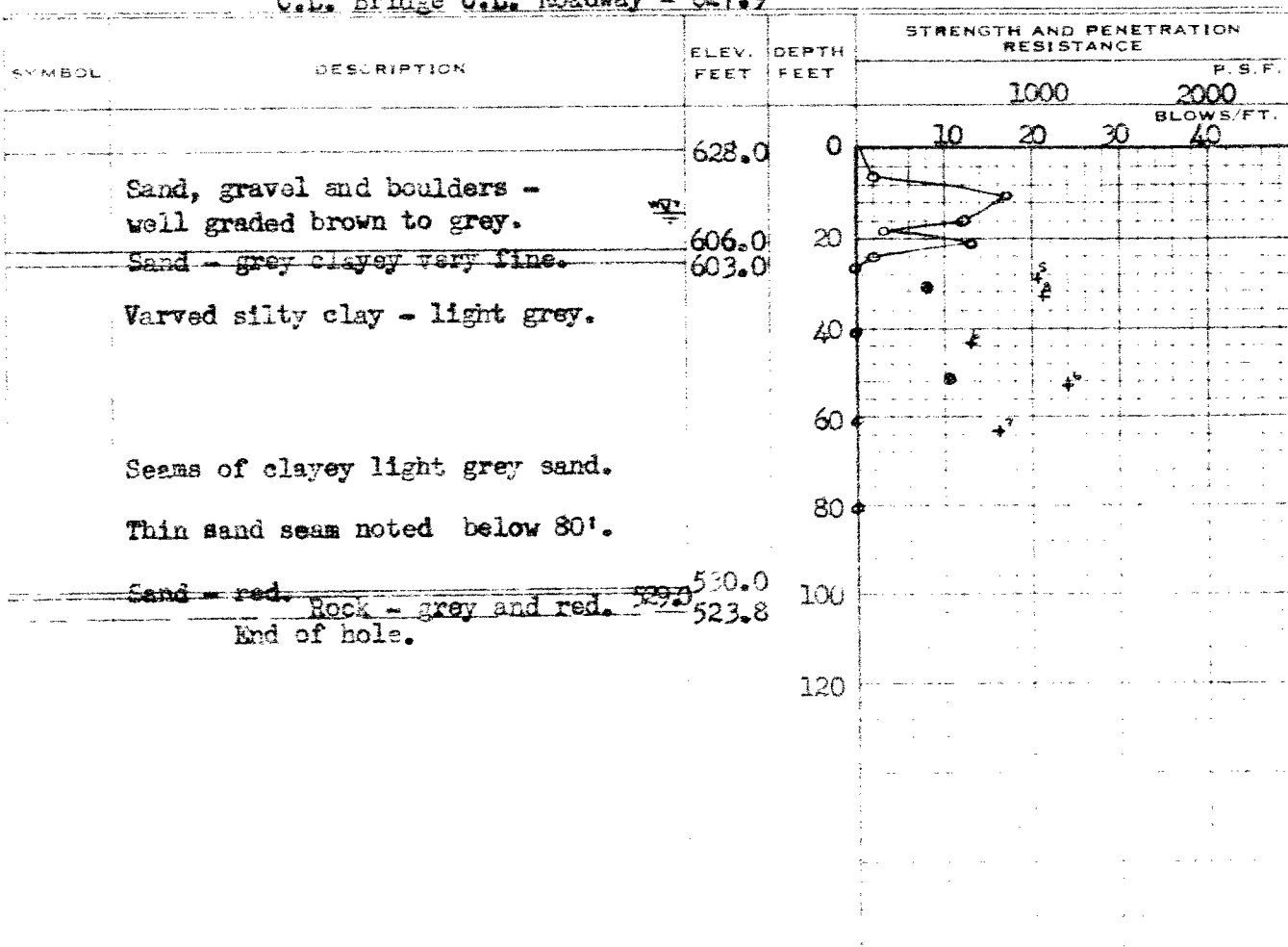
BOREHOLE NO. 3
 FIELD SUPERVISOR D.S.
 DRILLER E.S.
 PREP. D.S.

DRAWING NO.

4

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



CONSISTENCY			SAMPLE	NATURAL
MOIST. CONTENT- % DRY WT.				UNIT WT
40	50	60		P.C.F.
			SS1	89
			SS2	
			SS3	
			SS4	
			SS5	
	X ₂₅	X ₅₀	TW6	98
			SS7	
X ₂₅	X ₅₀	X ₆₀	TW8	
			SS9	
			TW10	
			SS11	

Natural Moisture: Xcs - total sample
 Xc - clay portion
 Xs - silt portion

PROJECT NO. J260

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

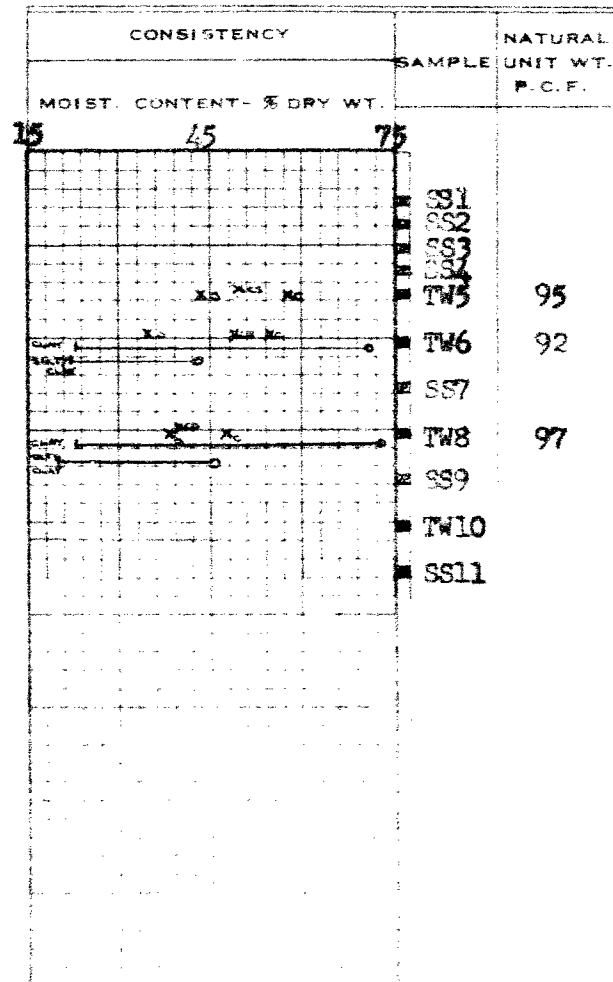
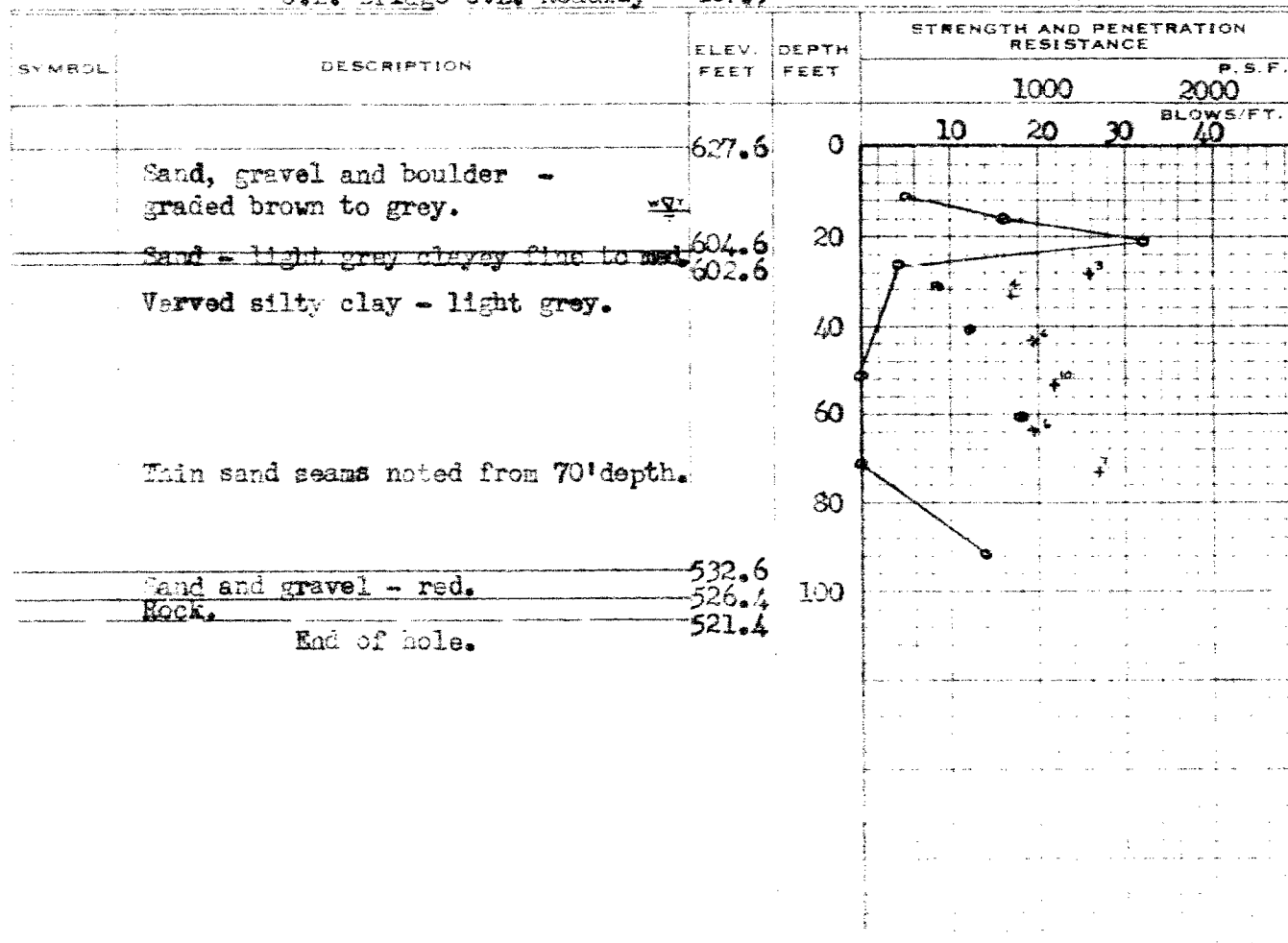
PROJECT Cypress River Crossing T.C.H. No. 17
 LOCATION 24 miles east of Nipigon, Ont.
 HOLE LOCATION See dwg. No. 1
 HOLE ELEVATION AND DATUM 627.9
 C.L. Bridge C.L. Roadway = 627.9

BOREHOLE NO. 4
 FIELD SUPERVISOR D.S.
 DRILLER H.J.
 PREP. D.S.

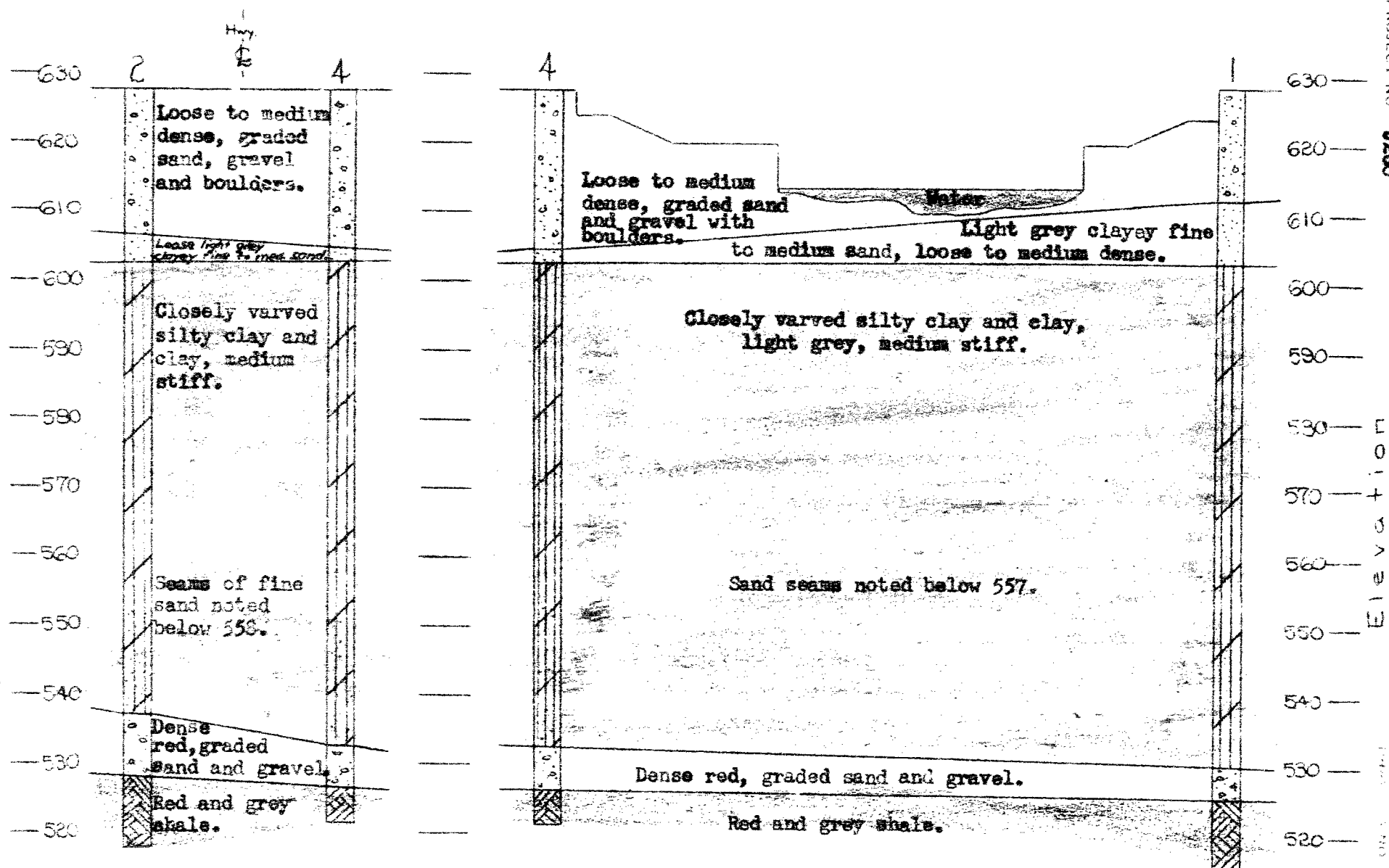
DRAWING NO. 5

LEGEND

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

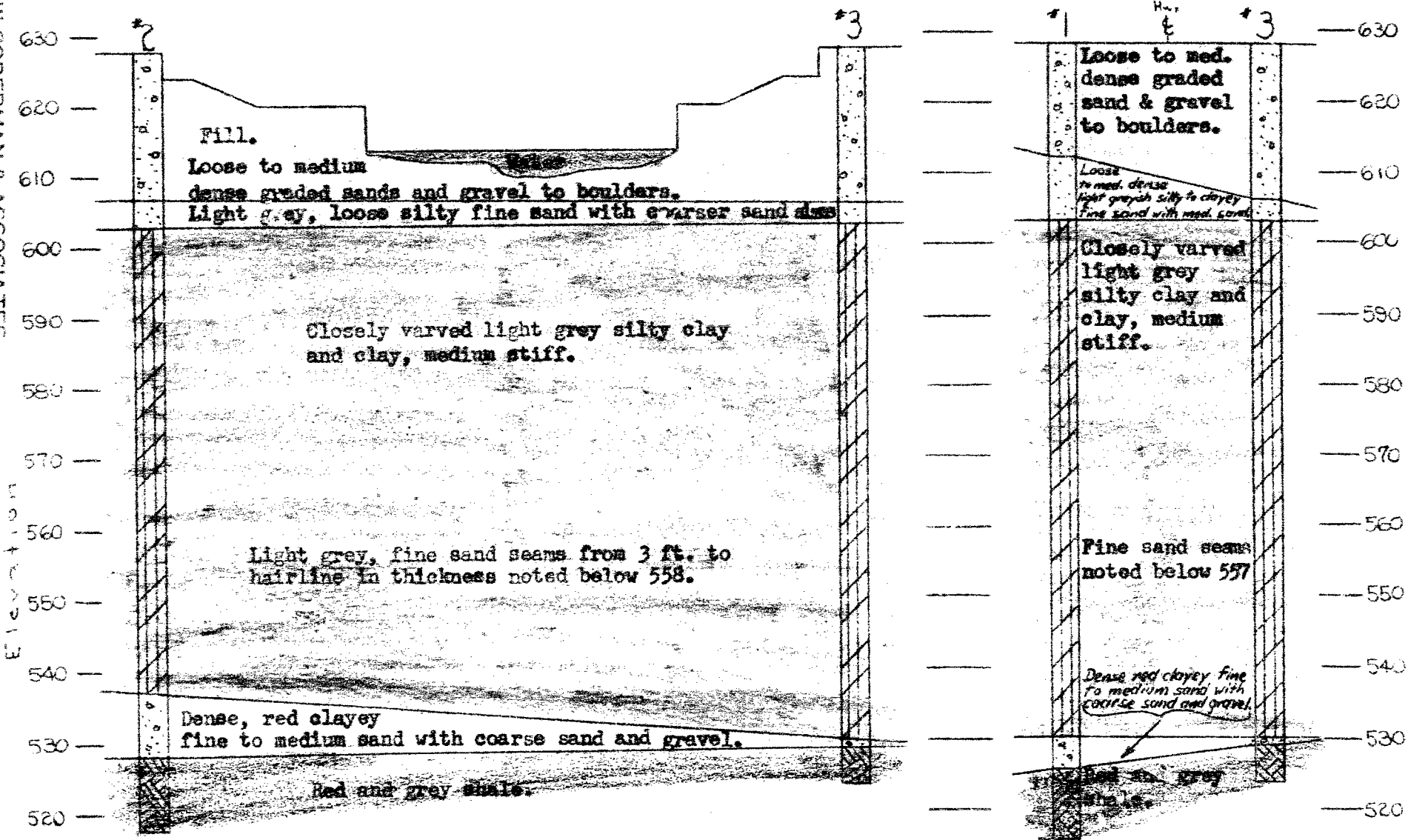


Natural Moisture: Xes - total sample
 Xc - clay portion
 Xs - silt portion

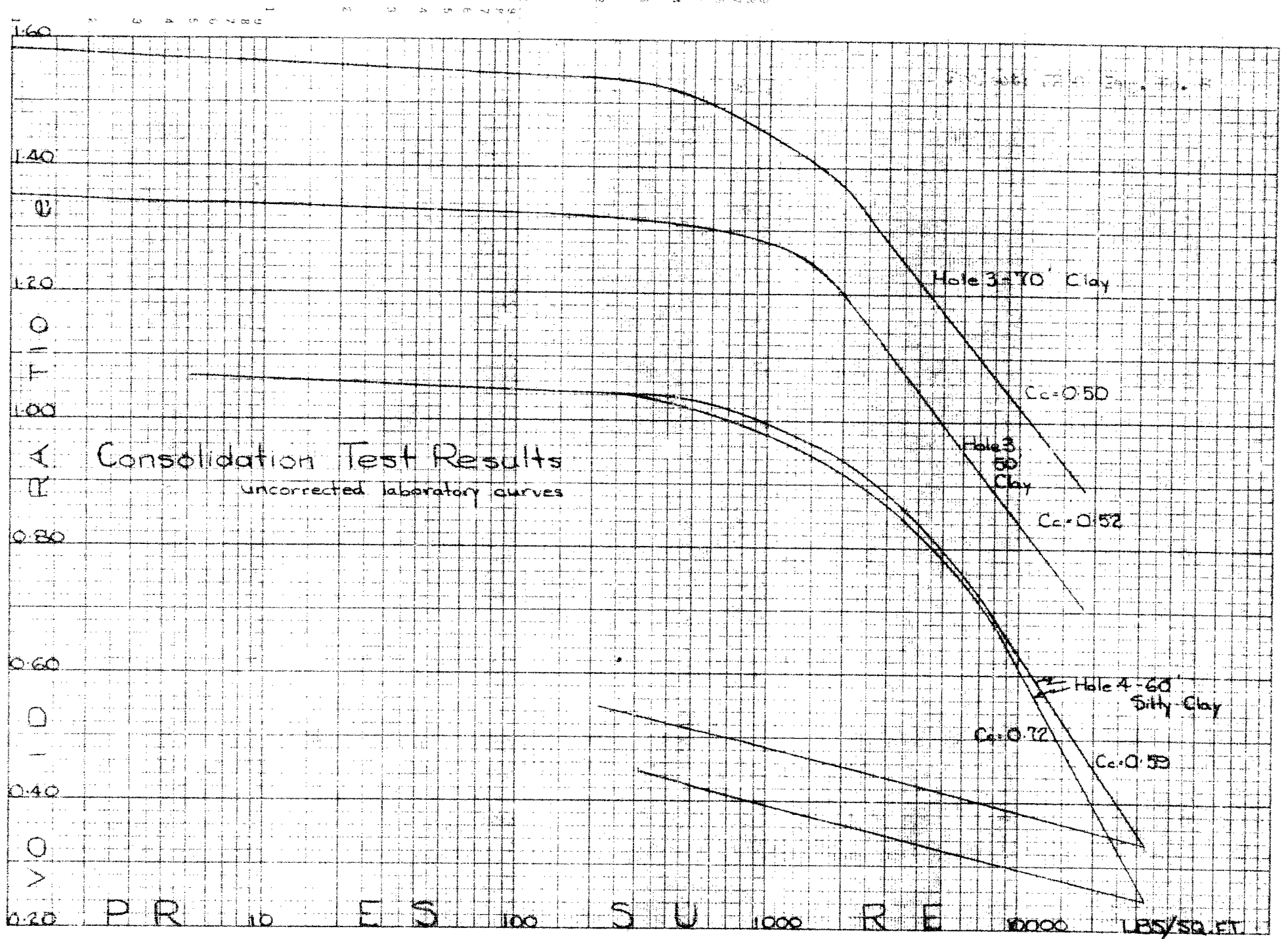


PROJECTED SOIL PROFILES

SCALE 1"=20'



PROJECTED SOIL PROFILES
SCALE 1" = 20'

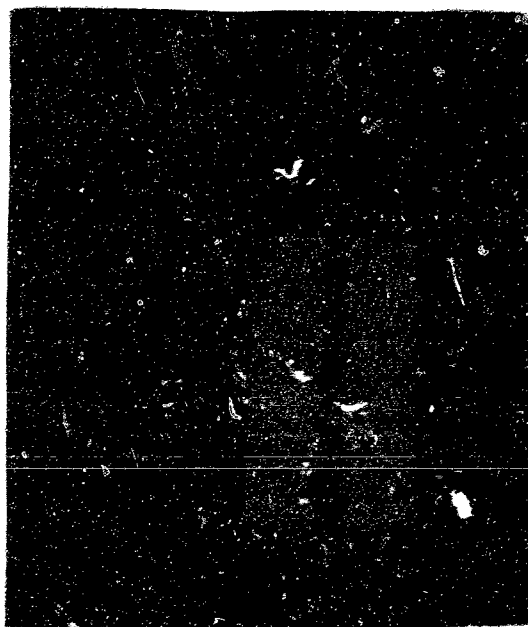




West Abutment Location
View Facing North

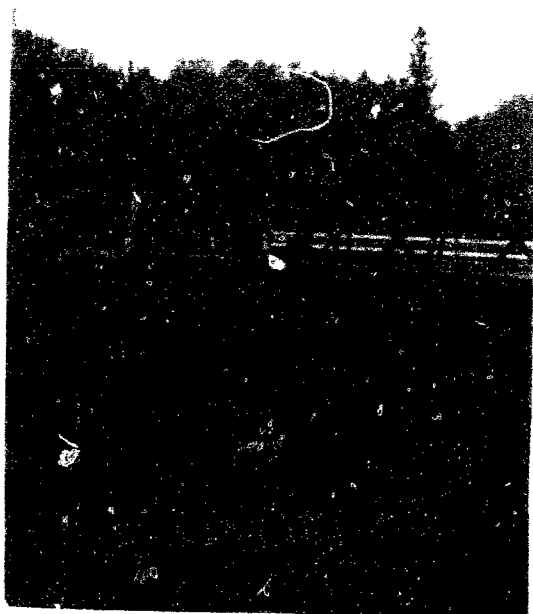


East Abutment Location
View Facing North



Sample of Partly Dried
Varved Silty Clay

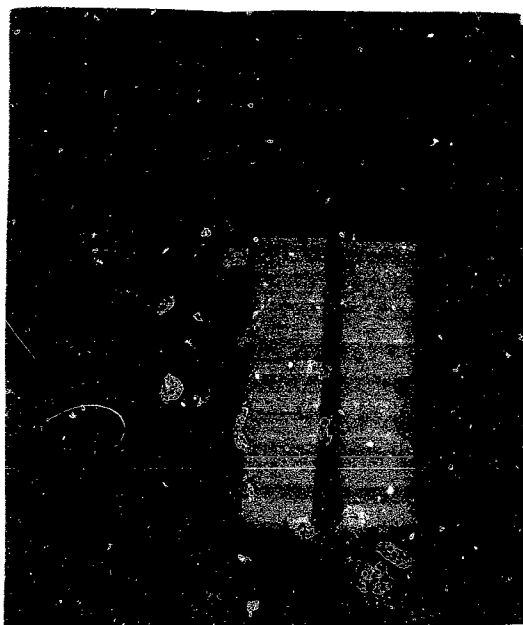
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West Abutment Location
View Facing North



East Abutment Location
View Facing North



Sample of Partly Dried
Varved Silty Clay

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